ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

BEHAVIOR OF FIBER REINFORCED SAND UNDER STATIC LOAD

M.Sc. THESIS

Ahmad DARVISHI

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Soil Mechanics and Geotechnical Engineering Programme

JANUARY 2014

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Thesis Advisor: Prof. Dr. Ayfer ERKEN

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<u>İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ</u>

FİBER İLE GÜÇLENDİRİLMİŞ KUM ZEMİNLERİN STATİK YÜKLER ALTINDAKİ DAVRANIŞLARI

YÜKSEK LİSANS TEZİ

Ahmad DARVISHI
(501101315)

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Tez Danışmanı: Prof. Dr. Ayfer ERKEN

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Ahmad DARVISHI, a **M.Sc.** student of ITU **Graduate School of Science Engineering and Technology** student ID **501101315**, successfully defended the **thesis** entitled "**BEHAVIOR OF FIBER REINFORCED SAND UNDER STATIC LOADS**", which he prepared after fulfilling the requirements specified in the associated legislations, before the jury whose signatures are below.

Thesis Advisor :	**Prof. Dr. Ayfer ERKEN** İstanbul Technical University	....
Jury Members :	**Assoc. Prof. Dr. Ayşe EDİNÇLİLER** Boğaziçi University	....
	Assist. Prof. Dr. Berrak TEYMUR İstanbul Technical University	....

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To my family,

FOREWORD

Throughout history, man are confronted with major problems. Among these problems, natural disasters endanger people's lives and it has caused to loss of their lives and property. In order to solve these problems, eliminate or reduce hazards many scientific studies have been carried out. Among the problems caused by natural disasters earthquake is one of the most dangerous disasters so to reduce the dangers arising from that a lot of research has been done in this area a nd with the development of technology in more detail in this research is carried out in a way.

In this study, fiber-reinforced sandy soil and their behavior under static loads are handled.

This master thesis has been prepared for submission to Istanbul Technical University, Civil Engineering Department, and Soil Mechanics and Geotechnical Engineering Program.

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Ahmad DARVISHI
(Civil Engineer)

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ABBREVIATIONS

A_R	: Cross section area of all fibers crossing the plane
A	: total cross-sectional area of the failure plane
c	: Cohesion
c'	: Effective Stress cohesion intercept
c_a	: Apparent Cohesion
c_{ap}	: Apparent Cohesion at peak
c_{ar}	: Apparent Cohesion at residual
C_u	: Coefficient of Uniformity
C_c	: Coefficient of Curvature
D_{10}	: Effective size
D_{30}	: Diameter corresponding to 30% finer
D_{60}	: Diameter corresponding to 60% finer
D_r	: Relative Density
e	: Void Ratio
E	: Elasticity modulus, Young's modulus
E_{max}	: Maximum Elasticity modulus, Maximum Young's modulus
E_r	: Fiber Modulus
G	: Shear Modulus
i	: Fiber Orientation angle
k	: Shear Distortion
L	: Length
u	: Pore Water Pressure
V_{pur}	: Volume of water at unreinforced soil samples
$V_{f.r}$	: Volume of Water at reinforced samples
w_f	: Weight of Fiber
w_s	: Weight of Soil
σ_n	: Normal Stress
τ	: Shear Stress
ΔL	: Horizontal Displacement
ϕ	: Shear Strength Angle
ϕ_p	: Peak Shear Strength Angle
ϕ_r	: Residual Shear Strength Angle
ρ	: Fiber Content by the percentage of weight

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BEHAVIOR OF FIBER REINFORCED SAND UNDER STATIC LOADS

SUMMARY

Construction of building and other civil engineering structures on weak or soft soil is highly risky because such soil is susceptible to differential settlements, poor shear strength, and high compressibility. In civil engineering in order to make a safer and more economical construction, methods of soil improvement is gaining importance day by day by using new materials and technology. Soil improvement methods to improve engineering properties of soils are developing. Soils were always exposed to various loads and developing technology in the last century due to increasing population and a high proportion of soils are exposed to static and dynamic loads. In order to implement a construction project safer and more economical it has been used various ground improvements, depending on the ground type methods. Various soil improvement techniques have been used to enhance the engineering properties of soils. In the recent years by industrial development soil, improvement methods are developed by using of various additives materials. Soil reinforcements by fiber material is considered an effective ground improvement method because of its cost effectiveness, easy adaptability, and reproducibility. One of the most commonly used soil improvement type is the addition of substances such polypropylene fibers. Hence, in the present investigation, polypropylene fiber has been chosen as the reinforcement material, and it was randomly included into the sandy soil at three different percentage of fiber content, i.e., 0.1%, 0.5% and 1.0% by weight of soil.

Mainly, fiber-reinforced soil can be defined as mixing soil with discrete elements, fibers that are produced naturally or artificially of several materials as account, lingocellulosed, palm, straw, polyester, steel and polypropylenes. The soil and fiber mixture is not only combined with randomly distribution, but also they can be placed in layers. In this study the behavior of the soil under static loads are discussed and scrutinized by adding unadulterated as well as fiber. Conducted laboratory tests are Direct Shear test and consolidation test. The main objective of this research is to focus on the strength behavior of soil reinforced with randomly included VHP (Virgin Homopolymer Polypropylene) fiber. The stress- strain response under monotonic loading and shear strength parameters are determined for unreinforced and reinforced sand specimens. While the type of fiber and sand is kept constant, effect of, different parameters are tested in each test. The main aim is to find the effect of fiber inclusion on the behavior of tested sand. In first type of the experimental study, direct shear tests are performed on unreinforced and reinforced samples. In the second part of that, permeability test was applied on samples to obtain Void Ratio and Relative densities of samples. The effect of fiber inclusion on the shear strength parameters are discussed by using the experimental results obtained from direct shear tests. As a conclusion, the results obtained from laboratory work presented that fiber inclusion improve static behavior of loose sand. It also makes soils more resistance against to earthquake-induced liquefaction, depending on the test condition and fiber content.

FİBER İLE GÜÇLENDİRİLMİŞ KUM ZEMİNLERİN STATİK YÜKLER ALTINDAKİ DAVRANIŞI

ÖZET

İnşa edilen veya edilecek olan yapıların yumuşak ve zayıf zeminlerde yapılması hem maddi hem de can güvenliği açısından birçok risk taşımaktadır zirâ bu tür zeminler oturmalara meyilli, düşük kayma direnci ve büyük oranda sıkışa bilirlik gibi problemler potansiyeline sahipler. Zeminlerin mühendislik özelliklerinin arttırılması ve zeminlerin iyileştirilmeleri için birçok yöntem kullanılmaktadır. Zemin mekaniği mühendisliğinde daha güvenli ve ekonomik bir proje yapmak için zemin iyileştirme yöntemleri gün geçtikçe önem kazanmaktadır. Zemin iyileştirme yöntemleri zeminlerin mühendislik parametrelerini iyileştirmek amacıyla geliştirilmiş olan yöntemlerdir. Zeminler her zaman çeşitli yüklere maruz kalmışlardır ve son bir asırda gelişen teknoloji ve artan nüfusa bağlı olarak zeminler yüksek oranda statik ve dinamik yüklere maruz kalmaktadırlar. Daha güvenli ve ekonomik bir inşaat projesi uygulamak için zemin türlerine bağlı olarak değişik zemin iyileştirme yöntemleri kullanılmaktadır.

Diğer sanayinin getirdiği gelişmelerle birlikte günümüzde son yıllarda yaygın hale gelen zemin iyileştirme yöntemlerinden biri de çeşitli zeminlerde çeşitli katkı maddeler kullanmasıdır. Genel anlamda zemin iyileştirmesi, zeminlerin sıkışabilirlik, kayma mukavemeti ve permabilite gibi mühendislik özelliklerinin daha elverişli duruma getirilmesi olarak tanımlanabilir. Burada elverişlilik ile anlatılmak istenen üzerine yapı yapılacak zeminin amaca uygun bir duruma getirilmesidir.

Zemin iyileştirmesinin yapılma amaçlarının arasında şunlar öncelikle sayılabilir; zeminin stabilizesini arttırmak, taşıma gücünü arttırmak, oturma potansiyelini ve dolayısıyla oturmaları azaltmak, yatay deformasyonları engellemek.

Günümüzde gelişen teknoloji ile beraber farklı prensiplere dayalı birçok zemin iyileştirme metodu geoteknik mühendislerince uygulanmaktadır. Son yıllarda önce laboratuvarda araştırılan daha sonra pratik mühendislik uygulamalarında kullanılan bir zemin iyileştirme metodu da doğal kaynaklardan veya suni olarak üretilmiş fiberlerin zeminle rastgele karıştırılarak homojen ve temiz zemine göre mühendislik özellikleri iyileşmiş zemin fiber karışımı elde etmektedir.

Temel olarak, bitki liflerin veya köklerin zeminin stabilizesine sağladıkları katkı göz önüne alınarak geliştirilmeye çalışılan bu teknikte, fiber-zemin karışımı fiberlerin zemine göre çok yüksek olan çekme mukavemetlerinin sonucu olarak fiber katkısız zemine göre çok büyük olan kayma mukavemeti değerlerine ulaşabilmektedir. Bu nedenle özellikle efektif gerilmelerin buna bağlı olarak kayma mukavemetinin düşük olduğu yüzeye yakın zeminlerde, fiberlerin verimli olacağı düşünülmektedir. Bununla birlikte fiberlerin zemin iyileştirmedeki avantajları söyle sıralanabilir.

Fiberlerin zemin ile karıştırılması stabilizasyon için kullanılan çimento ve kireç gibi diğer malzemelerin karıştırılması kadar kolaydır. Ayrıca, homojen karışımı sağlandığında, fiberler zemin içinde izotropik mukavemet sağlarlar. Diğer malzemeler ile maliyet açısından karşılaştırıldığında; diğer stabilizasyon malzemeler ile birim fiyatta yarışabilir hale gelmiştir. Ancak fiber ile iyileştirilirmiş bir zemin, çimento veya kireç ile

iyileştirilmiş bir zemine göre ortam koşullarından (Y.A.S.S gibi) çok daha az etkilenmektedir.
Fiber ile zemin iyileştirmesi için kullanılacak malzeme yelpazesi oldukça geniştir. Doğal fiberler (bitki kök ve lifleri) ile sentetik fiberler (polietilen, polipropilen) yanında geri dönüşümden elde edilen atıkların bir kısmı yine fiber olarak kullanılabilmektedir. Tüm bunların yanında fiberler mekanik olarak zeminde çekme gerilmelerinden oluşabilecek çatlakların göçme mekanizmalarını değişime uğratıp, zeminde ciddi mukavemet kaybını engellerler.
Bahsedildiği gibi zeminler çeşitli yüklere maruz kalabilmektedir. Bu yükleme durumları basitçe statik yükler ve dinamik yükler adı altında iki alt başlıkta sınıflandırılabilir. Statik yükler bazen üstyapıdan gelen yükler; bazen hidrostatik kuvvetler ve bazen de iksa ve istinat yapılarında olduğu gibi zeminin kendi ağırlığından dolayı oluşan yanal yükler olmaktadır. Diğer taraftan zeminler; deniz kenarına yakın bölgelerde dalga yükleri; fabrika veya büyük imalathanelerin temel altı zemindeki makine titreşim yükleri; patlamalar ve en önemlisi de deprem yükleri gibi dinamik etkilere maruz kalmaktadırlar. Depremler esnasında üstyapılarda zemin kaynaklı birçok hasar görülmektedir. Bu hasarlara neden olan en önemli olaylardan biri zeminlerin göçmesi ve sıvılaşmadır.
Zemin iyileştirmesine ve hatta bir yapının yapılmadan önce yapılan en önemli işlerden biri zeminlerin özelliklerini Arazi ve Laboratuvar deneyleri ile belirlemektir. Arazide bir yapı temeli veya toprak altında kalacak veya herhangi bir başka yüklemeye maruz kalacak zemin tabakalarının gerilme-şekil değiştirme davranışlarını ve kayma mukavemetlerini belirlemek için bu tabakalardan numune almak ve bunları laboratuvarda deneye tabi tutmak amacı ile birçok deneysel yöntem geliştirilmiştir. Bunlar arasında, daha yaygın olarak kullanılan yöntemler Kesme Kutusu deneyi, serbest Basınç Deneyi, Dinamik Üç Eksenli Deneyleridir.
Bu tez çalışmasında kum zeminlerin statik yükler altındaki davranışlarını katkısız ve de fiber katılarak Kesme Kutusu Deneyi ile incelenip ele alınmıştır. Kesme kutusu deneyinde, zemin numunesi dikdörtgen veya dairesel kesitli ve iki parçadan oluşan rijit bir kutu içinde yerleştirilmektedir. Uygulanan bir kesme kuvveti altında, kutunun bir parçası sabit tutulurken diğer parçası yatay bir düzlem üzerinde hareket edebilmekte ve böylece numunenin ortasından geçen yatay düzlem boyunca zemin kaymaya zorlanmaktadır. Numune üzerine normal bir gerilme uygulanarak ve böylece önce zeminin konsolide olması ve kesme sırasında normal gerilmelerin kontrol altında tutulması mümkün olmaktadır. Bu deneyde zemin önceden belirlenmiş (numunenin ortasından geçen) yatay bir düzlem boyunca kırılmaya (göçmeye) zorlamaktadır. Belirli bir normal gerilme altında, uygulanan kesme kuvveti ile meydana gelen yatay yer değiştirmeler ölçülmekte ve eğriler elde edilmektedir. Eğrilerin şekli zeminin cinsine ve başlangıç durumuna bağlıdır. Deney sırasında ulaşılan en büyük kayma gerilmesi veya göçme Kabul edilebilecek şekil değiştirmelere yol açan kayma gerilmesi zeminin belirli bir normal gerilme altında kayma mukavemetini vermektedir. Bu çalışmada deney değişik normal gerilmeler altında tekrarlanarak zeminin mukavemet zarfı elde edildi. Yani farklı normal gerilme değerleri için (σ_1, σ_2, σ_3) farklı kayma mukavemeti (τ_1, τ_2, τ_3) değerlerini değişik fiber oranlarındaki hazırlanmış olan numunelerde bulunarak zarfları elde edildi. Bu zarflardan fiber katkılı zemin numunelerine ait olan Mohr-Colomb kırılma zarfını sunmaktadır. Değişik normal gerilmeler altında elde edilen kayma mukavemetlerini bir eğride çizerek buradan kayma gerilmesi açısı (ϕ) elde edilmiştir. Kesme kutusu deneyleri, permabilite deneylerini kullanarak zeminin davranışı değişik fiber oranlarında ve değişik yükler altında incelenmiştir. Temiz kum ve fiberle güçlendirilmiş kum için Statik yükler altında gerilme-deformasyon davranışı ve kayma mukavemeti parametreleri belirlenmiştir. Deneylerde kullanılan fiber ve kum çeşidi sabit

tutularak çeşitli parametrelerin etkisi incelenmiştir. Ayrıca deneyleri iki değişik sıkılıklarda yani düşük (20%-37%) ve yüksek (60%-75%) rölatif sıkılıkta hazırlanıp test edilmiştir. Kesme kutusu deneyinde ilk olarak kum zeminin fiber katkısız olarak farklı statik yükler altına kayma mukavemeti parametrelerini elde etmek için test yapılmıştır. Daha sonra kuru kumun ağrılığının 0.1%, 0.5% ve 1.0% ağırlıklarında fiber karıştırılarak zemin aynı koşullarda test edilmiştir. Elde edilen bu sonuçlar karşılaştırılarak fiber oranının kayma mukavemetinde olan etkisi ve de yüksek ve düşük rölatif sıkılıkların kayma mukavemeti parametrelerindeki etkisi incelenmiştir.
Deneysel çalışmanın ikinci kısmında ise kesme kutusunda kullanılan kumların aynı özellikler ve fiber oranları ile permabilite aletini kullanarak permabilite katsayısı ve fiberin buna olan etkisi incelenmiştir. Bu deneyde aynı kesme kutusu deneyindeki gibi kullanılan fiber ve kum çeşidi sabit tutularak permabilite parametrelerin incelenmiştir. Numunelerin belli bir rölatif sıkılıkta (55%) hazırlanıp permabilite katsayısı elde edilmiştir. Bu deney kapsamında fiberlerin kum zeminlerde içine alacak su muhtevası ile ilgili de deney yapılmıştır. İki değişik rölatif sıkılıkta (60% ve 70%) numuneler kalıplara koyularak 0.01-0.03 MPa vakum ve 1-3bar CO_2 uygulanıp belli bir süre su geçirilerek fiberle donatılmış zeminlerin su absorbe potansiyeli ve numunelerin su hacmi incelenmiştir.
Genel olarak, yapılan laboratuvar deneyleri sonucu fiberle güçlendirilmiş kum zeminlerin statik yükler altında davranışlarının fiber katkısız zeminlere oranla iyileştiğini göstermektedir. Fiber miktarı arttıkça, kayma mukavemetinin artması gözlemlenmiştir. Ayrıca; gevşek ve sıkı koşullarda hazırlanmış olan numuneler üzerinde yapılan kesme kutusu deneyi sonucunda yüksek bir rölatif sıkılıkta hazırlanmış olan numunelerin daha yüksek bir kayma mukavemeti değerleri görülmüştür. Statik deneylerin sonuçlarına bakıldığında; fiber oranı arttıkça zeminin kayma mukavemeti artmıştır. 0.1% 'lik fiber muhtevasında bu değer çok önemli ölçüde değişmese de 0.5% ve 1.0%'lik numunelerde önemli ölçüde kayma mukavemetinin artışı gözlemlenmiştir.
Özet olarak Bu çalışmada kum zeminlerin statik yükler altındaki davranışlarını katkısız ve de fiber katılarak incelenip ele alınmıştır.
Kesme kutusu deneyleri, permabilite deneylerini kullanarak zeminin davranışı değişik fiber oranlarında ve değişik yükler altında incelenmiştir. Temiz kum ve fiberle güçlendirilmiş kum için statik yükler altında gerilme-deformasyon davranışı ve kayma mukavemeti parametreleri belirlenmiştir. Deneylerde kullanılan fiber ve kum çeşidi sabit tutularak çeşitli parametrelerin etkisi incelenmiştir.
Kesme kutusu deneyi sisteminde, temiz ve fiberle donatılı kumlar bu deney ile test edilmiştir. Bu deney çalışmasında değişik fiber oranları kullanarak kumun davranışları incelenmiştir. Fiber eklemenin kayma mukavemeti parametreleri üzerinde etkisi elde edilerek incelenmiştir bu sebeple ileride yapılacak olan çalışmalar ile uygulamada kolaylık ve kontrol edilebilirlik açısından, fiber koyma şekilleri, fiber oranı ve fiber boy/genişlik oranı değişken parametreler olabilir.

1. INTRODUCTION

Soils are subjected to different types of loading; that can be separated into two groups as static and dynamic loads. Static loads can refer to building load, weight of soil mass, hydrostatic loads and so on. On the other hand, the dynamic loads can be thought the result of wind, blasting, wave and mainly seismic and earthquake. There are several problems where the behavior of soils in static and dynamic loading are due to its complexity, uncertainly. In practice, for both static and dynamic soil problems, variety of soil improvement solution techniques are used to improve the engineering properties of soils; moreover solution of the problems, which are mentioned above, can be less expensive and much safer. The problem of static liquefaction of saturated sand is nowadays a classical soil mechanics subject. Castro (1969) found that sudden increases of pore water pressure, induced by monotonic shearing under undrained conditions, lead to the liquefaction of sand layers. Sand liquefaction can result in landslides, subsidence of foundations, and damage to earth structures, lateral movement of structures resting on soil, and disruption of services. It is thus important to consider the liquefaction potential of dams, embankments, slopes, foundation materials and placed fills (Krishnaswamy and Isaac, 1994). At present, the methods most commonly adopted to prevent liquefaction are densification, draining and soil reinforcement (Krishnaswamy and Isaac, 1994). Nevertheless, densification of deep deposits and draining is often ineffective and require suitable field equipment, so soil reinforcement has been considered recently (Vercueil et al., 1997; Li and Ding, 2002; Unnikrishnan et al., 2002; Boominathan and Hari, 2002; Diambra et al., 2010).

1.1 Purpose of Thesis

A wide range of reinforcements has been used to improve soil performance. Increasing the soil strength has caused increased interest in identifying new accessible resources for reinforcement. Short discrete fibers made of polymeric or natural material have been used to improve the shear strength of soil (Gray and Ohashi, 1983; Gray and

Alrefeai, 1986; Maher and Gray, 1990). Studies were performed recently using polymeric fibers (Nataraj and McManis, 1997; Santoni et al., 2001; Yetimoglu and Salbas, 2003; Heinecet al., 2005, Tang et al., 2007). It has been suggested that natural resources may provide superior materials for improving soil structure, based on their cost-effectiveness and environment friendly aspects (Prabakar and Sridhar, 2002). In scope of this thesis, statics behavior of fiber-reinforced sand is determined by performing laboratory tests. For the experimental study, random distribution of fibers is preferred as sample preparation only requires simply mixing fibers into sand and random distribution of fibers provide strength isotropy. (Yetimoglu and Salbas, 2002).
First, brief information about the most commonly used soil improvement techniques are presented. Secondly, static behavior of sand of sand is presented along with the previous studies on fiber-reinforced sand and samples. Considering the information obtained from literature study, the experimental program has been prepared.

In the experimental part of this study, direct shear test and immediate settlement and permeability tests are performed. The engineering properties of Akpinar sand are determined. One type of polypropylene fiber is chosen to mix into poorly graded Akpinar sand. The effect of fiber on the static behavior of sand is analyzed.

2. SOIL IMPROVEMENT

2.1 Introduction

Soil improvement techniques are used to improve the engineering properties of soils. These techniques vary by the application methods and soil types that can be improved. These methods are required not only when the top soil is not able to support structures but also when the deeper layers need to be improved. In general, the aim of soil improvement methods is to (Das, 2007):

1. Improve the shear strength of soils and increase the bearing capacity of shallow foundations.
2. Reduce the shrinkage and swelling of soils.
3. Reduce the settlement of structures.
4. Increase the factor of safety for possible slope failure of embankments and earth dams.

Tezcan and Özdemir (2004) explained the essential issues that have to be mentioned in the selection and execution of the improvement methods as follows:

- Applicability of the method
- Effectiveness of the methods
- The ability to verify the reliability of the mitigation achieved
- Overall cost of the implementations
- Environmentally and regulatory issues

Impe (1989) classified the soil improvement techniques in three categories according to aim of usage as follows:

1. Temporary soil improvement techniques: determined time to the period of construction
2. Permanent soil improvement techniques: these are applied to increase the engineering properties of natural soil with mechanical techniques
3. Permanent soil improvement techniques with the addition of materials

The vital issue in this classification is the soil layer type, actually, it is cohesive or not. Changing the parameters of soils, which satisfies the required strength, permeability and settlement condition in construction site, can be shown as soil stabilization. Another term, can be used for minor change in soil properties, is modification. In granular soils, reducing the void ratio and in cohesive soils, mixing soil with stabilizer and preloading to reduce settlement can be given as example for modification.

In practice, there are several soil improvement methods according to geotechnical problems and soil type. In this section of thesis, widely used soil improvement techniques, for remediation liquefaction, are mentioned; methods of practical application and their effects on soil properties are explained briefly.

2.2 Soil Improvement Methods

2.2.1 Mechanical stabilization

The aim of this method is to change the grain size distribution of the soil by adding binder materials that will fill the voids. In case of granular soils, the binder material adds cohesion to soil. Best results are obtained when the cohesive material occupies 75-90 % of the voids of granular materials. For cohesive soils, the granular binder material is mixed with soil (Bowles, 1997).

2.2.2 Compaction

Densification of soil is the most fundamental type of liquefaction-induced hazard remediation method. Its principle is to increase the resistance against liquefaction by decreasing the void ratio of the soils and changing stress state. Dense sand does not deform so easily as loose sand since dense sand does not develop high excess pore water pressure as loose sand does. Furthermore, densification methods can effects the original soil with compaction in different characteristics (JGS, 1998) such as compaction by penetration, compaction by vibration, compaction by impact energy. All these characteristics of compaction aim to reduce the initial void ratio of original soil. The benefit of densification methods can be shown with an example given in Figure 2.1, after Kobe earthquake (1995), red-colored areas were liquefied in Port Island, but in other fields, liquefaction did not exist because of application of soil densification method.

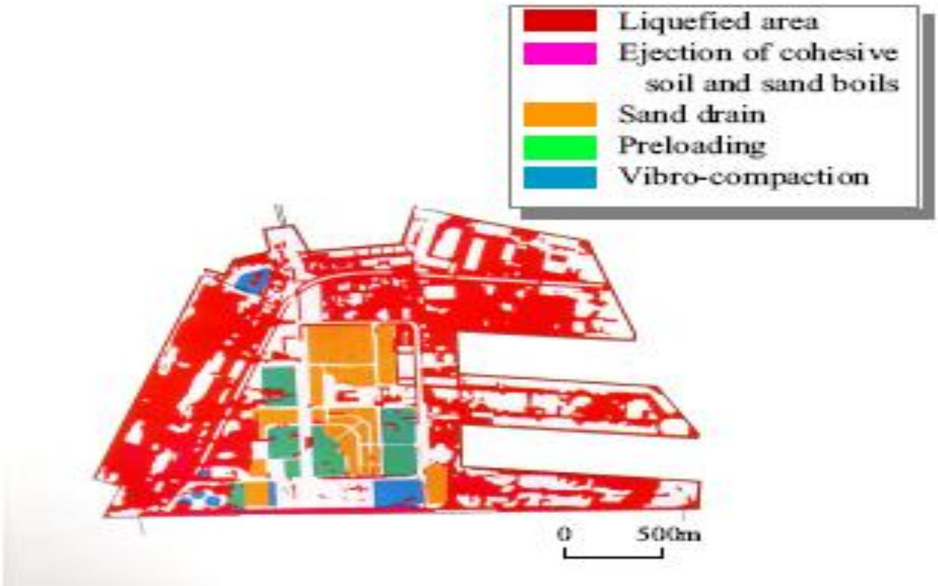

Figure 2.1 : Liquefied sites in Port Island (JGS, 1996).

In practice; sand compaction pile method, vibroflotation method, dynamic compaction and vibratory tamper method are applied to areas, which are susceptible to liquefy.

2.2.3 Sand compaction method

One of the most used methods in application of soil densification is installing of columns of dense sand to loose soil as Figure 2.2 illustrates. Initially, a soil, that is loose, is made denser by either pushing extra volume of sand columns into loose subsoil or relating vibration. As a result, the sand in initial condition is pushed outward in the lateral direction upon installation of sand columns. Matching with a suitably comprehensive previously treated confirmation program to check required mitigation has been achieved is essential point of this method. Sand compaction methods provides four following advantages in thick loose sandy layer (JGS, 2004);

1. Decrease in void ratio of original soil (Increase of relative density)
2. Raise in shear strength of sol and horizontal resistance by compaction
3. Change in the earth pressure condition by sand piles
4. Provide uniformity of sandy soil by compaction.

Hansbo (1993) defined the geometry and grids of installation of sand drains in construction side. Historically, at the beginning of the application, the drain diameter was large relatively, 0.4-0.6 m. Development of the technique brought smaller diameters.

For instance, in ‘sand wick’ application 0.05 m. in diameter and in ‘fabridrains’ application that the sand is packed into a synthetic fiber net-type tube, avoids narrowing of drain diameter.

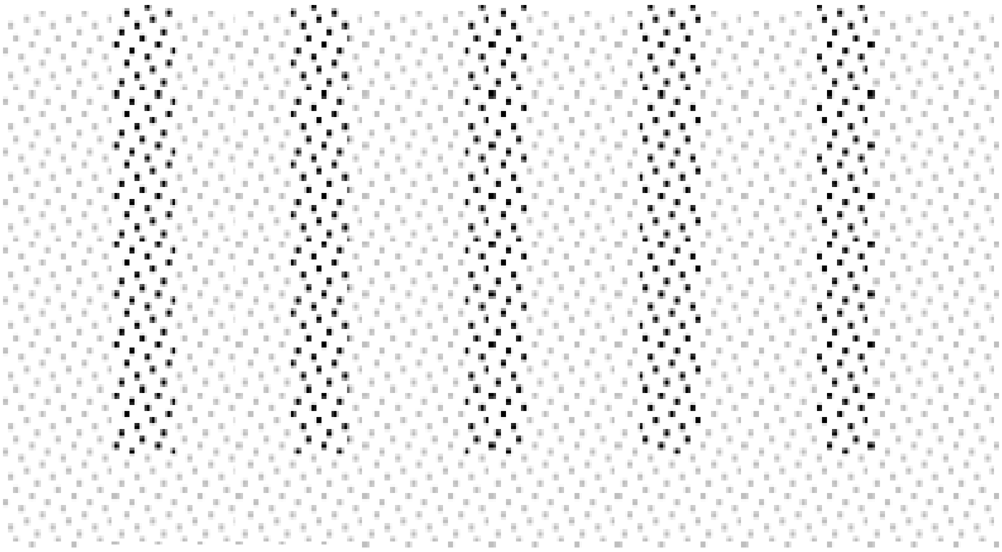

Figure 2.2 : Loose sandy ground compacted by sand columns (JGS, 1996).

2.2.4 Vibroflotation method

Vibroflotation method is suitable for compacting loose clean sand deposits that are above or below GWTL. It tends to reinforce the soil layer and keep from liquefaction by producing horizontal vibration and water compact effect. The apparatus of this method is a cylindrical probe that has an eccentric weight can rotate about the vertical axis and transfers horizontal vibration to the probe. In the Niigata earthquake of June 1964, particular there is no failure in structures on ground improved by vibroflotation method. Moreover, because of factors such as the permeability of soil, compaction time and energy, existence of cohesion influence improvement. Hence, the technique is commonly proper for coarse-grained sandy soils in practice. To summarize, vibroflotation is famous for its usefulness as a countermeasure against sand liquefaction that has been tested in practice in a full-scale field test (JGS, 2004). Properties of probe used during application of the method, can be given as, it has 15-40 kN weight, 30-50 cm. diameter and 2-5 m. length (Sandermann and Wehr, 1993).

Sandermann and Wehr (1993) explained a technique to put in the vibrator into the soil deposit easily; after vibrator is lifted, temporarily stable cylindrical cavity is filled with coarse material as gravel or block, and then this coarse material is compacted by repetitive use of the vibrator. Moreover, they said that the vibroflotation method is not applicable in nearly liquid state soil with low undrained cohesion because of not providing lateral support. According to them, with this method, soil in 25 m. can be improved successfully.

2.2.5 Dynamic compaction method

Dynamic compaction is also a kind of soil densification methods, which is performed by applying dynamic impacts and vibration to the surface of the soil layer by repetition of dropping of a heavy tamper that is lifted by mobile crane. JGS(2004) defined that free descending of a weight of approximately 10-55 ton forces from a height of 20-30 m. transmits an impact force in a wide range such as from hundreds ton forces to thousands ton forces to the soil layer, which is sent deep into the ground. This impact force is applied repeatedly 10-50 times in one position. Dropping of the heavy tamper into the ground surface repeatedly is named as 'tamping technique' in literature. This method can be used for compacting saturated soils that are classified as silty or clayey sand and/or gravels. The increment of fineness content causes the decrease of the improvement. While partially saturated clays above ground water table level can be improved by this method, there can be no improvement for fully saturated clays (Bowles, 1997). The schematically drawing of the dynamic compaction method is shown in Figure 2.3.

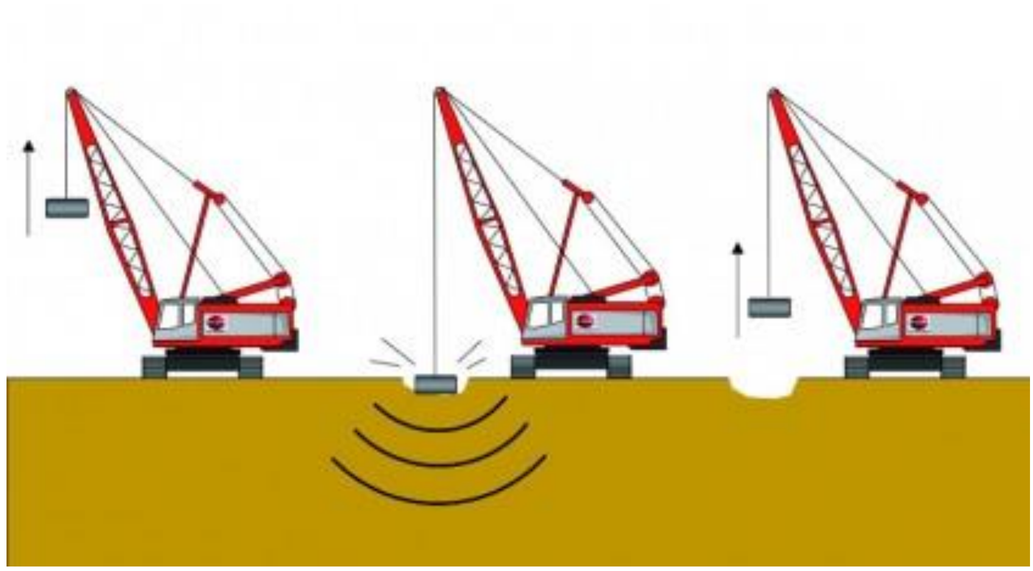

Figure 2.3 : Dynamic Compaction Method (Gunaratme, 2006).

2.2.6 Vibratory tamper method

This method is applied for the shallow soil layer, which is depth at 5-6 m from the ground surface. In practice, vibratory tamper method is applied to the surface layer together with the sand compaction to improve. The main issues in designing of this method are the number of applications and the time for compaction by the relationship between the increase of density and vibrating energy transmitted to the ground (JGS, 2004).

2.2.7 Soil replacement methods

The principle of soil replacement method is replacing soil with materials that are not susceptible to liquefaction. Day (2002) defined two kinds of soil replacement methods; (a) grazing and replacing (b) replacement. In practice, the first type is widely used. Comparing with other methods, soil replacement methods has difference that is changing the engineering properties of the original soil (e.g. permeability, density, void ratio) (Tezcan and Ozdemir, 2004). When this method is applied to the ground, after application, these three essential check have to be followed up; geotechnical investigations are related before the application, quality of water is tested before/during/after the application, check borings. The properties of material, which is replaced for increasing liquefaction of the original soil, is measured by experimental study in laboratory. For instance, if a gravel is used as the replacement material, then sieve analysis must be performed to obtain grain size distribution for evaluating liquefaction mitigation of it. After finishing of application of the method, check borings, grain size analyses and SPT are performed to estimate the improvement success.

Impe (1989) listed of the soil replacement method steps, respectively. It starts with excavation of original soil or dredging it, and then the soils, which are excavated and replaced, are transported and finally, the replaced material is squeezed by heavy weight. According to him, one of the most important point that has to be considered before the replacement is usage of light weight material soil, that means the replacing soil has, at least, same unit weight of the original soil, even preferably it should has smaller unit weight.

2.2.8 Lowering groundwater table method

Groundwater table is the surface of the groundwater. In geotechnical applications, groundwater affects the project and causes hazards. In similar way, most failures types in earthquake can be related with groundwater. Saturation, seepage pressures, uplift force and liquefaction causes loss of shear resistance of soil. Avoiding these hazardous effects of groundwater, lowering the groundwater table is not only beneficial but also only economic methods. Besides, Japanese Geotechnical Society (2004) listed the factors of improving effects of lowering groundwater table as following; the soil layer, which is risky to liquefy, will be located above the lowered GWTL so it will become unsaturated which means it has low risk to liquefy. On the other hand, the thickness of the liquefiable layer is a limitation to apply this method; furthermore, this method can change behavior of soils under seismic loads. In addition to this, there are essential investigations that have to be done to apply this method; JGS (2004) mentioned these investigations as follow;

1. Estimation of the risk of liquefaction of observed fields.
2. Evaluation of the decrease of susceptibility by lowering
3. Selection of dewatering methods
4. Comparison with other methods

Mainly, lowering groundwater table is applied by two ways: deep wells and drainage trenches.

2.2.8.1 Deep wells

The deep well process aims at stabilizing the soil by transferring the pore water in the sand layers through lowering the groundwater table. Deep well method is recognized as a cost effective and efficient way for medium to long term dewatering of larger projects where excavation is greater than 4 meters. If groundwater table is kept lowered than the settlement related with it and liquefaction risk are decreased or prevented. Deep wells are usually applied temporary works such as large-scale excavations or protection of the cutting face in tunnel excavation. Deep well method is illustrated in Figure 3.5. According to JGS (2004) the main points, that have to be considered, are ordered as following:

1- The number and diameter of deep wells
2- Selection of screens
3- Selection of filter materials
4- Selection of pumps.

2.2.8.2 Drainage trenches

Another lowering groundwater table method to reduce the possibility of liquefaction is drainage trench that method includes using culverts and channels, which will decrease the initial groundwater table to a depth such that no liquefaction exists. By this method, groundwater level is not only lowered during earthquakes but also permanent, so that damage to the buildings will be prevented. Besides, it can be used to control seepage in which case the top soil layer is thin and the pervious foundation is shallow so that the trench substantially perforates the aquifer.

2.2.8.3 Dissipation of excess pore water pressure

This method provides prevention of liquefaction by applying drains with different materials as gravel or other artificial soils in sand layer that has risk to liquefy, to dissipate PWP induced by earthquakes. Because of being, low noisy and low vibrating, it can be commonly preferred in urban areas.

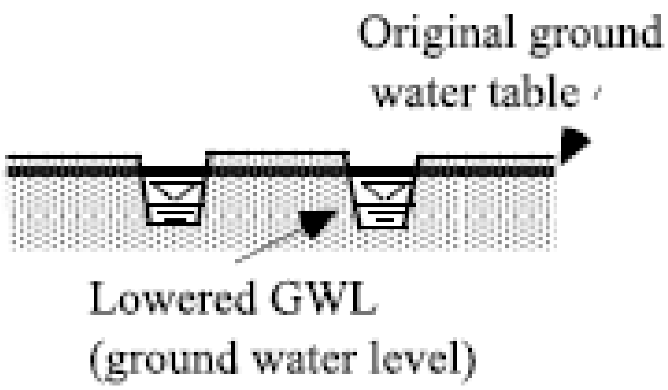

Figure 2.4 : Lowering of groundwater table by trench.

This method can be separated into two groups according to the material of drain; gravel and artificial drain. Briefly, columns of gravels are located in holes that are drilled in liquefiable soil. Since the gravel drain application does not induce lateral displacement of original soil, moreover compaction of initial soil and damage in foundation do not occur. The main purpose of gravel drain columns is the rapid dissipation of excess pore water pressure by shortening the path of drainage from vertical one toward the ground surface to a horizontal one to nearby gravel columns, hence avoiding the increment of excess PWP less than 100% during earthquake shaking. In 1993, Kushiro

Harbor with gravel drain columns deflected a large distortion; however, minor settlement approximately 10 cm. or so exist as shown in Figure 2.5.The artificial drain method can be defined as usage of long, slender pipes made of synthetic materials as drains.

Figure 2.5 : Subsidence of ground with gravel drains relative to stable quay wall (Kushiro Harbor, 1993).

2.2.9 Lowering groundwater table method

Mainly, soil solidification method based on adding chemical stabilizer to soil to increase the liquefaction resistance of soil. These methods can be considered applicable for all kind of soils, from fine-grained to coarse-grained, except for injection.

On the other hand, soil solidification methods are a chemical mixing treatment, Tezcan and Ozdemir (2004) emphasized essential points during application of soil solidification as follow:

1. Small development in strength is observed in organic soils
2. Homogenous solidification cannot be satisfied
3. Groundwater can be polluted because of the stabilizer.

4. The solidification methods can be categorized in two groups as follow; mixing, and grouting (Tezcan and Ozdemir, 2004). In this part of the thesis, these methods will be explained.

2.2.9.1 Mixing method

Mixing can also be subdivided into three groups such as deep, surface and premixing. Deep mixing can be considered grouting which is carried out by mixing soil with a cement-like material both by jetting or mechanical mixing. This method prevents liquefaction by stirring and mixing chemical stabilizer in the ground for solidification. For instance, deep jet mixing mixes cement powder with soil, and ground water is used to start to solidify but it is clear that this is not useful in dry soil. In contrast, cement deep mixing mixes cement slurry (water/cement ratio = 0.8/1.2). In similar way, this method may not be good in such soil of very high water content, because of softening effect of the additional water slurry. Figure 2.6 indicates an example of mechanical deep mixing.

Figure 2.6 : Deep mixing of liquefiable sand for reinforcement of river dike.

Surface mixing tends to prevent liquefaction in surface and shallow depths so that it should be used together with other improvement methods. If it is used as main improvement techniques then it has to be applied for light construction. In the premixing method, which is developed for land reclamation, a small amount of admixture for stabilizing is mixed with the earth fill before dumping it into the sea in order to improve the liquefaction facilities of reclaimed soil.

2.2.9.2 Grouting

Grouting consists of forcing a material under pressure, to fill joints and voids in rock, soil and similar materials. It can also change soil through the filling of voids or solidification into denser state. The main component of a grout process may be cementitious material, a liquid or solid chemical containing hot bitumen, or other one of the different resins (Warner, 2004). Generally, usage of two or more components is preferred; these grout materials can have nearly any consistency, ranging from a true fluid to a very stiff state.

The reason why usage of grouting becomes a widespread improvement technique is its ability of connection cracks, voids, and fissures, pore space that is generally unknown size, volume and configuration by filling. Moreover, predominantly, grouting provides strengthening or curbing the flow of water through soil deposits.

While deciding the suitable grouting materials and technique, it is exactly vital to perform a preliminary test injection and the information that gained from this procedure has to be compared with the result of geotechnical investigation, which is done before. Boring possibilities in the soil, the stratigraphy and non-homogeneities, permeability of the soil should be checked and evaluated (Impe, 1989)

Kutzner (1996) defined some phenomena that have to be considered in investigation of grouting techniques. These are:

1. The flow and hardening behavior of the grouting materials
2. The pressure for grouting
3. Effective radius
4. Grouting time
5. The construction site

Grouting has started and developed in practice first, not in theory. Apparently, fulfilling voids increase the engineering properties of grouted soil or make it stiffer and tighter than its initial state that means strengthen soil layers, either temporarily during construction or permanently for increased strength and load-bearing capacity. According to Warner (2004), solidification, cohesion increase, reinforcement and chemical stabilization are the mechanisms for achieve of increasing strength and bearing capacity.

Solidification of soil deposits can be provided by compaction grouting. This method injects the grouting material into the soil layer without mixing, but it makes a distinct interface in layer. In 1980, ASCE defined solidification as compaction grouting in following; ‘‘Compaction grout- grout injection with less than one inch (25 mm) slump. Normally a soil-cement with sufficient silt sizes to provide plasticity together with sufficient sizes to develop internal friction. This grouting technique is usually considered as improvement method in soils of new construction fields, especially for remediation of the liquefaction potential during strong ground motions such as earthquakes. The schematic drawn of compaction grouting is shown in Figure 2.7.

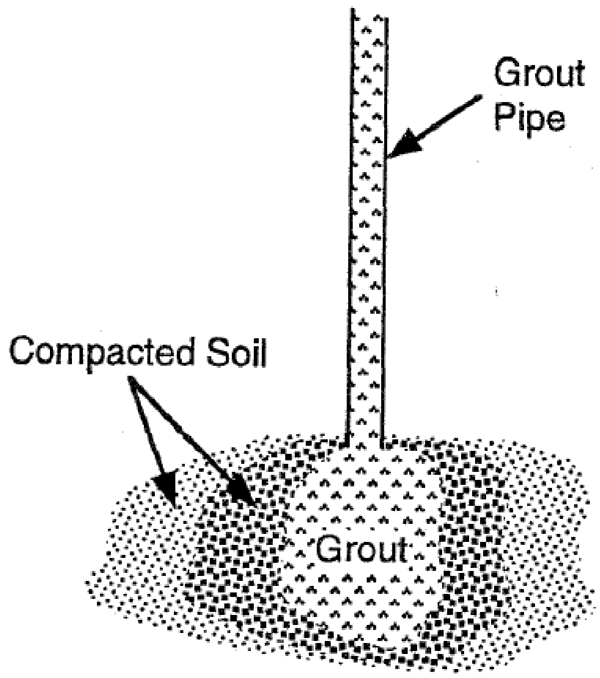

Figure 2.7 : Conceptual Drawing of Soil Densification by Compaction Grouting (Andrus and Chung, 1995).

The grout generally does not enter soil pores but remains in a homogenous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures, or both.’’ In practice, it is recommended that the grid of the holes should be designed 1.2-3.6 m. range which is depending on the required depth to improve (Warner, 2004). An example belongs to compaction grouting hole location plan view, which is in Pinopolis West Dam, is presented in Fig. 2.8.

Increasing of the soil cohesion with grouting by filling pores in soils with chemical or cement provides advantages to engineers in design process. In practice or theory, filling the pores and cracks in soil with an admixture is defined as permeation grouting. Permeation grouting does not only increase of the bounding ability of soil particles and cohesion but also makes soil gained unit weight. According to Warner (2004) there are two main purposes why permeation grouting is commonly used in practice, first one is strengthening of soil and second one is blocking of the flow water. It has been successfully applied to control ground water flow, stabilize excavations in soft soil deposits, underpin existing foundations and mitigate the hazard of earthquake-induced settlement and liquefaction. Figure 2.10 presents the conceptual diagram of permeation grouting.

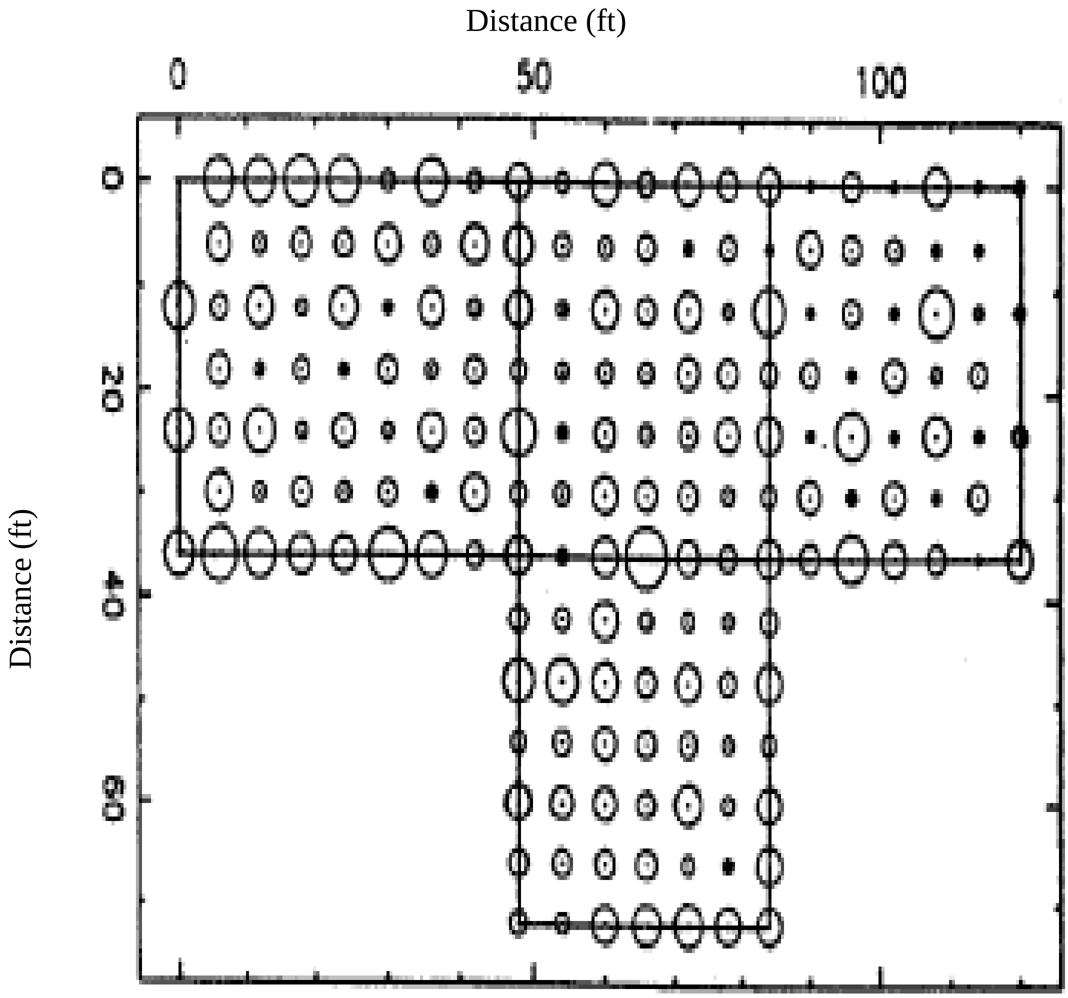

Figure 2.8 : Equivalent Scaled Compaction Grout column Diameter For Several Injection Location Pinopolis West Dam (Beaz and Henry, 1993).

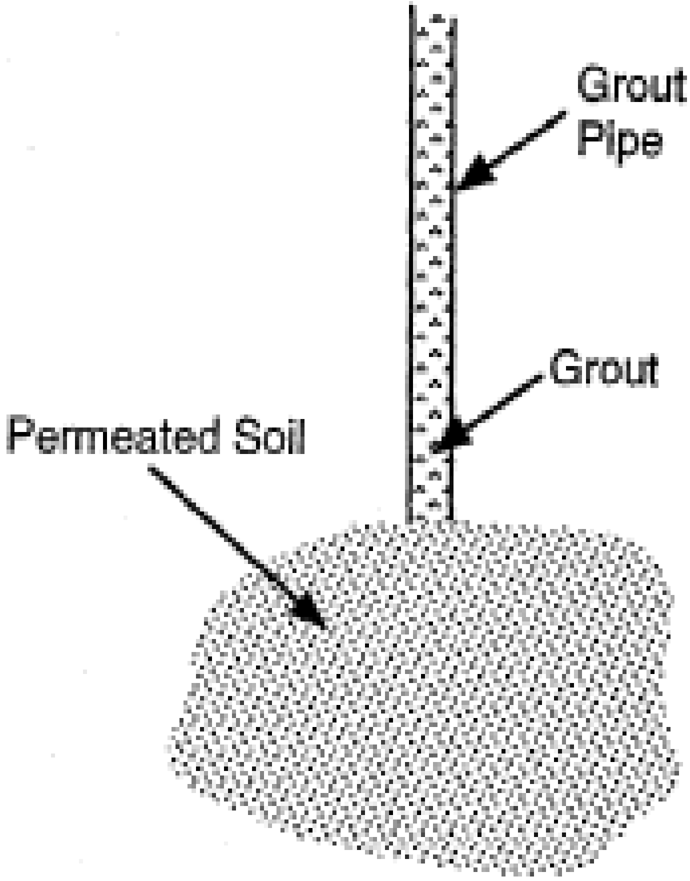

Figure 2.9 : Schematic diagram of permeation grouting (Andrus and Chung, 1995).

Andrus and Chung (1995) listed the factors that effect on success of permeation grouting as ; type of soil permeated, earth pressure, ground water conditions, grout mixture, grout injection pressure, rate and volume, grout hole spacing, injection sequence. The analyses and comparison of cost of permeation grouting is essential part of the grouting projects. According to Welsh (1991) the cost to mobilize and demobilize permeation grouting apparatus change within limits from \$15,000 and \$25,000 per carriage for projects using micro-cement grout, and over \$25,000 per carriage for projects using sodium silicate grout. The cost of injection push and grout materials start at approximately \$130 per cubic meter of improved soil for micro-fine cement grout, and \$200 per cubic meter of improved soil for sodium silicate grout. In Figure 2.10 presents the in-situ application of permeation grouting.

There are several case studies, where permeation-grouting method was applied to improve loose soils for mitigating liquefaction potential, can be given as example. In Table 2.1, some studies and its results are summarized.

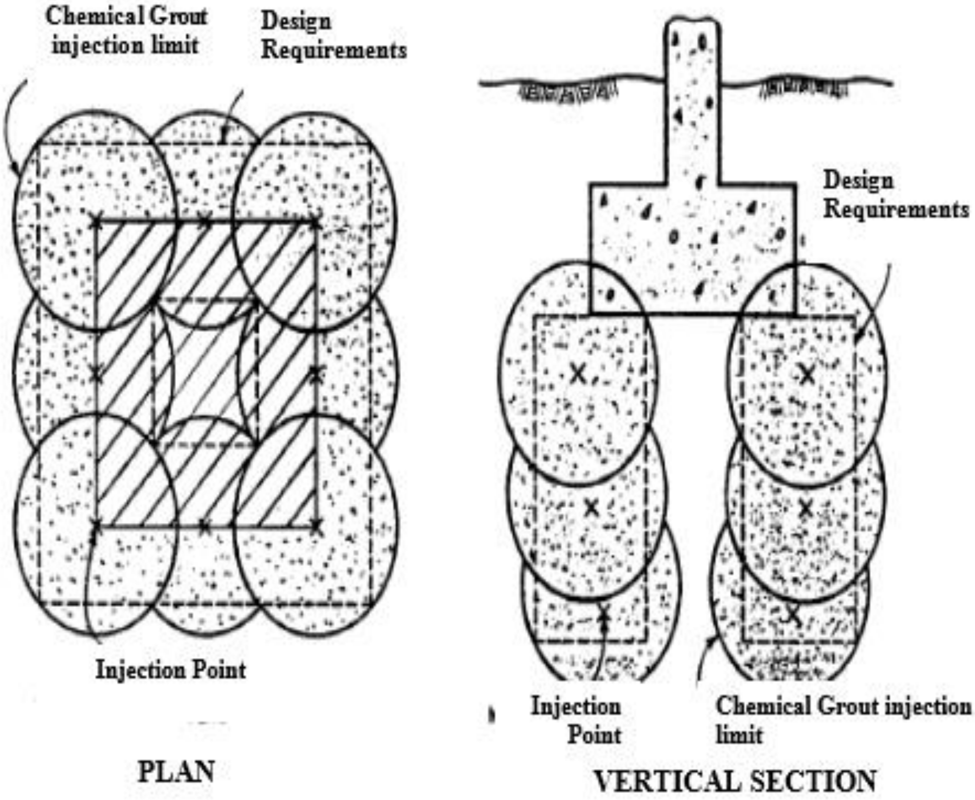

Figure 2.10 : Injection Patterns for Permeation Grouting Beneath Existing Spread Footings (Graf and Zacher, 1979).

Jet grouting is mostly used grouting method in the geotechnical soil improvement applications. It involves the mixing of in-situ soils with a cementitious suspension grout to produce new, relatively homogenous, mixed masses. In jet grouting, high-pressure fluid jets are used to erode, mix and replace soil with grout, respectively. This method have originally been applied in Japan, the UK and Italy (Kutzner, 1997). Although common idea is jet grouting can be applied to any kind of soil, it is more suitable in coarse-grained soils such as clean sand and gravels. In granular layers, degradation consequence is higher than in fine-grained soil so that grouting provides larger fields to be improved in this kind of soils. Furthermore, mixing grouting material and soil is more successful in coarse soil than in fine soils. In addition, energy consumption for jet grouting in clay is greater than granular soils (Warner, 2004).Another issue that has to be considered, is groundwater conditions. Sometimes seepage causes grout to be leached out, or chemical solutions in groundwater can be hazardous for grout columns. In Figure 2.11, the procedure for jet grouting is presented.

2.2.10 Fiber and biotechnical reinforcement

The principle of soil improvement with fiber can be define as a soil layer or mass which includes randomly distributed, discrete components, i.e. fibers, that supply an improvement in the engineering properties and mechanical behavior of the soil matrix. The behavior of soil that reinforced by fiber, is similar with a composite material in which fibers of relatively high tensile strength are planted in a soil mass (Hejazi et al., 2011). Mainly, usage of fiber in soil for reinforcement imitates the behavior of plant roots and helps to the stability of soil mass by adding strength to the soils at shallow depth where the effective stress is low (Wu et al 1988, Greenwood 2004).

Reinforcing the soil by biotechnical methods is also beneficial way. It involves the usage of live plants or trees to stabilize slopes against erosion and shallow mass movements (Gunaratme, 2006). Besides, bacteria and fungi are being used in soil stabilization especially at mitigating hazards of pollutant in soil and water (Karol, 2003).

Table 2.1 : Case Studies of Remediation for Seismic-Induced Settlement and Liquefaction by Permeation Grouting (Andrus and Chung, 1995).

Site	Site Characteristics	Reasons for Method Selection	Construction Program	Performance
Riverside Avenue bridge Santa Cruz, CA (Mitchell and Wentz, 1991)	Loose to medium dense gravity sand. River level at high tide 2.7 m above bottom of concrete slab-apron	Treatment beneath existing concrete noise pier and slab-apron; limited working space	Grout composed of sodium silicate N grade, MC 500 micro-fine cement, and less than 0.1 % by volume of phosphoric acid to control set time	No settlement or detrimental ground movement reported after 1989 Loma Prieta earthquake a_{max}= 0.45 g
Roosevelt Junior High School, San Francisco, CA (Graf and Zacher, 1979; Graf, 1992a)	Loose to medium dense silty sand and sand extending to depth of 4.6 m. N-values ranged from 3 to 15 before grouting	Existing structure and limited working space.	Sodium silicate based grout used. Stage down grouting in 0.3 m. intervals	Unconfined compressive strentgh ranged from 269 kPa to 879 kPa. No settlement reported after 1989 Loma Prieta earthquake; a_{max} = 0.15 g
Concrete structure remodeled into supermarket, San Francisco, CA (Graf,1992a)	Loose clean Sand	Existing Building	Sodium silicate based grout with an inorganic reactant for areas requiring low strentgh, and an organic reactant for areas requiring higher strength.	Unconfined compressive strength above the specified minimum. No settlement reported after 1989 Loma Prieta earthquake; a_{max}= 0.15 g

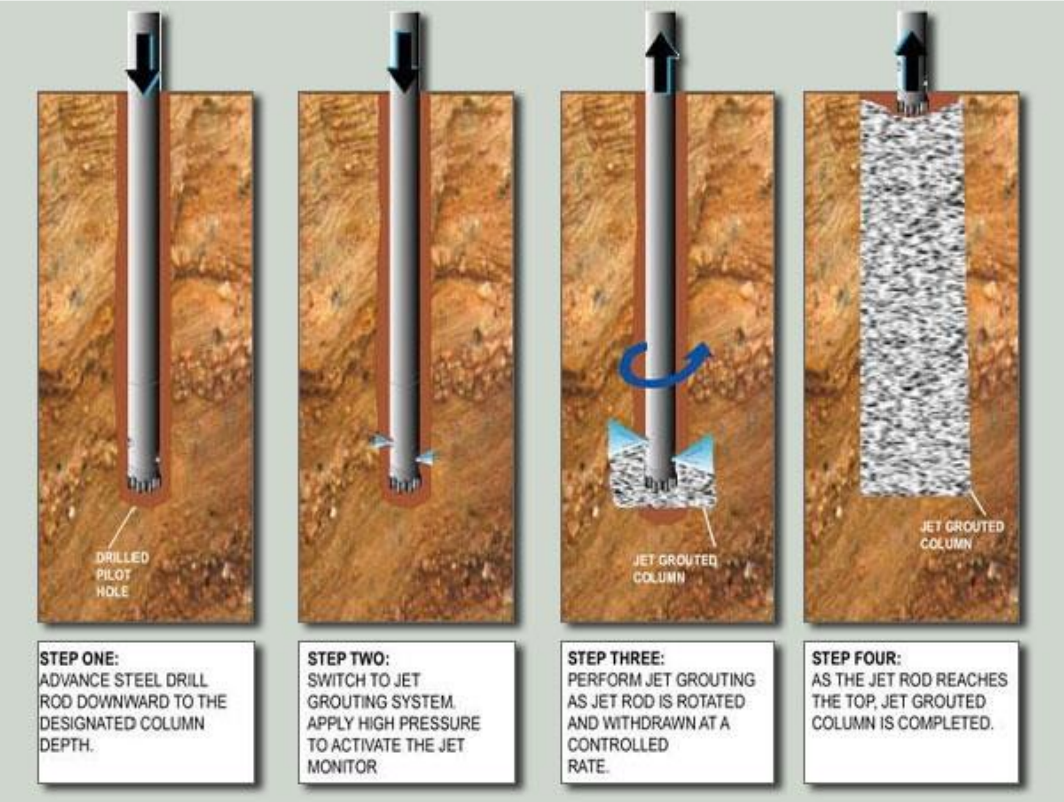

Figure 2.11 : Procedure for jet grouting (Ichihashi et al., 1992).

3. LITERATURE REVIEW

3.1 Introduction

In this part of the thesis, the static properties of sand are explained briefly. Furthermore, previous studies about randomly distributed fiber reinforced and polyurethane grouted sand are presented. The relationships between amount of fiber included, sample preparation methods, test methods are explained.

3.2 Static Behavior of Sand

Sand owns some complexities in its behavior because of its particulate nature and mode of deposition. Fine grained soil such as clay can be considered that all parts of the sediment have a unique initial point, due to its starting form of slurry and its current state effects from consolidation and swelling stages. However, this assumption is not sensible for sands because of its alluviation at different rates, effecting in a range of initial densities that determine resultant behavior. Pestana (1994) observed the compression behavior of two samples belong to Ticino sand; first specimen's initial void ratio equals to $e_0 = 0.6$, the looser one has void ratio with $e_0 = 0.8$. Both of them are compressed one dimensionally, and follow the normal compression line (NCL) that move towards a unique virgin compression lines (VCL). Figure 3.1 shows the behavior of sand, which has different void ratios mentioned above, under different effective stress. For loose Ticino sand, the VCL is accomplished only at $\sigma_v' = 10$ MPa, on the other hand, for the denser sample, higher vertical stress required because of more contact points compared to loose sand. As a result, the stress levels and initial void ratio influence the behavior of sand under static loads.

Under shear forces, its initial density also influences the sand behavior. Kuwano (1998) performed several triaxial undrained extension and compression tests on loose Ham River Sand and dense Dunkirk sand. In loose sand, for all OCRs the stress paths deflect to the left, furthermore this deflection causes positive pore pressure. If loading is continued, this compression tendency decreases and dilation occurs so negative pore water pressure builds up and stress path bended into the right. In dense sand, a similar contraction tendency occurs and dilatancy follows it.

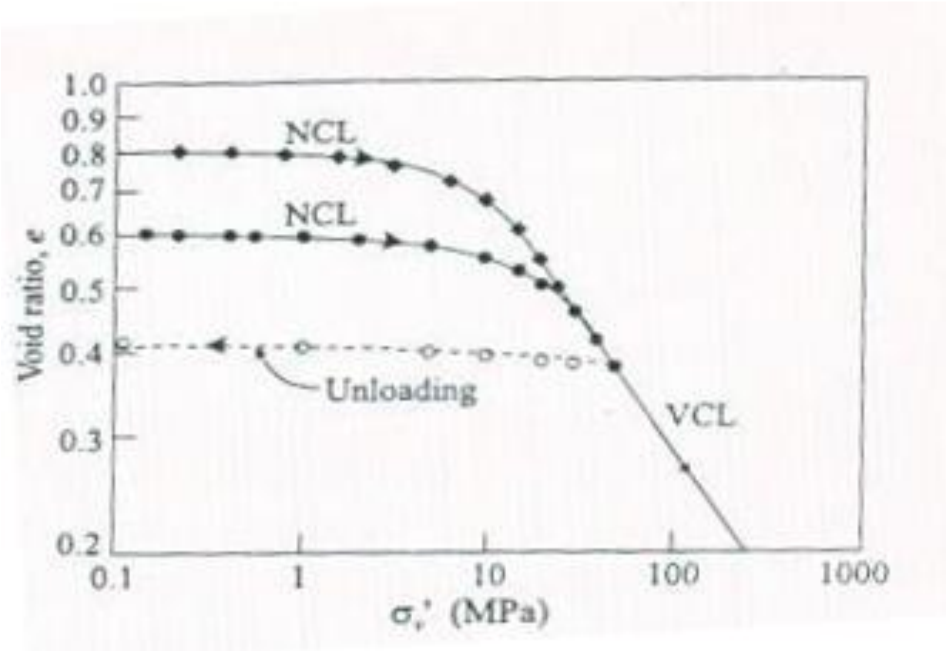

Figure 3.1 : One-dimensional behavior of Ticino Sand (Pestana, 1994).

To conclude, dense sands seek to dilate on shearing, whereas loose sands attempt to compress. Nevertheless, loose sands may turn back to a disposition to dilate when failure is approached. The tendency to dilate also depends on mean effective stress. At low stress levels, there is a greater tendency to dilate, whereas at very high stress levels even dense sands compress. Another issue, which has to be mentioned, is volume change in undrained condition. If a fully saturated soil is sheared undrained, the volume change is subdued. However, the volume change tendencies are still there and thus negative shear caused PWP are generated in dilatancy and positive shear induced PWP occurring the compressive phase. Ishıhara (1996) presents another study, which explain stress-strain relation of sand under static load.

In Figure 3.2 below, the stress-strain, relation of saturated sand specimens, which are sheared, undrained. In dense sand, the strain hardening behavior is observed and the shear stress raises up along with the shear strain. At this stage, dilation starts in dense sand. The strain softening behavior is observed in the specimen with a decrease of shear stress followed by large strains. This behavior is named as flow type.

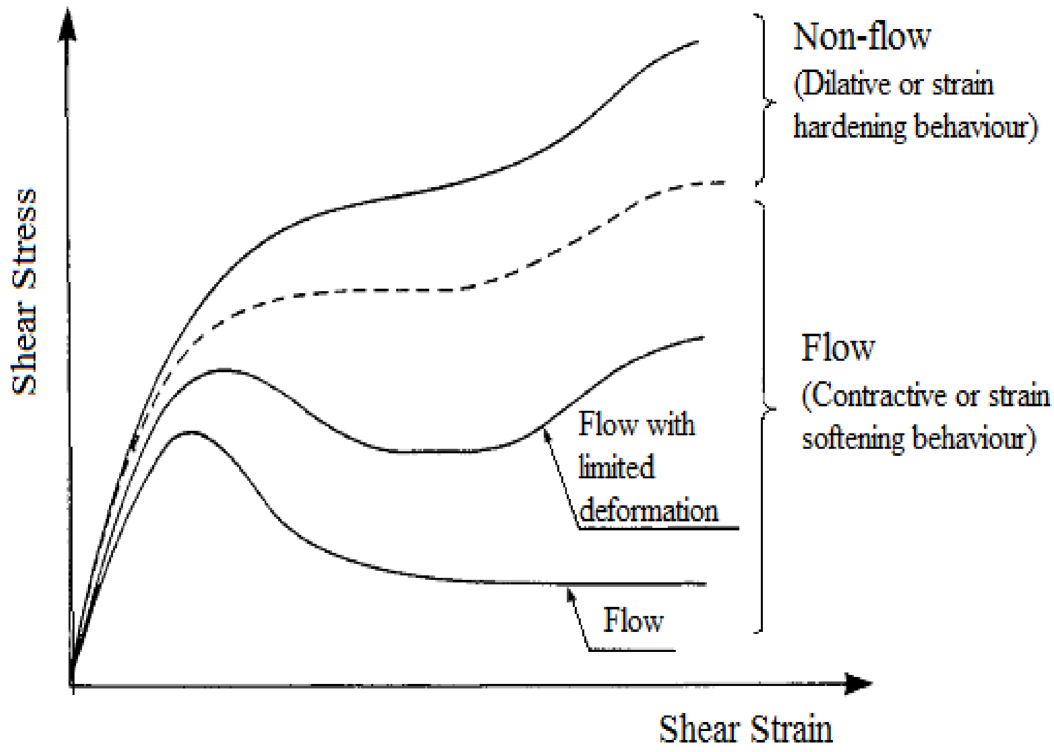

Figure 3.2 : Undrained behavior of sandy soils based on contractiveness and dilativeness (Ichihara 1996).

3.3 Settlements of Saturated Sands

Lee and Albaisa (1974) studied the settlement of sands caused by dissipation of excess pore pressure through stress-controlled cyclic triaxial tests. They reported that the amount of reconsolidation volumetric strain ($\varepsilon v = \Delta Vpl/Vpc$, where ΔVpl is post-loading volume change, and Vpc is post consolidation, pre-loading volume) for in-complete liquefaction conditions ($ru<1$) is a function of (1) grain size (2) relative den-sity, and (3) developed excess pore pressure generation. For a given soil, density, and confining pressure, the volumetric strain for non-liquefied saturated soils was depend-ent on the excess pore pressure buildup, as shown in Fig. 3.3. The volume change data formed a unique curve whether the loading disturbance was a few intense cycles or a large number of weak cycles. They concluded that for pore pressure ratios less than about $ru=0.6$, the representative average line (dashed line) as shown in Fig. 3.3 could approximate the relationship between volumetric strain and peak pore pressure ratio. They also concluded that the settlement was almost independent of how the excess pore pressure was generated (i.e. either under cyclic or static loading conditions). No conclusions were drawn about the possible effect of the induced strains on the post liquefaction settlement.

Table 3.1 : Summary of previous research on settlement of sands.

Testing Technique	Soil Tested	Nature Of Testing	Reference
Simple Shear	Dry sand	Strain controlled	Silver and Seed, 1971
Triaxial	Saturated sands	Stress controlled	Lee and Albaisa, 1974
Simple Shear	Saturated sands	Stress controlled	Tatsuoka et al., 1984
Previous Studies' Data Analysis	-	-	Tokimatsu et al., 1987
Simple Shear	Saturated sands	Strain controlled	Hsu and Vucetic, 2004
Biaxial Laminar Shear Box	Saturated sands	“Stress controlled	Ueng et al., 2009

In the same study, Lee and Albaisa (1974) also investigated the influence of confining pressure and grain size on the volumetric strain. The influence of confining pressure was found to be significant only for developed excess pore pressure ratios greater than r_u=0.6. In general, larger volumetric strains were observed with increasing confining pressures (ε_v=0.8% and 1.0% under 206kPa and 413kPa confinement, respectively, at r_u=0.9). Soil type and size of the grains were reported to have relatively significant importance on the amount of volumetric strains. Coarser grained soils led to larger volumetric strains compared to the finer grained sands at all excess pore pressures. Lee and Albaisa pointed out that grain shape may be a more fundamental characteristic than the grain size.

Continued application of cyclic stresses beyond the initial liquefaction results in progressively increasing transient axial strains (Seed and Lee, 1966; Lee and Albaisa, 1974). However, no quantitative data have been reported to this effect in the literature. Lee and Albaisa (1974) reported that their test results were widely scattered showing no obvious trend between volumetric strain beyond initial liquefaction and any of the variables they had investigated. They attributed the scatter in the data to the high distortions developed in the sample after initial liquefaction.

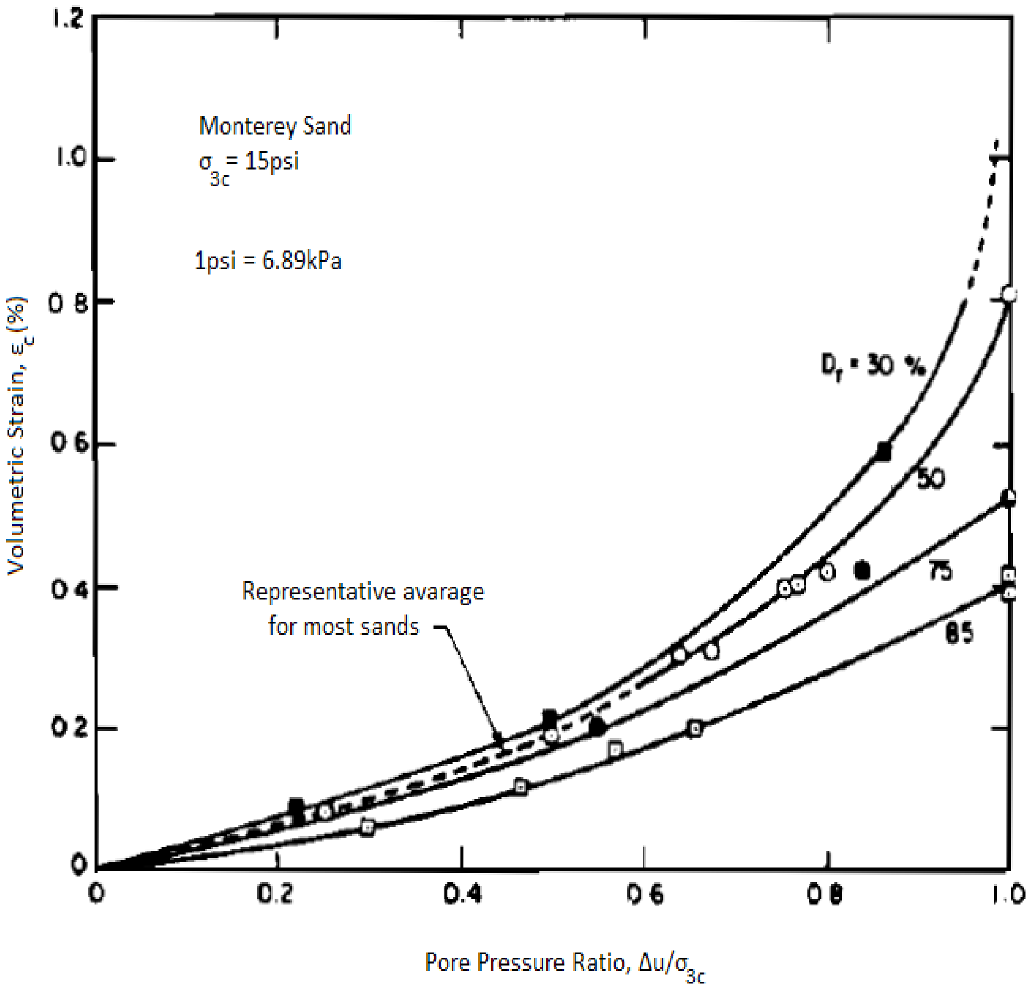

Figure 3.3 : Relationship between Volumetric Strain and Induced Pore Pressure Ratio (after Lee and Albaisa, 1994).

Tatsuoka et al. (1984) studied the influence of various parameters on volumetric strains after initial liquefaction (*ru*=1) through cyclic stress-controlled, undrained simple shear tests. They found that the amount of settlement significantly depends on the induced maximum shear strain and density of soil. The settlement was found to be relatively insensitive to the overburden pressure. Tokimatsu et al. (1987) compiled previous data (Tatsuoka et al., 1984; Lee and Albaisa, 1974) and reported that the maximum shear strain is an important factor influencing the settlement after liquefaction. It should be noted that these studies referred to the maximum shear strain that developed in the sample during cyclic stress-controlled testing, and it is unclear if the influence of the shear strains on settlement was due to the nature of stress-controlled testing (i.e., progressive increase in strain). A direct evaluation of the strain effect may be achieved through strain-controlled testing. Tokimatsu et al. (1987) presented correlations between relative density, maximum shear strain and the volumetric strain, as shown in Figure 3.4. The data show consistent trends despite the fact that different sands were used in each of the investigations. The volumetric strain decreases significantly with increasing relative density. It is also evident that larger induced shear strains result in larger volumetric strains at a constant relative density.

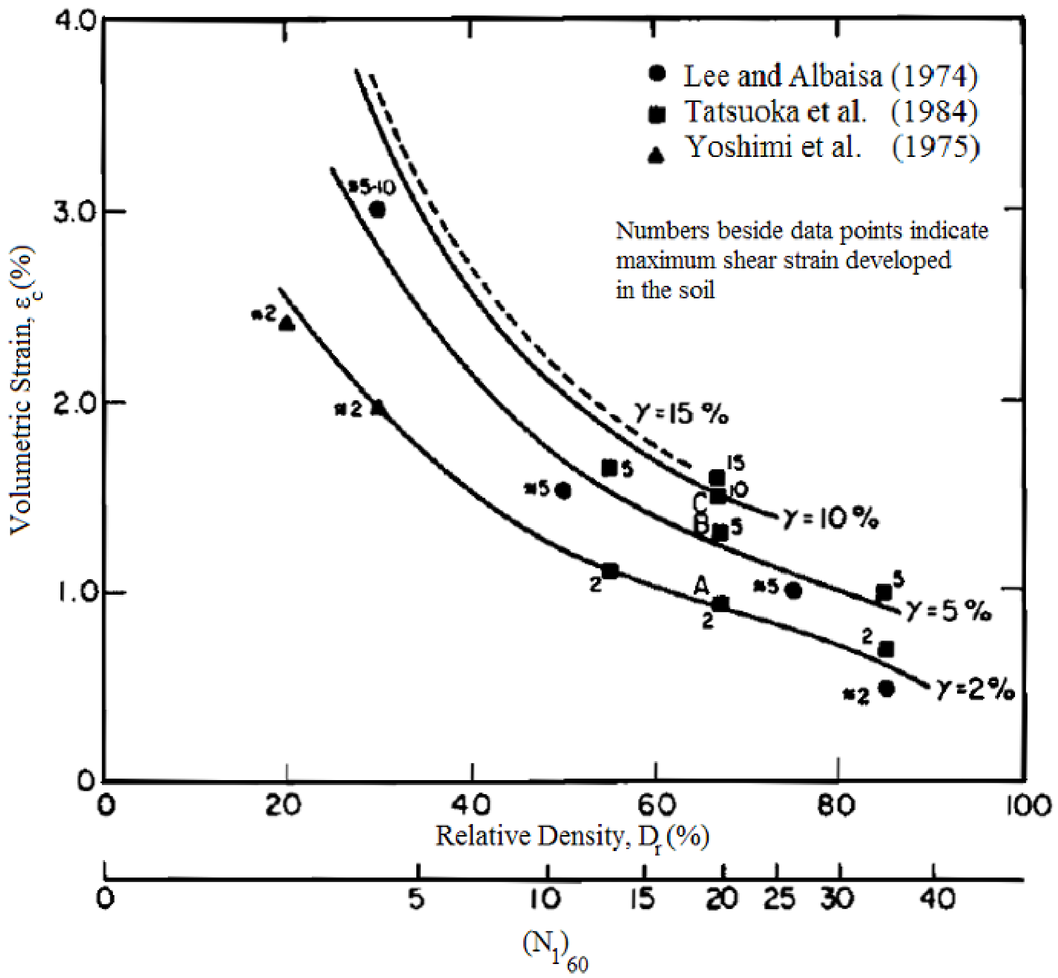

Figure 3.4 : Relationship between volumetric strain, induced strain and relative desity for sands (after Tokimatsu et al. 1987).

As discussed before, liquefaction resistance in stress-controlled approach depends on factors such as method of sample preparation, and the stress history.

Tokimatsu et al. (1987) reported that these factors were less significant when dealing with post-liquefaction volumetric strain. He attributed this to the relatively large strains developed during liquefaction and suggested that the settlement behavior of sands is primarily controlled by the relative density and the induced maximum shear strain. More recently, Ueng et al. (2009) conducted a study on settlement of a saturated clean sand using a large biaxial laminar shear box. Various one and multi-directional sinusoidal input motions were imposed by a shaking table at different frequencies and accelerations. Loading accelerations, varying from 0.03g to 0.15g, and durations, from 5s to 30s, caused both liquefied and non-liquefied results. It was found that the settlement of a sand deposit without liquefaction during shaking was generally very small ($\varepsilon_v \approx 0.1\%$).

Significant volume changes (ε_v up to 8%) occurred only when there was liquefaction of sand. They reported that post-liquefaction volumetric strain of the sand decreased with increasing relative density regardless of shaking amplitude, frequency and direction (1-D or 2-D shaking), but increased with shaking duration (i.e. number of loading cycles).

Moreover, their results indicated that surcharge mass (applied to simulate about 1m thick overlying soil layer) did not significantly affect the settlement characteristics for sand without liquefaction, while surcharge slightly increased volumetric strains for liquefied specimens.

Most of the previous studies on saturated sand settlements were performed through stress controlled testing. The amount of settlement was correlated to shear strains that developed at the liquefaction cycle (depending on the liquefaction criterion. i.e. either r_u approached to 1.0 or double amplitude axial strain of about 5%). Saturated sand settlement has not been thoroughly investigated by strain-controlled testing. Few studies (Seed and Silver, 1971; Youd, 2001; Vucetic 1994; Hsu and Vucetic, 2004) employed strain controlled simple shear tests in soil settlement tests, but they also either referred to the settlement behavior of cohesive materials (Vucetic 1994, Hsu and Vucetic, 2004) or concentrated on different aspects (such as dry soils, threshold values of shear strains and other aspects) of the soil settlement and no direct evaluation of the strain effect on postcyclic loading settlement has been done.

Silver and Seed (1972), and Youd (1972) studied the behavior of dry uniform silica sand under seismic loading conditions using Norwegian Geotechnical Institute (NGI) type simple shear apparatus. Silver and Seed showed that the shear strain, rather than shearing stress, controls the rearrangement of soil particles and consequently the settlement of sand deposits. They found that the important parameters influencing the settlement were: (1) relative density, (2) magnitude of the cyclic shear strain, and (3) number of strain cycles.

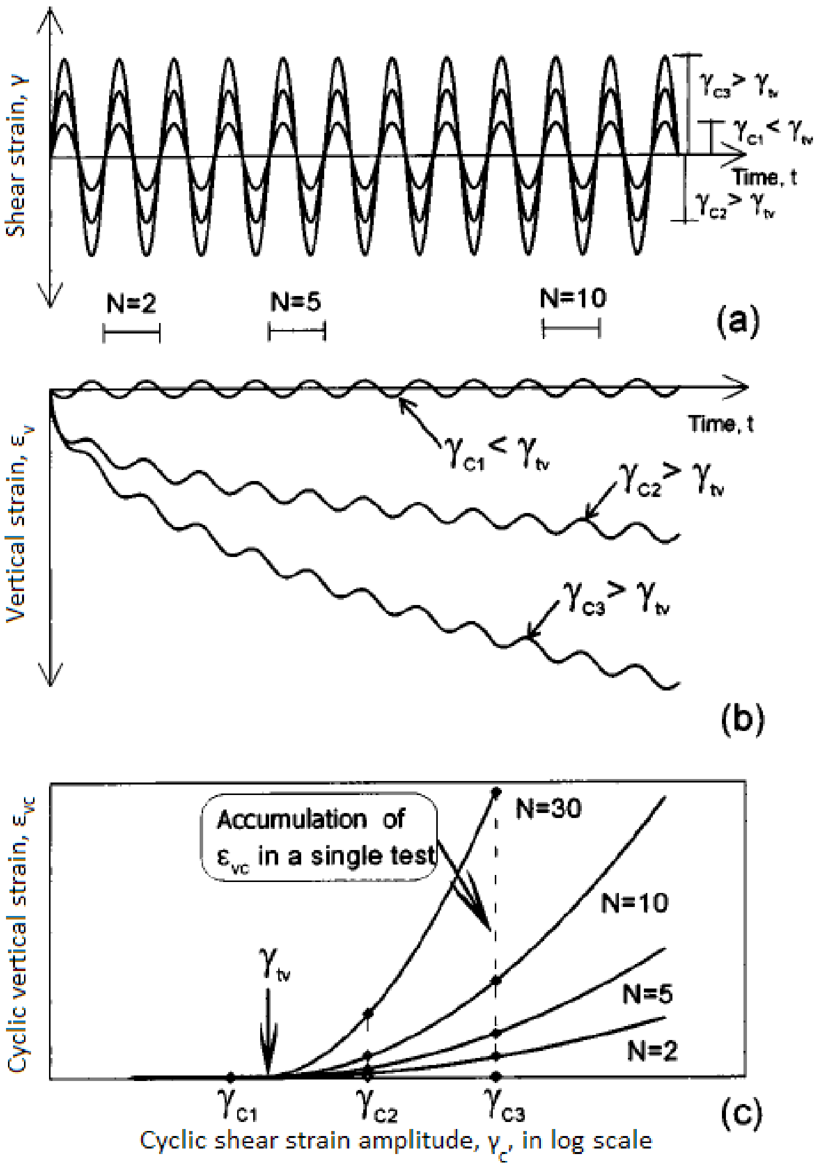

Figure 3.5 : Sketch of typical results of cyclic simple shear strain-controlled tests with definitions of volumetric cyclic threshold strain: (a) strain time histories of three cyclic strain controlled settlement tests; (b) variation of vertical strain, εv, over time (c) relationship among cyclic vertical strain, εvc, cyclic shear strain amplitude, γc, and number of cycles, N (after Hsu and Vucetic, 2004).

They also reported that the settlement amount was relatively insensitive to the vertical stress. Youd also confirmed that shear strain was the primary factor causing compaction for dry granular materials. He showed how shear strains could be used to predict in situ density changes due to static, vibratory or seismic sources.

Hsu and Vucetic (2004) evaluated a threshold value for shear strain, γ_{tv}, below which little to no volumetric strain occurs using the results of direct simple shear multistage strain controlled tests. Specimens subjected to cyclic straining smaller than threshold shear strain value, $\gamma_{cyc}<\gamma_{tv}$, experienced no permanent volumetric strain, while those tested at shear strains higher than the threshold value settled appreciably. Such behavior is depicted in Fig. 3.5. In their study, Hsu and Vucetic reported that γ_{tv} is larger for clays than for sands and that it generally increases with the soils plasticity index. $\gamma_{tv}\approx$ 0.01-0.02% was obtained for sands, and $\gamma_{tv}\approx$ 0.012-0.022% (PI=9.5), $\gamma_{tv}\approx$ 0.020-0.033% (PI=23), and $\gamma_{tv}\approx$ 0.04-0.09% (PI=30) were obtained for clays.

4. METHODOLOGY

The shear strength of soils is an important aspect in many foundation engineering problems such as the bearing capacity of shallow foundations and piles, the stability of the slopes of dams and embankments, and lateral earth pressure on retaining walls.
In this chapter, the shear strength characteristics of granular and cohesive soils and the factors that control them will be discussed (Das, 1983).
In 1910, Mohr presented a theory for rupture in materials. According to this theory, failure along a plane in a material occurs by a critical combination of normal and shear stresses, and not by normal or shear stress alone. The functional relation between normal and shear stress on the failure plane can be given by

$$\tau_f = f(\sigma_f) \qquad \textbf{(4.1)}$$

Where τ_f is the shear stress at failure and σ_f is the normal stress on the failure plane. The failure envelope defined by Eq. 4.1 is a curved line, as shown in Figure 4.1. In 1776, Coulomb defined the function $f(\sigma)$ as

$$\tau_f = c + \sigma_f \cdot tg\Phi \qquad \textbf{(4.2)}$$

Where c is cohesion and Φ is the angle of friction of the soil. Eq. 4.2 is generally referred to as the Mohr-Coulomb failure criteria. The significance of the failure envelope can be explained using Figure 4.1. If the normal and shear stresses on a plane in a soil mass are such that they plot as point A, shear failure will not occur along that plane. Shear failure along a plane will occur if the stresses plot as point B, which falls on the failure envelope. A state of stress plotting as point C cannot exist, since this falls above the failure envelope; shear failure would have occurred before this condition was reached (Das, 2005). In geotechnical engineering, compressive stresses in soils are assumed positive. Soils cannot sustain any appreciable tensile stresses, and we normally assume that the tensile strength of soils is negligible. Strains can be compressive or tensile.

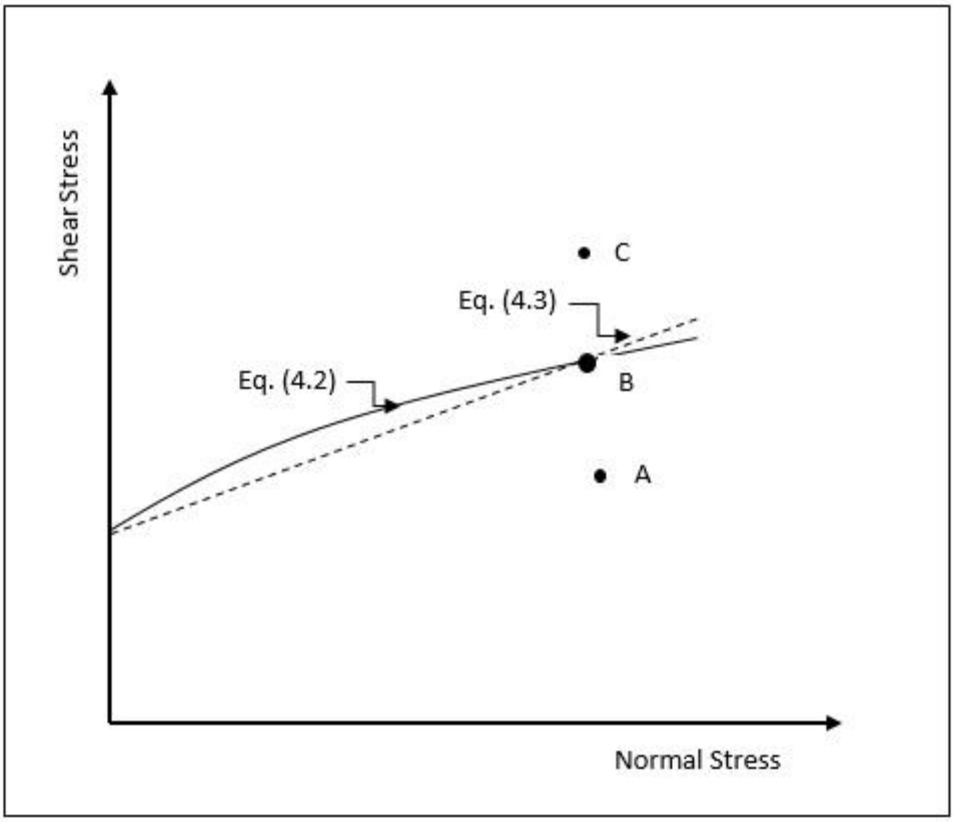

Figure 4.1 : Mohr-Coulomb failure criteria (Das, 1983).

$$\tau_f = c' + \sigma_f' \cdot tg\Phi' \quad \textbf{(4.3)}$$

Where,

τ_f : shear stress at failure plane

c: cohesion intercept in terms of total stresses

c': cohesion intercept in terms of effective stresses

σ_f: total normal stress at failure plane

σ' effective normal stress at failure plane

ϕ: friction angle in terms of total stresses

ϕ': friction angle in terms of effective stresses

Equation (4.2) is applicable for σ_f defined as total normal stress and c and $\phi$are termed as total stress parameters. Equation (4.3) applies for $\sigma_f{}'$ defined as effective normal stress and $c'$and $\varphi'$are effective stress parameters. Effective stress affecting the frictional resistance between soil particles is accepted as the basic factor influencing strength. Effective parameters are generally used under conditions where effects of the drained conditions on the shear strength are more critical than that of undrained conditions such as the stability problems of the slopes. However, total parameters are generally taken into consideration for undrained conditions if they are more critical in shear strength problems such as the bearing capacity problems of shallow foundations (Head, 1982). In fact, shear strength of a soil depends on many factors, which:

- stress history,
- soil composition,
- water content,
- degree of saturation,
- soil structure,
- void ratio,
- drainage conditions,
- isotropic media in the soil,
- rate of loading.

All of the factors may be effective on the shear strength and values of c and ϕ can be depended on these factors. Consequently, a variety of types of friction angle and cohesion parameters can be defined for total and effective stresses.

4.1 Shear Strength of Cohesionless Soils

The shear strength of a cohesionless soil may be represented by Eq.4.4. This is a special case of Eq. 4.3, where c = zero.

$$\tau_f = \sigma_f \cdot tg\Phi \tag{4.4}$$

Generally, the value of ϕ is influenced by,

- The state of compaction and the void ratio of the soil. The friction angle increases with decreasing void ratio (increasing density), but not linearly.
- Coarseness, shape and angularity of the grains. Angular grains interlock more effectively than rounded ones, thereby creating a larger friction angle.
- Mineralogical content. Hard gravel particles result in higher friction angles than soft grains, which may crush more easily, thereby reducing the interlocking or bridging effects. For sand, however, the mineralogical content seems to make little difference except if the sand contains mica. In that case, the void

ratio is usually larger, thereby resulting in loose interlocking sand lower friction angle.

- Grain size distribution. A dense, well-graded sand usually displays a higher friction angle than a dense uniform-sized sand (Cernica, 1995).

The characteristics of dry and saturated sands are the same provided there is zero excess pore water pressure in the case of saturated sands. However, the shear strength might be altered significantly by a change in the pore pressures. Hence, Eq. 4.5 might give the shear strength of a saturated cohesionless soil.

$$\tau_f = (\sigma - u) \cdot \tan \Phi \qquad \textbf{(4.5)}$$

Whereσ is effective (intergranular) stress and u is the pore-water pressure. When the pore-water pressure approaches σ, the shear strength approaches zero. When that happens, we may approach impending instability or perhaps motion (e.g., slope failures, boiling). Fluctuation in the water table is a common cause of significant variations in the pore stress and, thereby, in the shear strength of the soil.
One of two methods; the direct shear test or the triaxial test commonly accomplishes the determination of the friction angle Φ. A brief theoretical background for direct shear test is explained in the following.

4.2 Direct Shear Test

Direct shear test is used to determine the shear strength of soils on predetermined failure surfaces. The principle of the direct shear test is illustrated in Figure 4.2. The soil sample confined inside the upper and lower rigid boxes, is subjected to the normal load N and is sheared by the shear force T. If A is the area of surface CD, the shear stress τ acting on surface CD is equal to T/A, and the normal stress σ is equal to N/A.

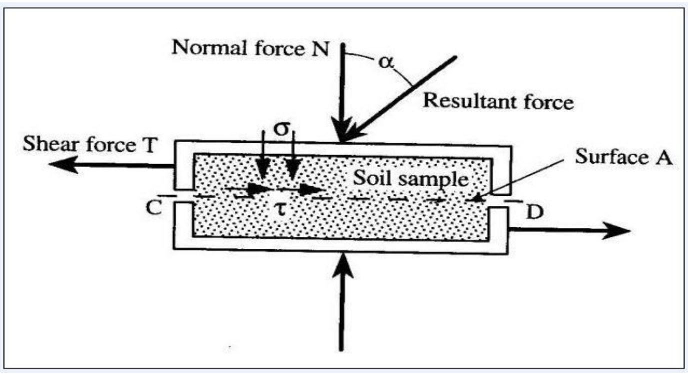

Figure 4.2 : Figure captions must be ended with a full stop.

The soil shear strength is the shear stress τthat causes the soil to slip on surface CD. It can be defined by Mohr-Coulomb theory (in Eq. 4.2).

The nature of the results of typical direct shear tests in loose, medium, and dense sands are shown in Figure 4.3. Based on Figure 4.3, the following observations can be made: In dense and medium sands, shear stress increases with shear displacement to a maximum or peak value τ_m and then decreases to an approximately constant value τ_{cv} at large shear displacements. This constant stress τ_{cv} is the ultimate shear stress.

For loose sands, the shear stress increases with shear displacement to a maximum value and then remains constant.

- For dense and medium sands, the volume of the specimen initially decreases and then increases with the shear displacement. At large values of shear displacement, the volume of the specimen remains approximately constant.
- For loose sands, the volume of the specimen gradually decreases to a certain value and remains approximately constant thereafter.

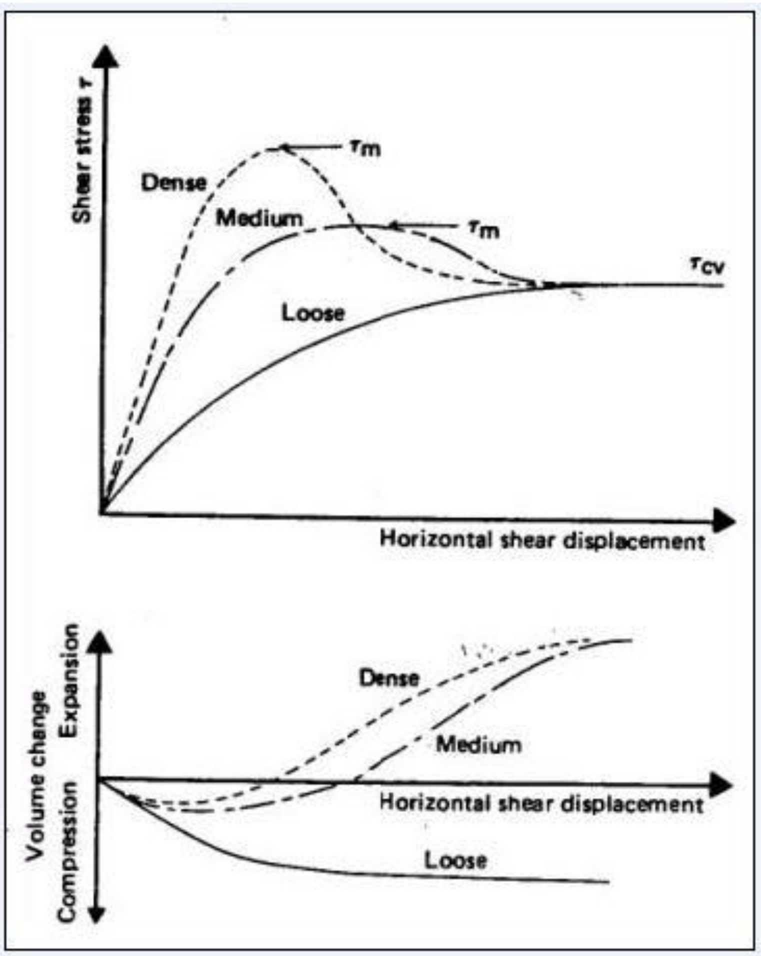

Figure 4.3 : Direct shear test result in loose, medium, and dense sands (Das, 1983).

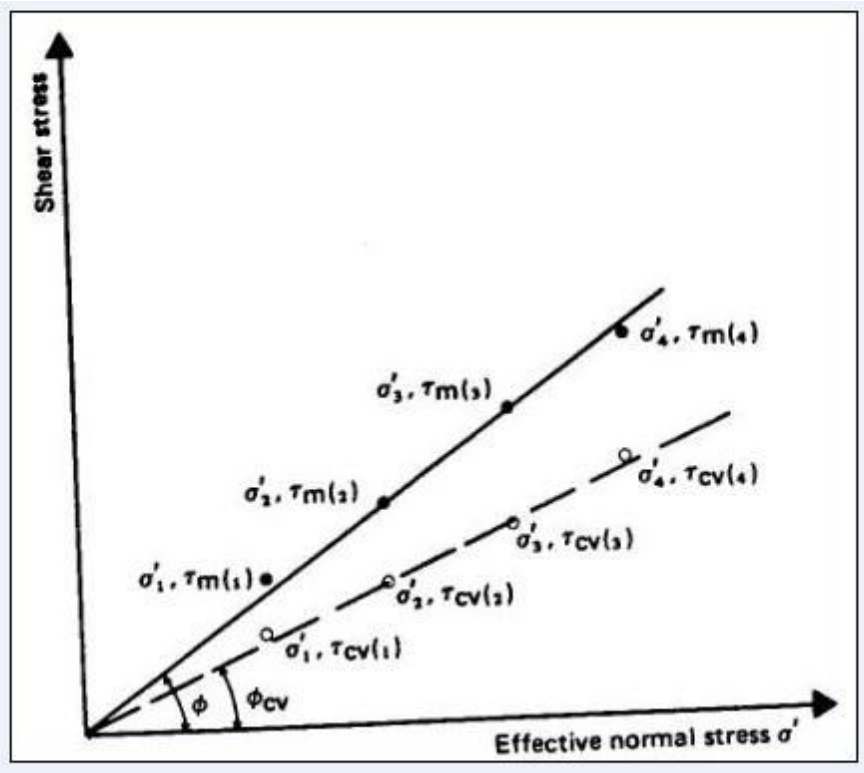

Figure 4.4 : Determination of peak and ultimate friction angles from the direct shear test (Das, 1983).

The angle of friction ϕ for the sand can be determined by plotting a graph of the maximum or peak shear stresses vs. the corresponding normal stresses, as shown in Figure 4.4. The Mohr-Coulomb failure envelope can be determined by drawing a straight line through the origin and the points representing the experimental results. The slope of this line will give the peak friction angle ϕ of the soil.

Similarly, the ultimate friction angle ϕ_{cv} can be determined by plotting the ultimate shear stresses τ_{cv} vs. the corresponding normal stresses, as shown in Figure 2.4. The ultimate friction angle ϕ_{cv} represents a condition of shearing at constant volume of the specimen. For loose sands, the peak friction angle is approximately equal to the ultimate friction angle.

4.3 Calculation

Shear stresses on the horizontal surface are computed for every gauge reading intervals as follows:

$$\tau = \frac{C_p \cdot \delta_p}{A_c} \quad \textbf{(4.6)}$$

where, τ is shear stress, A_c is the corrected area of the sheared specimen,C_p is proving ring constant, δ is proving ring deflection. The normal stress σ (in kN / m^2) on the horizontal surface is calculated from:

$$\sigma = \frac{N}{A_c} \quad \textbf{(4.7)}$$

Boxes in square and cylindrical shape are practicable in practice. The corrected area $_c$ A of the sheared specimen is for the square box of length a (in m.),

$$A_c = a.(a - \delta) \quad \textbf{(4.8)}$$

And for the circular box of internal diameter D (in m.),

$$A_c = \frac{D^2}{2} \cdot (\theta - \frac{\delta}{D} \cdot Sin\theta) \quad \textbf{(4.9)}$$

Where,

$$\theta = \arccos(\frac{\delta}{D}) \quad \textbf{(4.10)}$$

The contact area between the two specimen halves varies with the relative shear Dis-placement δ between the lower and upper boxes are shown in Figure 4.5. (Bardet, 1997).

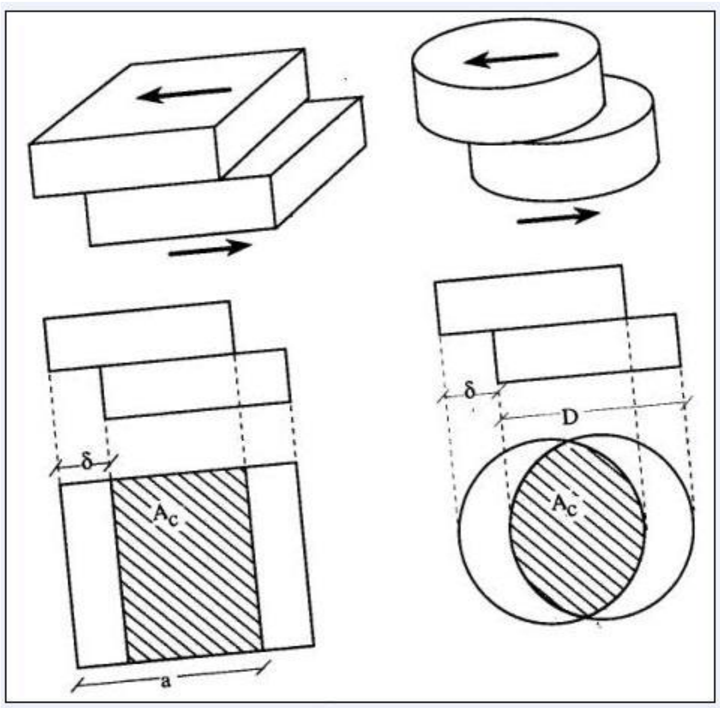

Figure 4.5 : Corrected area for the calculation of shear and normal stresses (Bardet, 1997).

4.4 Direct Shear Test on sand Reinforced with Randomly Distributed Fibers

Gary and Ohashi (1983) performed direct shear tests on dry sand specimens reinforced with different types of fibers. They developed theorical predictions based on a force equilibriums model and they compared their test results to the theorical predictions. The sand used in the direct shear tests was clean, quartz beach sand. Different types of fiber were used in order to provide a range of elastic moduli. Fibers diameters were from 1mm-2mm and the lengths were conducted on sand samples that relative densities of 20%-100% for both reinforced and unreinforced condition. Metal wires, natural and synthetic fibers were used as reinforcement. While most of the tests were conducted on fiber area ratios of 0.25% or 0.5%, the maximum amount fiber area ratio used was 1.67%. Direct shear tests were conducted as strain controlled and the test continued until a total displacement of 5mm or an average shear strain of 8% was recorded. The fibers were placed with various orientation angles. Tests were performed with different confining stresses up to 144 kN/m^2. The theoretical model in this study was developed for two different fiber orientations. Fibers that are perpendicular to the shear place can be seen in Figure 4.6 and fibers that are oriented at an arbitrary angle to the shear plane can be seen in Figure 4.7.

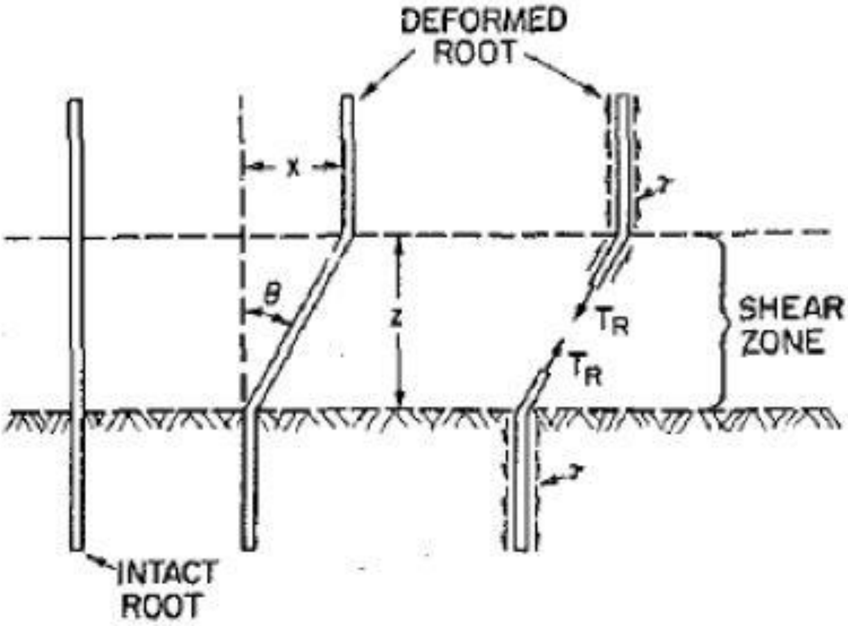

Figure 4.6 : Fiber Reinforcement Model for Perpendicular Orientation to Shear Surface (Gary and Ohashi, 1983).

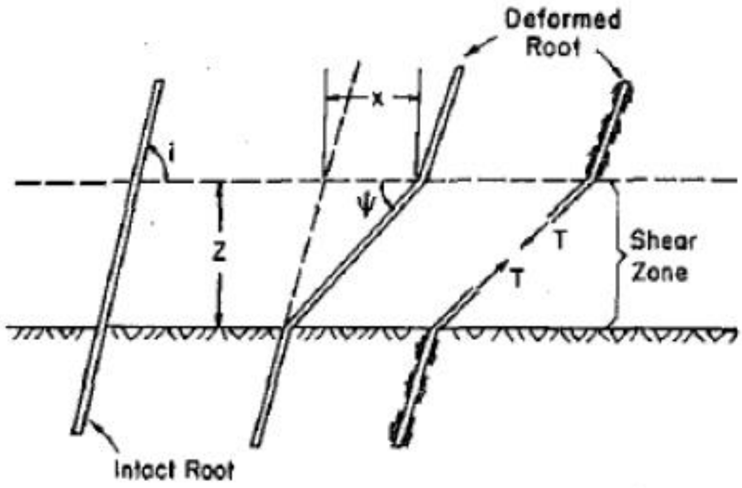

Figure 4.7 : Fiber Reinforcement Model for Fiber Oriented at Angle to Shear Surface (Gray and Ohashi, 1983).

The tensile force, formed because of shearing, has two components normal and tangential to the shear plane. It is stated that while tangential component resists shear directly, the normal component mobilizes shear resistance by increasing the confining stress. The shear strength increase caused by the addition of fibers is estimated as:

For perpendicular fibers:

$$\Delta S_R = t_R[\sin\theta + \cos\theta . tg\Phi] \tag{4.11}$$

For oriented fibers:

$$\Delta S_R = t_R[\sin(90-\Psi) + \cos(90+\Psi) . tg\Phi] \tag{4.12}$$

As the shear strength increase due to fiber reinforcement is represented by ΔS_R; tR represents the mobilized tensile strength of fibers per unit area of soil; angle of internal friction angle of sand is represented by Φ. The angle of shear distortion is represented by θ. The initial orientation angle of fiber with respect to shear surface is represented by i. The horizontal of shear displacement is represented by x; the thickness of shear zone is represented by z and the shear distortion ratio is represented by k. The mobilized tensile strength per unit area of soil (t_R) is defined as the product of tensile stress developed in the fiber at the shear plane and the fiber area ratio (Gray and Ohashi, 1983).The possible tensile stress distributions along the length of the fiber are considered as linear or parabolic distributions. The resulting tensile stress at the shear plane for corresponding tensile stress distributions are calculated as given:

For linear distribution:

$$\sigma_R = \sqrt{\left(\frac{4E_R.\tau_R}{D_R}\right)} \times \sqrt{\{\, z\,(\sec\theta - 1)\}} \qquad \textbf{(4.13)}$$

For parabolic distribution:

$$\sigma_R = \sqrt{\left(\frac{8E_R.\tau_R}{3D_R}\right)} \times \sqrt{\{\, z\,(\sec\theta - 1)\}} \qquad \textbf{(4.14)}$$

Where E_R is the modulus of the fiber, τ_R is the skin friction stress along the fiber, D_R is the diameter of fiber and z is the thickness of the shear zone. According to the experimental test results and theoretical predictions, it is stated that fiber reinforcement increased the ultimate shear strength. Figure 4.8 presents that, as the number of fibers increased the shear strength increased alongside with the limited reductions in the post peak shearing resistance. It is stated that the initial orientation angle of fibers did not affect the stress-strain relations in a considerable amount but it affected the peak shear resistance. According to the test results, an initial fiber orientation of 60° is calculated as the optimum orientation for maximum shear strength increase. The average strength increases recorded for loose and dense sand specimens were similar. On the other hand, in loose sand specimens larger strains were necessary for reaching the peak shear resistance. The effect of fiber length was to increase the shear strength of the reinforced sand until reaching a limit. It is stated that the predictions made based on force equilibrium theory were consistent with the experimental test results (Gray and Ohashi, 1983).

Yetimoglu and Salbas (2003), performed direct shear tests to investigate the shear strength of randomly reinforced sand specimens. They did some shear box test on samples with different fiber content percentage and compared its results with unreinforced samples test results. The results of their experiments are shown in Figure 4.9, Figure 4.10, Figure 4.11 and Figure 4.12. The experimental study consists of 45 direct shear tests performed at the vertical normal stresses of σn= 100, 200 and 300kPa. The loading rate was chosen to be 0.002 mm/s and shear stresses were recorded until a total displacement of 16 mm. The sand used in experiments was the uniform quartz river sand. It was sieved through ASTM 10 and washed through ASTM 20. The sand properties are shown in Table 4.1.

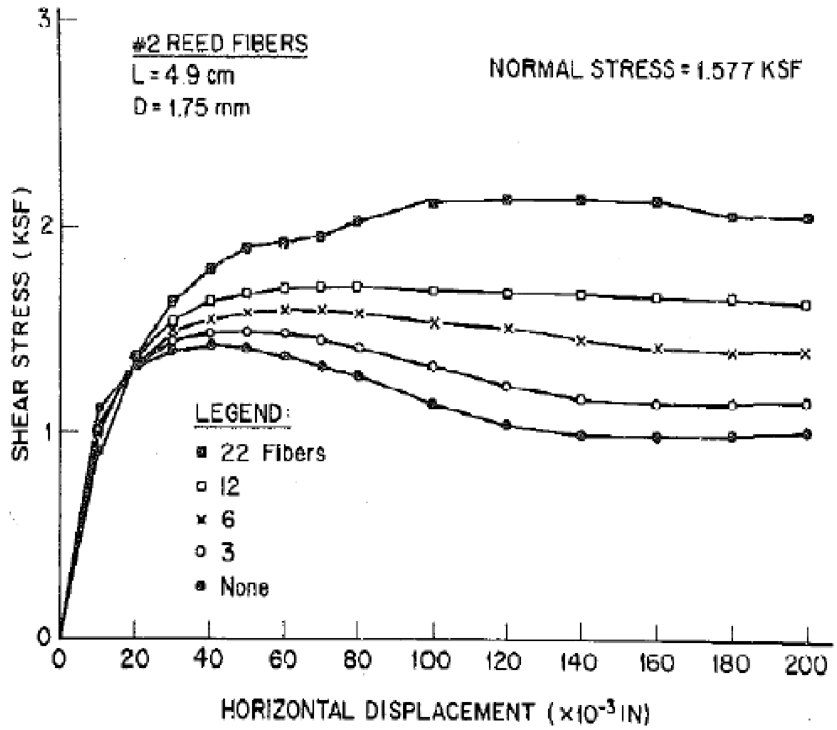

Figure 4.8 : Influence of Number of Fibers on Stress-Deformation Behavior of a Dense Sand (Gray and Ohashi, 1983).

Table 4.1 : Properties of sand (Yetimoglu and Salbas, 2003).

Property	Value
Specific Gravity	2.64
Maximum dry unit weight (kN/m^3)	17.48
Minimum dry unit weight (kN/m^3)	14.92
Maximum void ratio	0.77
Minimum void ratio	0.51
Coarse sand fraction (%)	2
Medium sand fraction (%)	53
Fine sand fraction (%)	45
Effective grain size D_{10} (mm)	0.20
D_{60} (mm)	0.33
D_{30} (mm)	0.26
Coefficient of uniformity C_u	1.65
Coefficient of curvature C_c	1.02
c (kPa) *	0
ϕ (deg) *	42
*Obtained from direct shear tests at D_r = 70 %	

Sand samples were reinforced with polypropylene fibers with the contents (ρ) of 0.10%, 0.25%, 0.50% and 1.00% of the dry weight of sand. The composite was mixed thoroughly by hand to prepare a uniform mixture. Figure 4.9, Figure 4.10, Figure 4.11 and Figure 4.12 show the shear stress-horizontal displacement curves obtained from the reinforced sands including fiber contents of 0.1%, 0.25%, 0.5%and 1.0% respectively. The peak shear strength angle and cohesion values are calculated. As the fiber content increased up to 1%, the shear strength angle decreased slightly from 42.3° to 40.4°. On the other hand, the cohesion value stayed constant as 0kPa. The results show that fiber inclusion affected the peak shear stresses and horizontal displacements at which peak shear stresses mobilized significantly. A smaller loss of post-peak strength was observed for fiber-reinforced samples. It is concluded that fiber reinforcement can change the brittle behavior of sand to a more ductile behavior (Yetimoglu and Salbas, 2003).

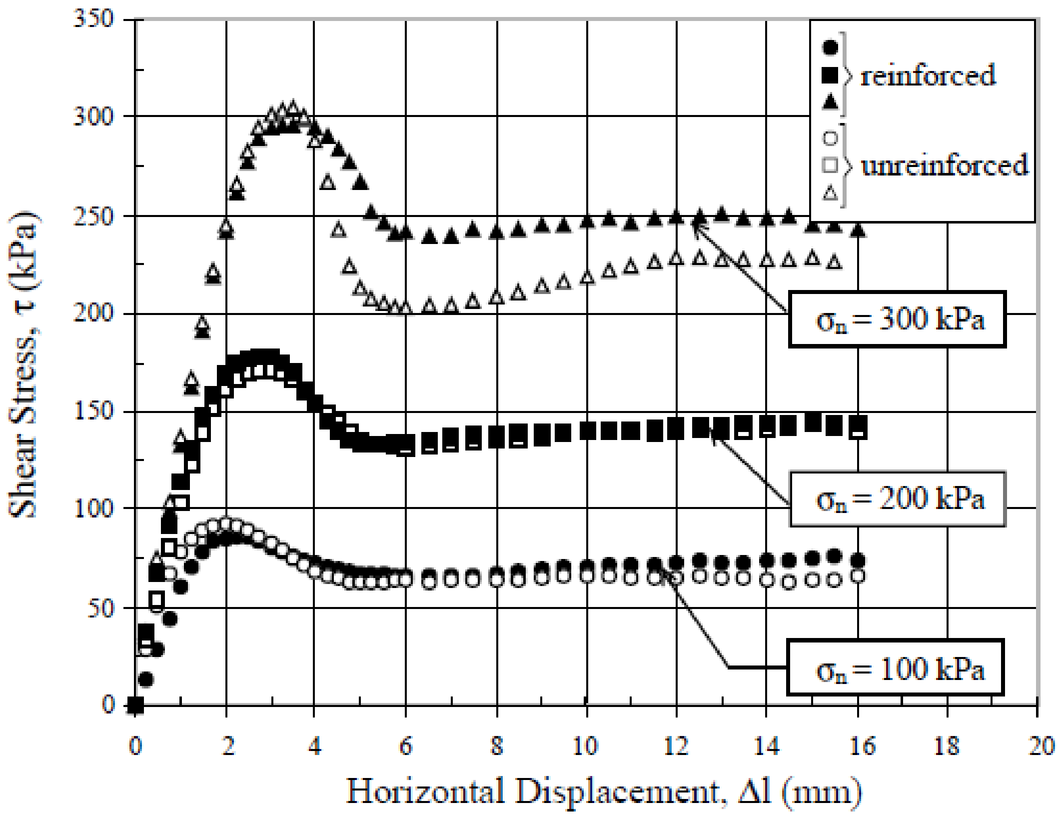

Figure 4.9 : Shear stress-horizontal displacement response for unreinforced sand and reinforced sand with fiber content of ρ = 0.10% (Yetimoglu and Salbas, 2003).

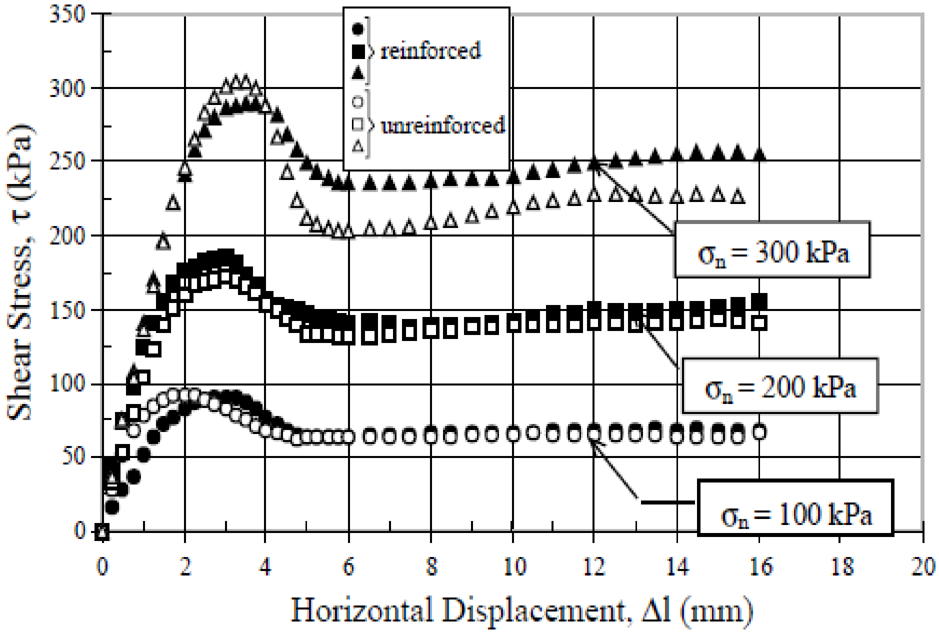

Figure 4.10 : Shear stress-horizontal displacement response for unreinforced sand and reinforced sand with fiber content of ρ=0.25% (Yetimoglu and Salbas, 2003).

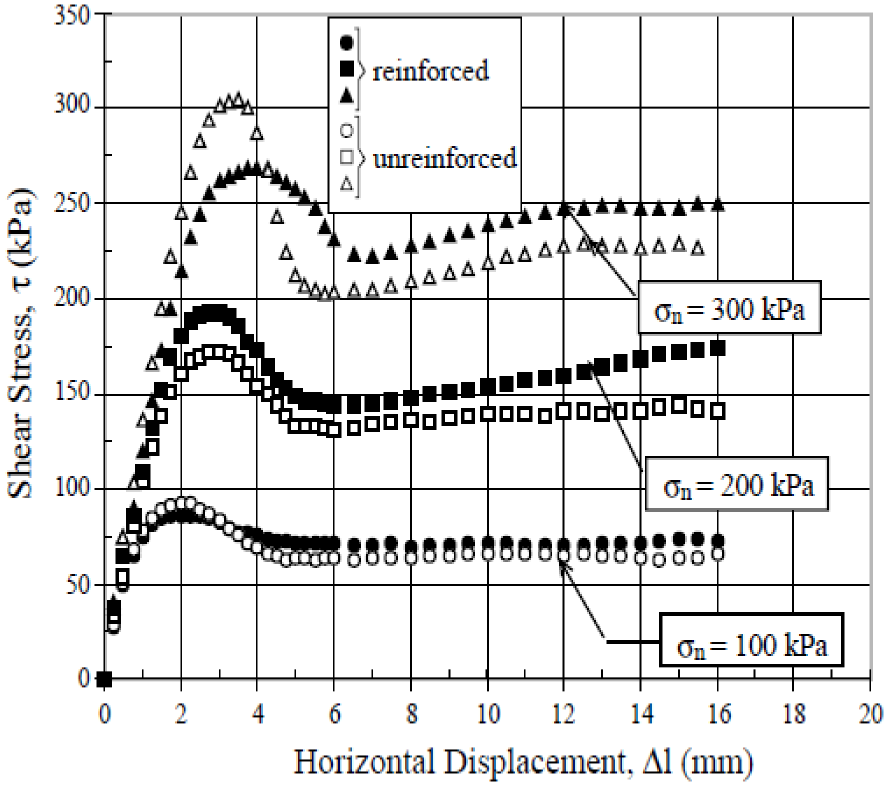

Figure 4.11 : Shear stress-horizontal displacement response for unreinforced sand and reinforced sand with fiber content of ρ=0. 5% (Yetimoglu and Salbas, 2003).

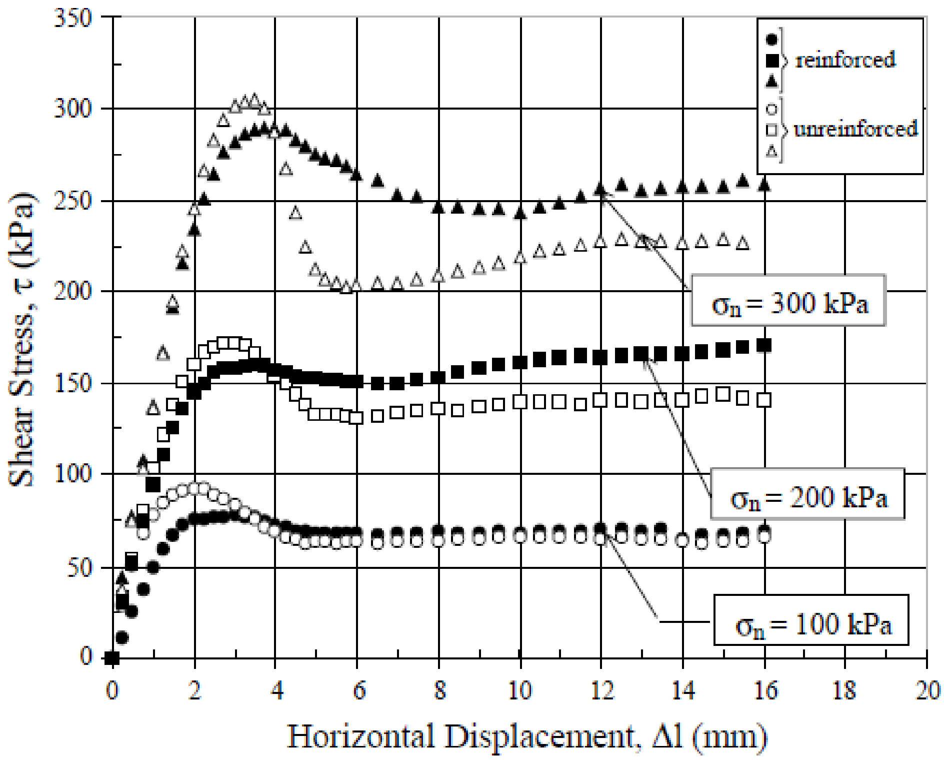

Figure 4.12 : Shear stress-horizontal displacement response for unreinforced sand and reinforced sand with fiber content of ρ=0. 5% (Yetimoglu and Salbas, 2003).

In another study by Hande Gerkus (2011), it was performed the direct shear test on unreinforced and reinforced sand samples at a relative densities of 55% to determine the effect of fiber inclusion on the shear strength parameters of sand. The samples are prepared at the water content of 10% and fiber contents of 0.5%, 0.75%, 1.0% and 2.0% were tested. The strain rate was kept constant at 0.12mm/min all tests. The shear stress-horizontal displacement curves of unreinforced and reinforced sand specimens are presented in Figure 4.13, Figure 4.14 and Figure 4.15. Under the same normal stress, the peak stress value recorded increases due to fiber addition. In the reinforced specimens, the recorded shear stress values increased with increasing horizontal displacement. Especially under the normal stresses of 200 kPa and 300 kPa, first the fiber-reinforced samples have reached a peak value at around horizontal displacement of 5 mm that is followed by small decrease of the shear stress and gradual increase due to increase in horizontal stress. While the effect of fiber inclusion is more apparent for the normal stresses of 100 kPa and 200 kPa, the fiber addition caused an insignificant improvement for the normal stress of 300 kPa.

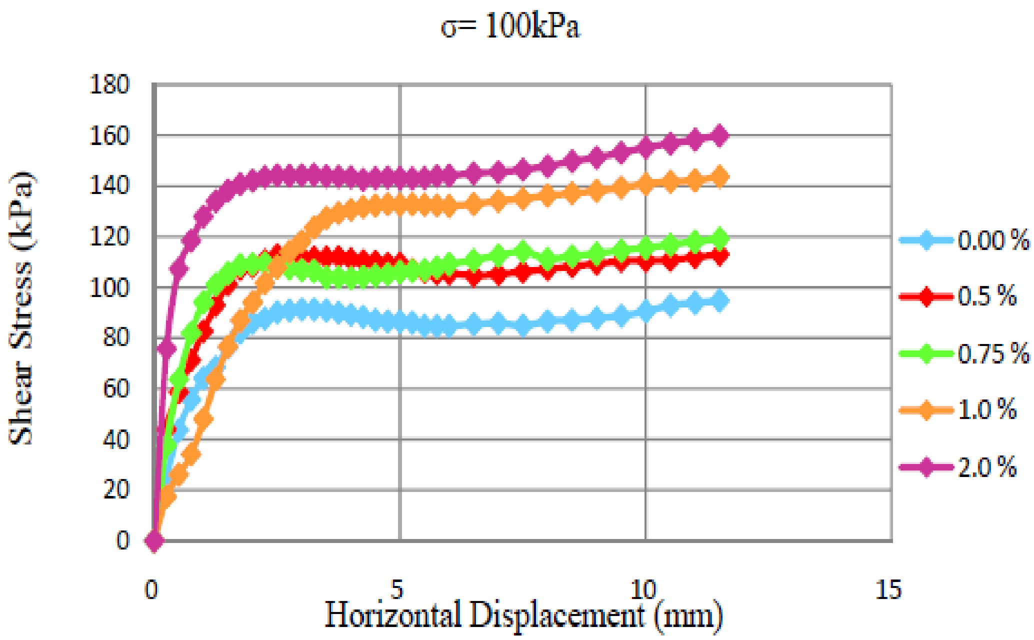

Figure 4.13 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ = 100 kPa (Hande Gerkus, 2011).

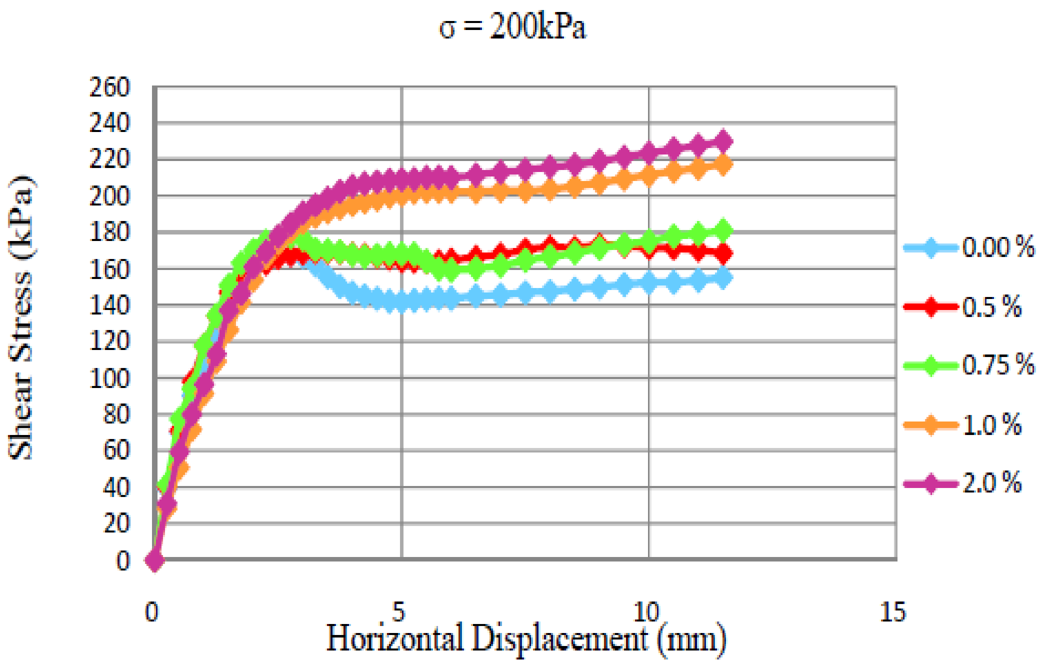

Figure 4.14 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ = 200 kPa (Hande Gerkus, 2011).

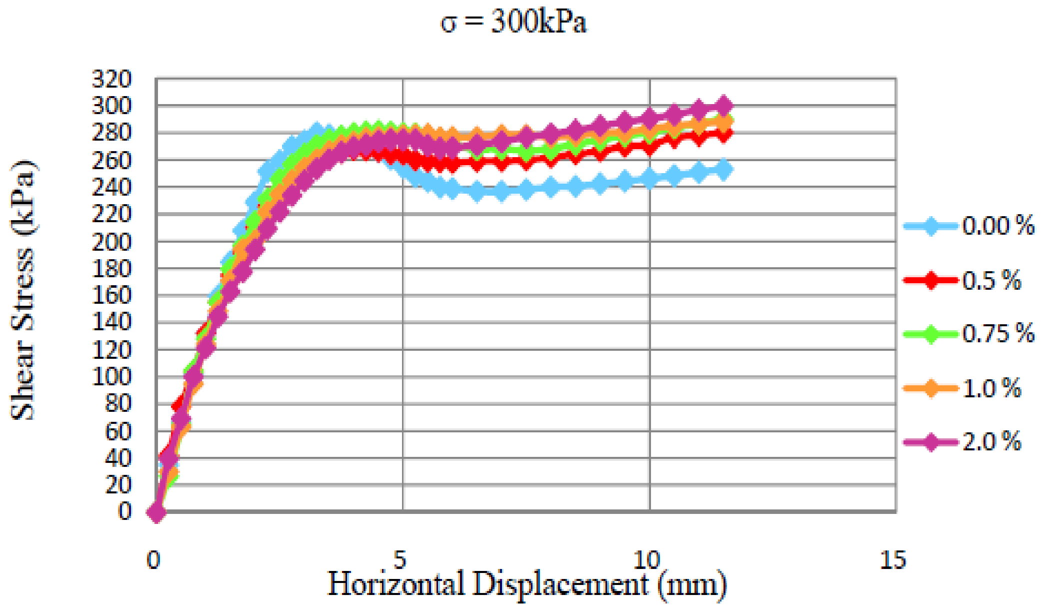

Figure 4.15 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ = 300 kPa (Hande Gerkus, 2011).

4.5 Results

In this part of the thesis, firstly brief information about the static behavior of sand is presented. Secondly, previous studies on fiber-reinforced sand are presented. In the previous studies, fiber reinforced sand samples are subjected to consolidation and direct shear tests. In these studies, the effect of several parameters on the static behavior of fiber-reinforced sand is investigated. The experiments were conducted on same sand types, fiber types, different fiber contents, fiber aspect ratios, confining pressures, strain rates, sample preparation and testing methods. The direct shear test were conducted in two different as low and high relative densities. The previous studies showed that, fiber reinforcement improves the static behavior of sand. On the other hand, the degree of improvement obtained at the studies depends on the tested parameters.

5. EXPERIMENTAL STUDY

The experimental study mainly consist of direct shear tests that are conducted unreinforced and reinforced sand samples prepared at two main relative densities. Three different fiber contents are tested at the same certain rate. The effect of fiber inclusion on the shear strain parameters are compared using the test results of direct shear tests on dry samples. In other part of experimental study it was employed the Permeability test as Constant Head and Falling Head tests. As same as direct shear test three different fiber contents are tested at the same certain rate as unreinforced, 0.1%, 0.5%, and 1.0% fiber content by weight of soil sample.

5.1 Test Materials

The Akpinar sand is used in the experimental study. The sand is first washed through ASTM # 10 sieve. According to the Unified Soil Classification System, the sand is classified as SP. The constant head permeability test is conducted and the Istanbul Technical University, Civil Engineering Faculty, and Construction Materials Laboratory have conducted analysis of the Akpinar sand. The results obtained from the binocular microscopes showed that sand particles are clean and semi-circular, semi-angular shaped. The composition mainly consist of quartz and it includes magnetite in small amounts. The grain size distribution of Akpinar sand is shown in Figure 5.1. Moreover, the parameters of the sand are shown in Table 5.1.

The fiber used as the reinforcement is called Virgin Homopolymer Polypropylene (FORTA MIGHTY-MONO) fiber. It is made of pure homopolymer polypropylene. The photo of fibers is shown in Figure 5.1 are produced according to ASTM C1116. The fiber parameters are presented in Table 5.2.

Table 5.1 : Properties of sand

Property	value
Specific Gravity	2.69
Maximum void ratio	0.876
Minimum void ratio	0.547
Permeability (m/sec)	4x10-4
Sand fraction (%)	99
Fine materials (%)	1
Effective grain size D10 (mm)	0.22
D60 (mm)	0.35
D30 (mm)	0.27
Coefficient of uniformity Cu	1.6
Coefficient of Gradation Cc	0.95
c (kPa)	0
ϕ (deg)	40

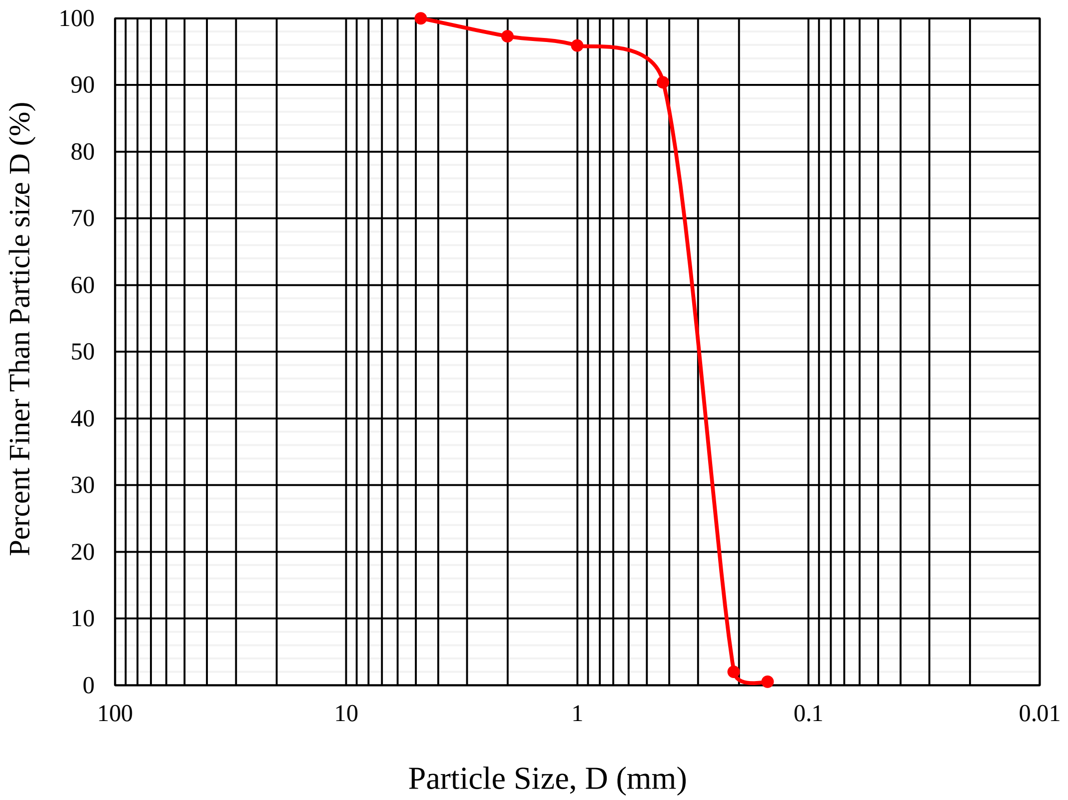

Figure 5.1 : Grain Size Distribution of Akpinar Sand.

Table 5.2 : Fiber Properties

Color	White
Structure	Single Fiber
Specific Weight (g/cm^3)	0.91
Length	19 mm
Water Absorption	0
Tensile Stress	570-660 MPa

Figure 5.2 : VHP Fibers.

5.2 Direct Shear Test

Direct Shear test are performed to determine the effect of fibers on the shear strength parameters of sand. The test were conducted on samples of 60×60×25 rectangular cube, having a relative density around 55%. The amount of fiber added to the sand was taken as a percentage of the dry weight of sand. The fibers are accepted as a part of the sand skeleton. The fiber contents tested are 0.1%, 0.5%, and 1.0%. The results are compared with the results of unreinforced sand. For sample preparation, fibers were separated to enable a uniform mixture. During the sample preparation, the difficulty of obtaining a uniform mixture increases as the amount of fiber mixed into soil increase. The sample including of fibers is shown in Figure 5.3 and it can be seen that a uniform mixture is obtained with randomly distributed fibers.

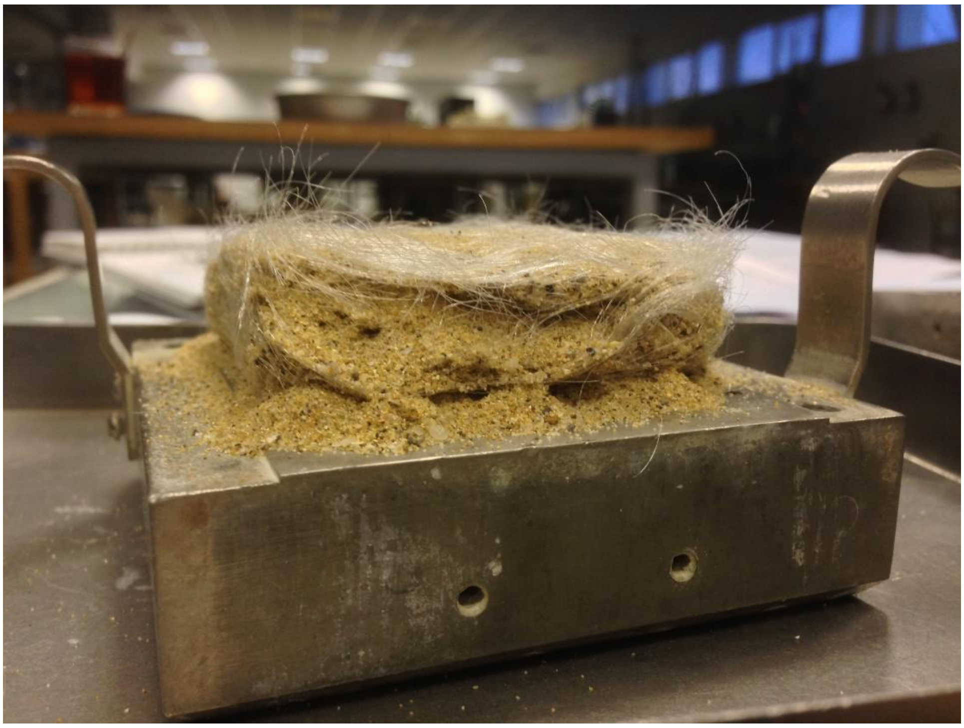

Figure 5.3 : Fiber reinforced sample

The sample is placed into the direct shear box tamping lightly. The porous paper are placed before and after the sample is placed. The surface of the sample must be smooth as shown in Figure 5.4. The direct shear test apparatus used is shown in Figure 5.5. The normal vertical stresses of σn =100, 200 and 300 kPa are applied at unreinforced and reinforced sand sample and the loading rate was 0.12 mm/min for all tests. The settlements are recorded for 15 minutes after the sample is loaded. Shear stress were recorded as a function of horizontal displacement up to total displacement of 12 mm in order to observe the post failure behavior.

Figure 5.4 : Direct Shear Test Sample

Figure 5.5 : ITU Soil Mechanics Laboratory’s Direct Shear Test Apparatus

5.3 Permeability Test

Permeability tests are performed by applying different fiber content to a soil sample and measuring the rate of the flow of water through soil. In this part of study, in order to determine the flow of water through soil and effects of fiber on this rate, the constant – head permeability test method was employed. The samples are prepared at relative densities of around 55% and the fiber contents are 0.1%, 0.5% and 1% of the dry weight of sand. In this experiment as well as the water permeability, coefficients of the volume change and the effects of fiber on volume changes were also examined. In order to obtain permeability, constant head test and its apparatus for water volume changing was used. The mold, which was used for constant head test, has a height of 35 cm and 10 cm diameter and in falling head test the mold has a height of 11.7cm and 10 cm diameters, which are shown in Figure 5.6 and Figure 5.7.

Figure 5.6 : Constant head permeability test mold

The constant head test were conducted on samples at a relative density around 55%-70%. The amount of fiber added to the sand was taken as a percentage of the dry weight of sand. The fibers are accepted as a part of the sand skeleton. The fiber contents tested are 0.0%, 0.1 %, 0.5% and 1.0%.

Figure 5.7 : Falling head permeability test mold

In falling head test in order to obtain volume of water were applied vacuum with a 0.01 MPa and CO_2 with a 1bar pressure to the samples at a two different 60% and 70% relative densities.

6. EXPERIMENTAL RESULTS

In this part, the experimental results of the laboratory tests conducted at Istanbul Technical University, Soil Mechanics and Geotechnical Laboratory are presented.

6.1 Direct Shear Tests Results

In order to determine the effect of fibers on sand samples Direct Shear Test are employed on unreinforced and reinforced sand samples at two different relative densities. Low relative densities were in a loss condition around 20% - 35% and a dense condition with a relative density in around 60-70%. The samples are prepared at the different contents of fiber as 0.0%, 0.1%, 0.5% and 1.0% of dry weight of sand. Fiber were mixed randomly into the soil. As the fiber content increases, mixing fibers into the sand become harder so higher fiber content are not tested. The strain rate was kept constant at 0.12mm/min for all tests. This test was repeated for two types of different relative density and its results were compared with each other. The shear stress-horizontal displacement curves and of unreinforced and reinforced sand specimens are presented in Figure 6.1, Figure 6.2 and figure 6.3 for test with high relative density and , Figure 6.8, Figure 6.9, Figure 6.10 for test with high relative density. Under the same normal stress, the peak stress and residual values recorded increase due to fiber addition. In the reinforced specimens, the recorded shear stress value increased with increasing horizontal displacement. Especially under the normal stress of 200 and 300 kPa, first the reinforced samples reached a peak value at around horizontal displacement between 3-5 mm that are followed by a small decrease of the shear stress and gradual increase due to increase in horizontal stress.

6.1.1 Dense sample results

In this part of Direct Shear Test the soil samples were performed with a different rang of fiber content as 0.0%, 0.1%, 0.5% and 1.0% of dry weight of sand at a high relative density around 50%-75%. In the unreinforced and reinforced samples, the recorded shear stress value with increasing horizontal displacement. Especially under the normal stress of 200 and 300 kPa. The shear stress-horizontal displacement, vertical displacement curves between unreinforced and reinforced sand specimens are presented in Figure 6.1, Figure 6.2, Figure 6.3 and Figure 6.4.

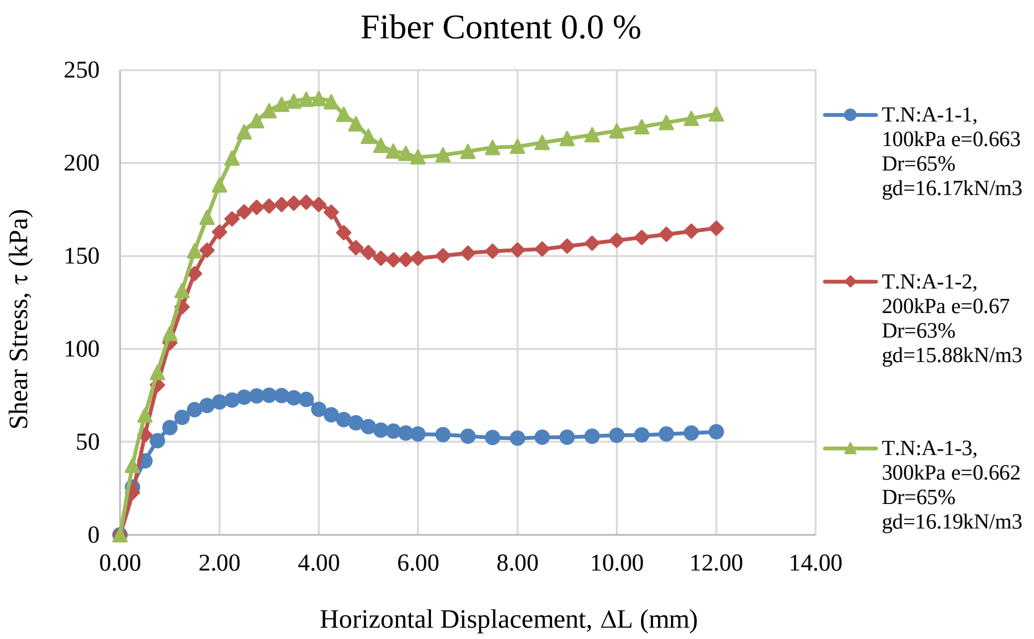

Figure 6.1 : Shear stress-horizontal displacement response for unreinforced sand samples at the normal stresses of σ_n=100, 200 and 300 kPa at high relative density.

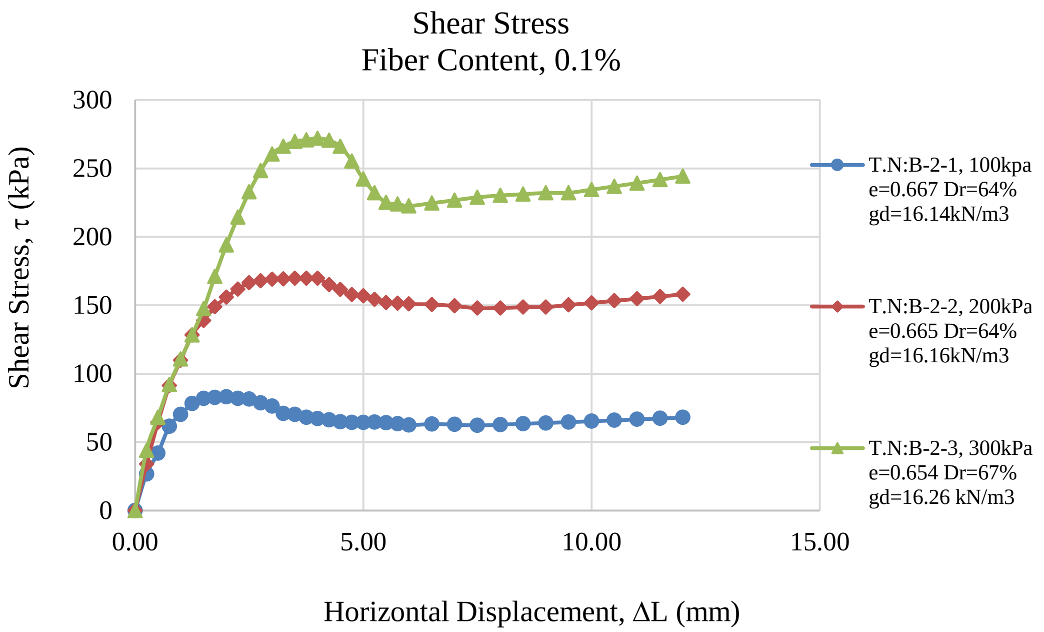

Figure 6.2 : Shear stress-horizontal displacement response for a reinforced sand samples with fiber content of ρ=0.1% at the normal stresses of σ_n=100, 200 and 300 kPa at high relative density.

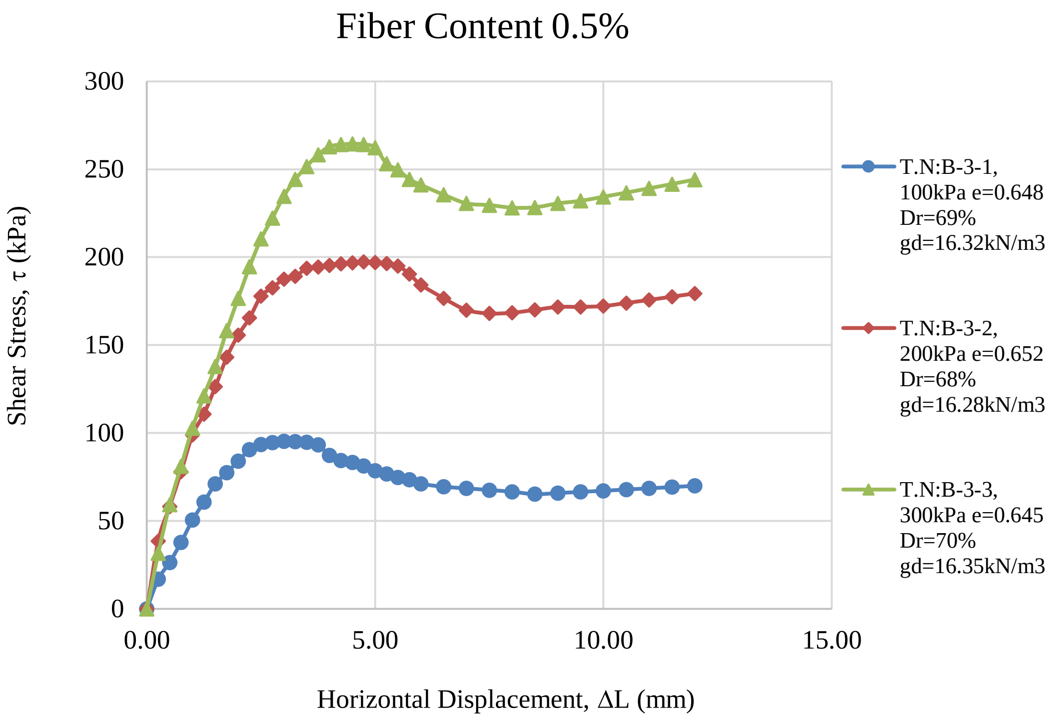

Figure 6.3 : Shear stress-horizontal displacement response for a reinforced sand samples with fiber content of ρ=0.1% at the normal stresses of σ_n=100, 200 and 300 kPa at high relative density.

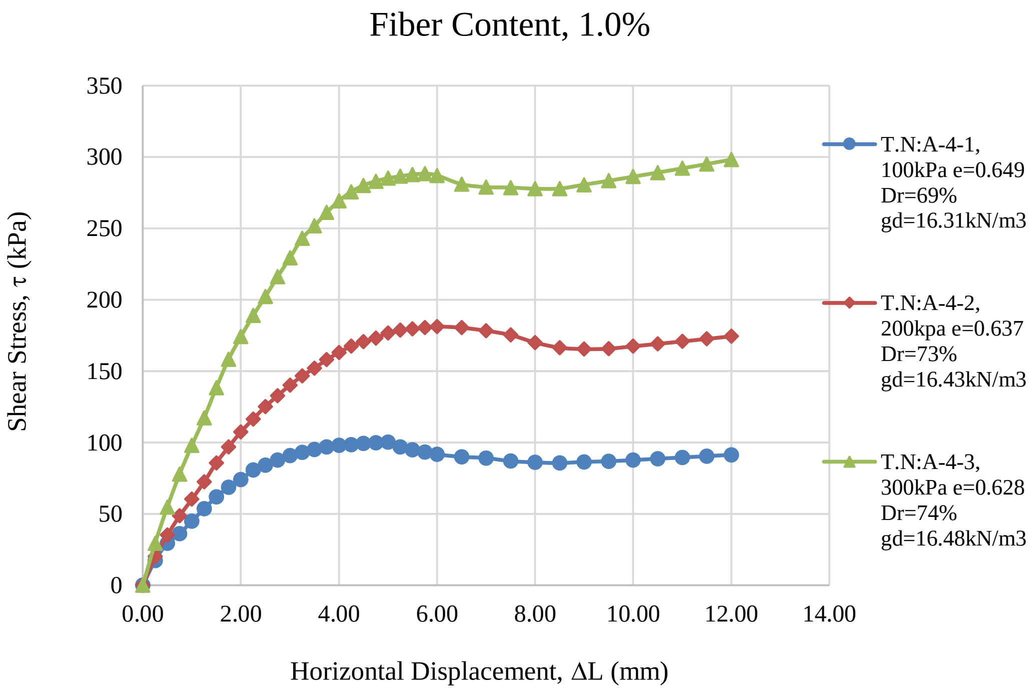

Figure 6.4 : Shear stress-horizontal displacement response for a reinforced sand samples with fiber content of ρ=0.1% at the normal stresses of σ_n=100, 200 and 300 kPa at high relative density.

The vertical displacement and effect of fiber on compression and dilation curve are shown in Figure 6.5, Figure 6.6, Figure 6.7 and Figure 6.8.

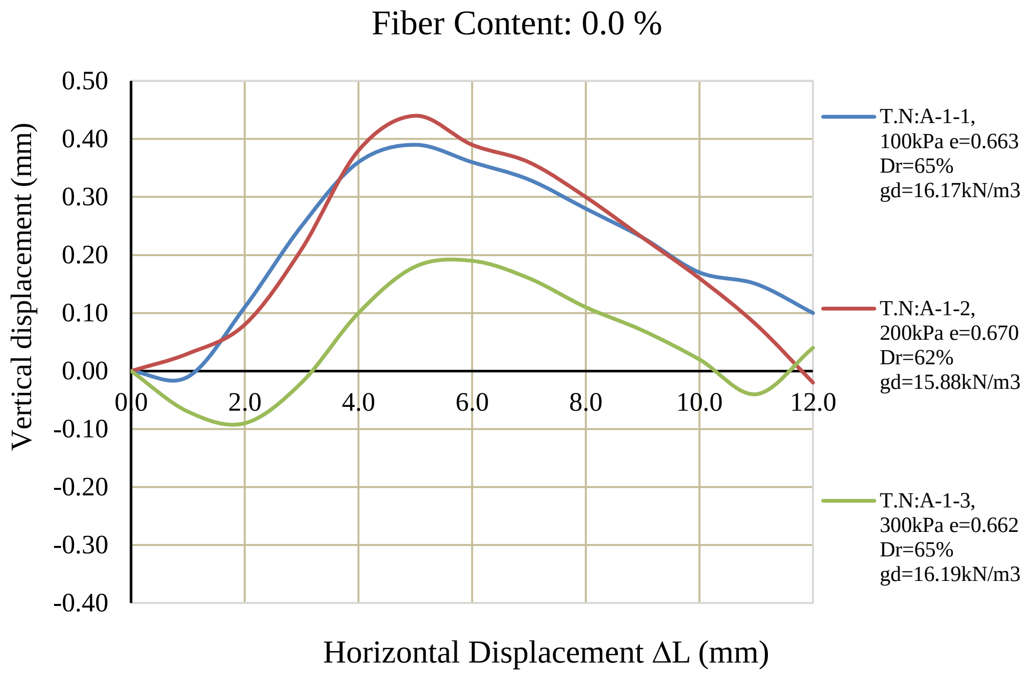

Figure 6.5 : Shear stress-horizontal displacement versus vertical displacements for a unreinforced sand samples at the normal stresses of σ_n=100, 200 and 300 kPa for at a high relative density.

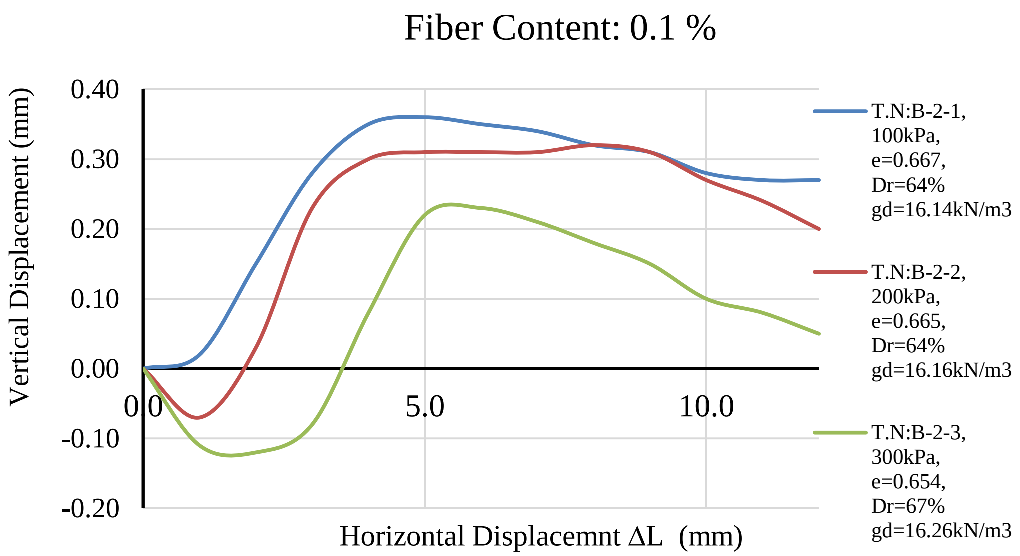

Figure 6.6 : Shear stress-horizontal displacement versus vertical displacements response for a reinforced sand samples with fiber content of ρ=0.1% at the normal stresses of σ_n=100, 200 and 300 kPa for at a high relative density.

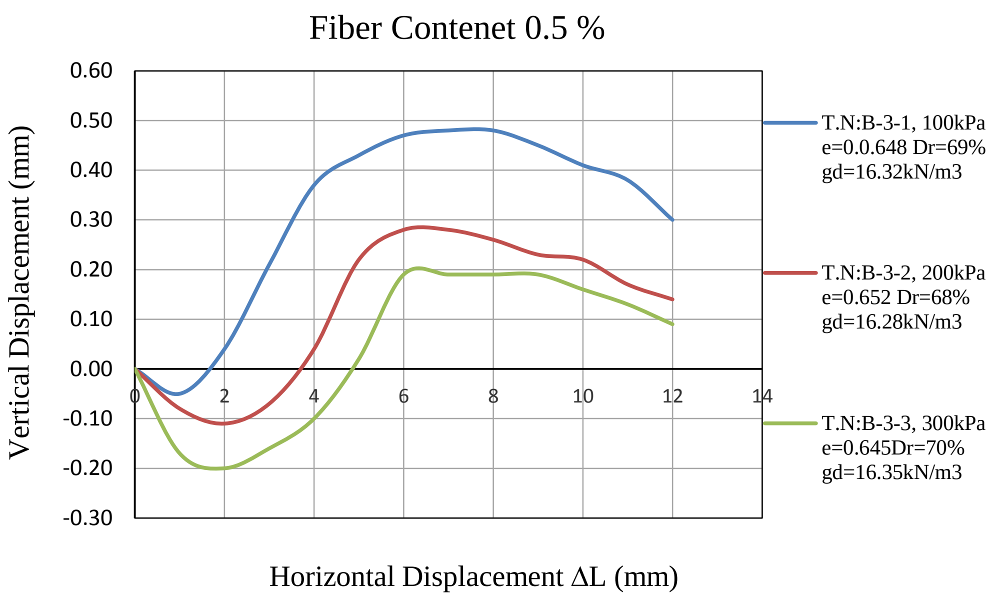

Figure 6.7 : Shear stress-horizontal displacement versus vertical displacements response for a reinforced sand samples with fiber content of ρ=0.5% at the normal stresses of σ_n=100, 200 and 300 kPa for at a high relative density.

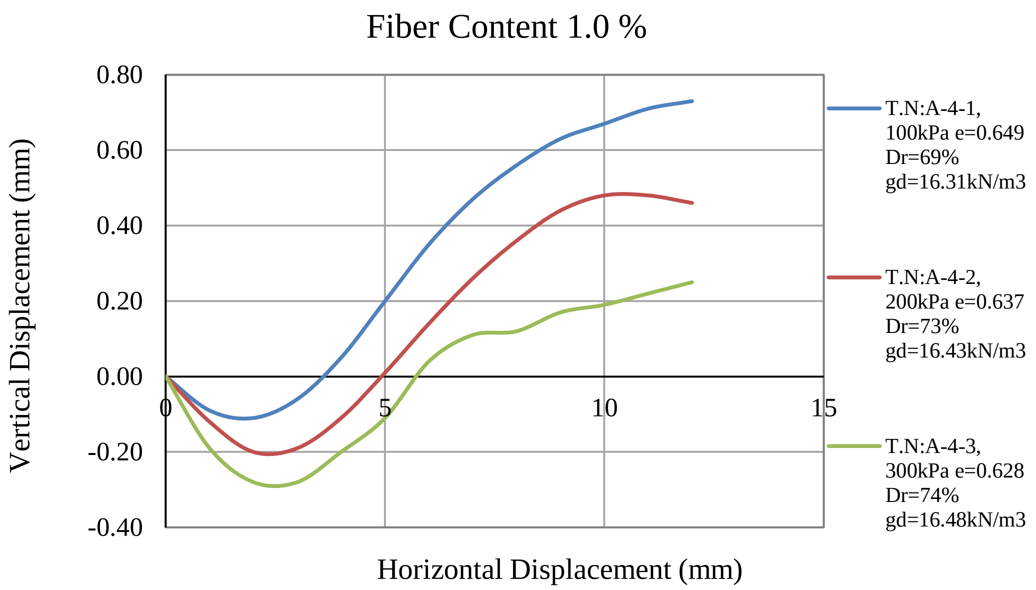

Figure 6.8 : Shear stress-horizontal displacement versus vertical displacements response for a reinforced sand samples with fiber content of ρ=1.0% at the normal stresses of σ_n=100, 200 and 300 kPa for at a high relative density.

The shear stress parameters for the unreinforced and reinforced sand for test A at a high relative density (60%-75%) is shown in Figure 6.9, Figure 6.10, Figure 6.11 and Figure 6.12. In the unreinforced sample while the apparent cohesion intercept is calculated as zero, the shear strength angle is 36^{o} .

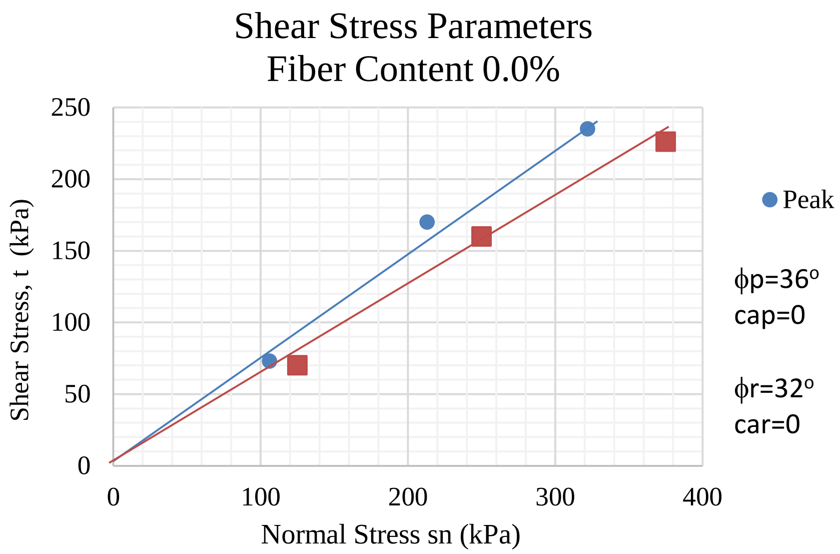

Figure 6.9 : Shear stress-Normal stress graph for unreinforced specimens.

The shear stress-normal stress graph for the sand reinforced with fiber content of 0.1% is shown in Figure 6.10. While the apparent cohesion is calculated as 10 kPa, the shear strength angle is calculated 37° and an apparent cohesion value of zero kPa for residual point with a 32° of shear strength angle. The addition of fiber increased both of parameters.

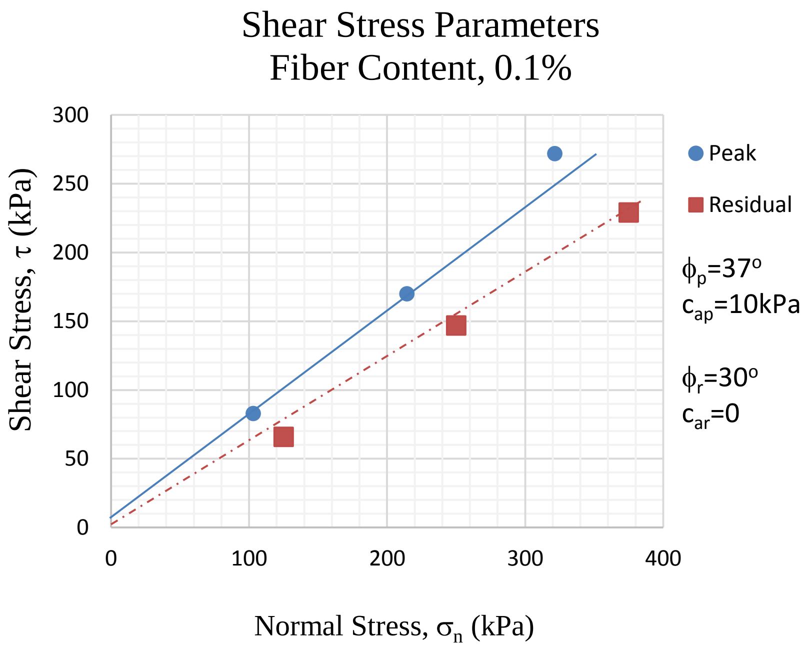

Figure 6.10 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.1%.

The shear stress-normal stress graph for the sand reinforced with the fiber content of 0.5% is shown in Figure 6.11. While the apparent cohesion is calculated as 20kPa for the peak and 5 kPa for residual point, the angle of shear strength at peak point is calculated 37°, the residual shear strength is 34°. The addition of fiber increased both of parameters.

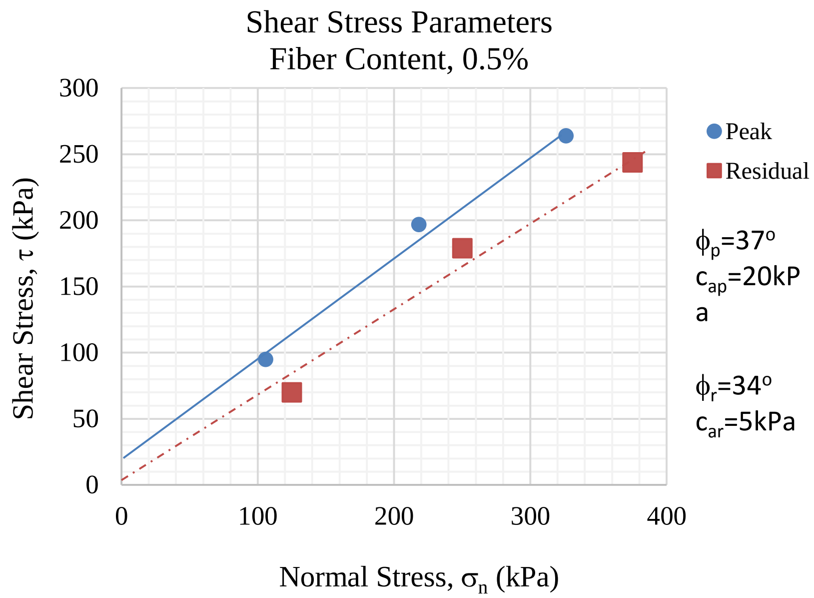

Figure 6.11 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.1%

The shear stress-normal stress graph for the sand reinforced with the fiber content of 1.0% is shown in Figure 6.12. While the apparent cohesion is calculated as 27 kPa, the angle of friction at peak point is calculated 36° while the residual shear strength is calculated 33° and residual apparent cohesion is calculated 20 kPa. The addition of fiber increased both of parameters.

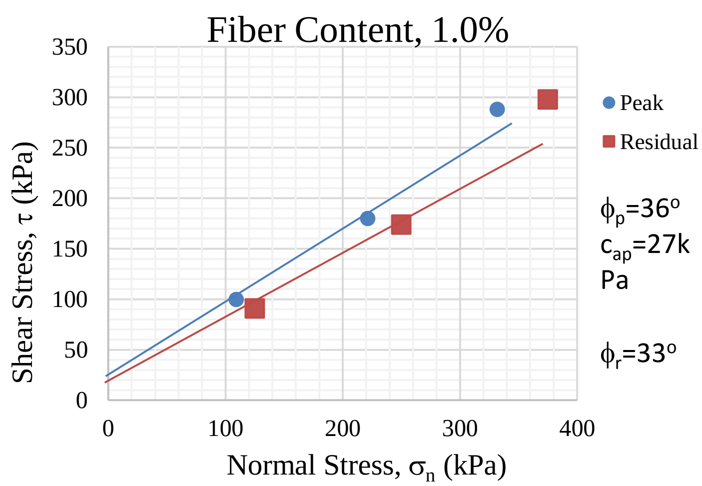

Figure 6.12 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.1%.

The shear stress-horizontal displacement curves and comparison between unreinforced and reinforced sand specimens under the same normal stress according to fiber content are presented in Figure 6.13, Figure 6.14 and Figure 6.15.

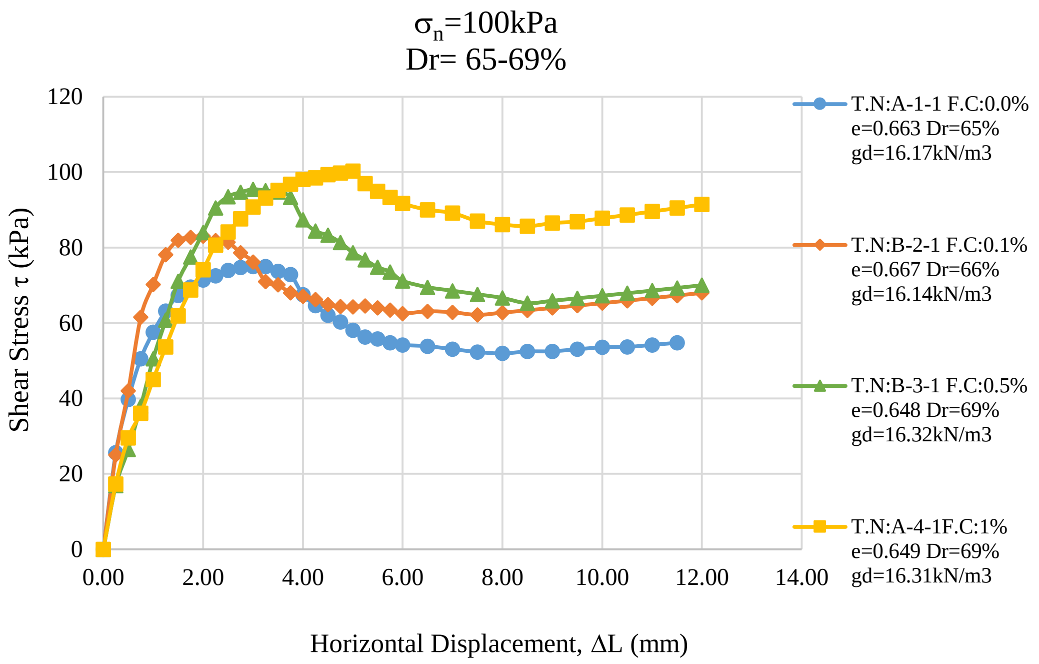

Figure 6.13 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=100 kPa for dense samples.

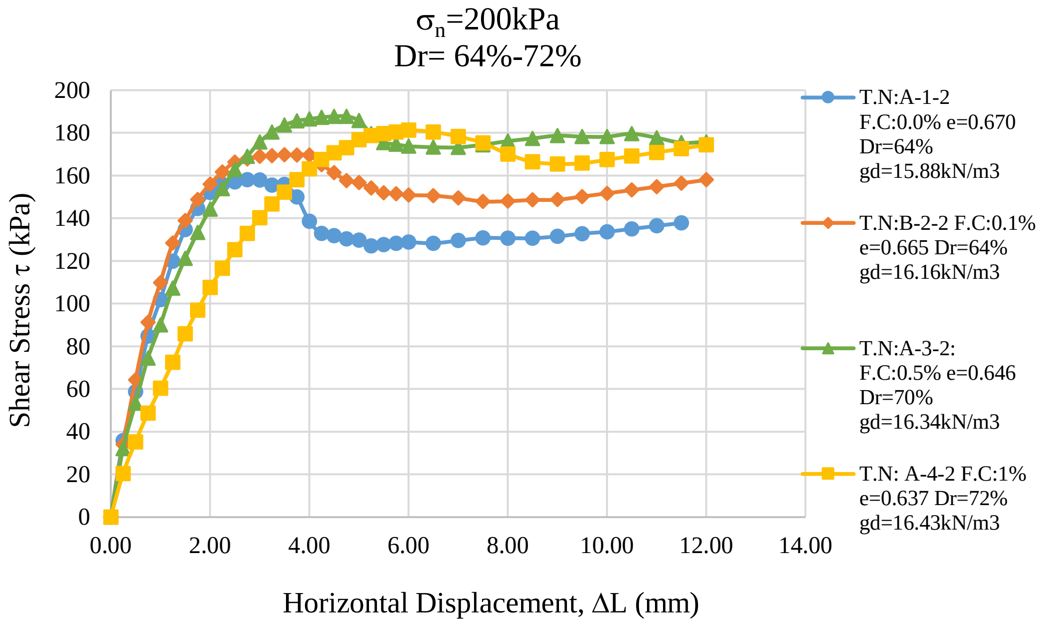

Figure 6.14 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=200 kPa for dense samples.

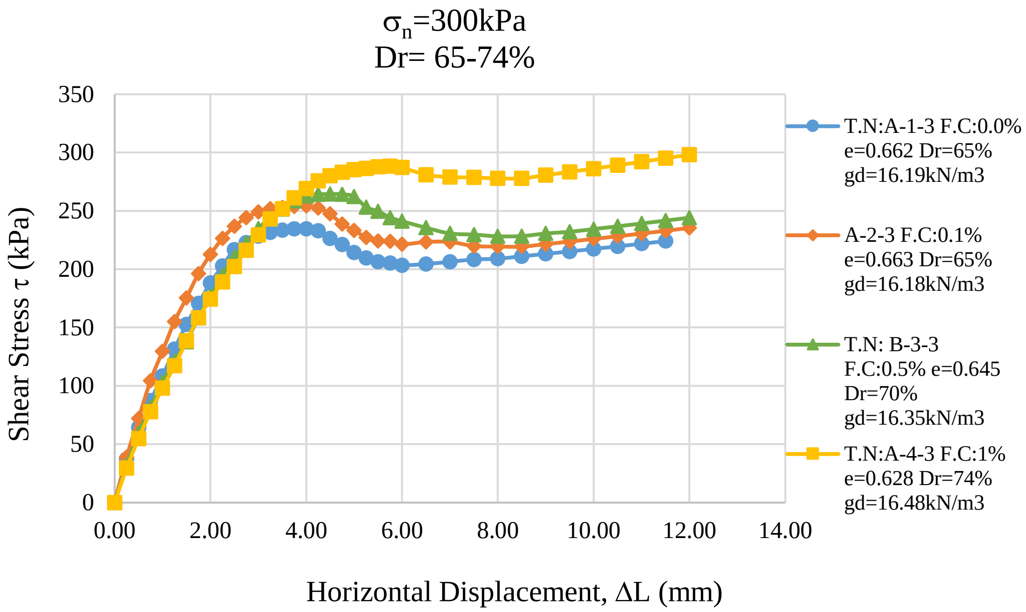

Figure 6.15 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=300 kPa for dense.

The summary of direct shear tests results for samples with a high relative density are presented in Table 6.1 for peak shear strength parameters and Table 6.2 for residual shear strength parameters. Best on the resulted, the randomly distributed fiber addition affected the shear strength parameters of sand. The peak shear strength angles (ϕ_p) were ranged from 36^o for unreinforced sample, 37^o for samples with 0.1% and 0.5% fiber content and 35^o for samples with a 1.0% fiber content. Other parameters like Void Ratio (e) and Maximum Dry Unite Weight (γ_d) are presented in Tables 6.1 and Table 6.2.

Table 6.1 : Direct Shear Test Result at peak point for sample at a high relative density around 63%-74%.

Dense Samples, D_r: 63%-74% at Peak Point								
Fiber Content (%)	Test Number	e	D_r (%)	γ_d	σ_p (kPa)	τ_p (kPa)	ϕ_p (o)	c_{ap} (kPa)
0	A-1-1	0.663	65	16.17	106	73	36	0
	A-1-2	0.67	63	15.88	213	179		
	A-1-3	0.662	65	16.19	322	235		
0.1	B-2-1	0.667	64	16.14	103	83	37	0
	B-2-2	0.665	64	16.16	214	170		
	B-2-3	0.654	67	16.26	321	272		
0.5	B-3-1	0.648	69	16.32	106	95	37	20
	B-3-2	0.652	68	16.28	218	197		
	B-3-3	0.645	70	16.35	326	264		
1	A-4-1	0.649	69	16.31	109	100	35	25
	A-4-2	0.637	73	16.43	221	180		
	A-4-3	0.628	74	16.48	332	288		

Table 6.2 : Direct Shear Test Result at residual point ΔL=12mm for sample at a high relative density around 63%-74%.

Dense Samples, D_r: 63%-74% at Residual Point								
Fiber Content (%)	Test Number	e	D_r(%)	γ_d	σ_r (kPa)	τ_r (kPa)	ϕ_r (o)	c_{ar} (kPa)
0	A-1-1	0.663	65	16.17	125	70	32	0
	A-1-2	0.67	63	15.88	250	160		
	A-1-3	0.662	65	16.19	375	226		
0.1	B-2-1	0.667	64	16.14	125	66	30	0
	B-2-2	0.665	64	16.16	250	147		
	B-2-3	0.654	67	16.26	375	229		
0.5	B-3-1	0.648	69	16.32	125	75	34	5
	B-3-2	0.652	68	16.28	250	179		
	B-3-3	0.645	70	16.35	375	250		
1	A-4-1	0.649	69	16.31	125	91	31	20
	A-4-2	0.637	73	16.43	250	174		
	A-4-3	0.628	74	16.48	375	298		

In this test, Figure 6.16 and 6.17 show the apparent cohesion and shear strength values calculated for unreinforced and reinforced sand specimens by performing Direct Shear Test on samples. The apparent cohesion and shear strength values increases due to fiber addition. The peak and residual shear strength angle (ϕ_p) and (ϕ_r) are calculated and are presented in Figure 6.18 and Figure

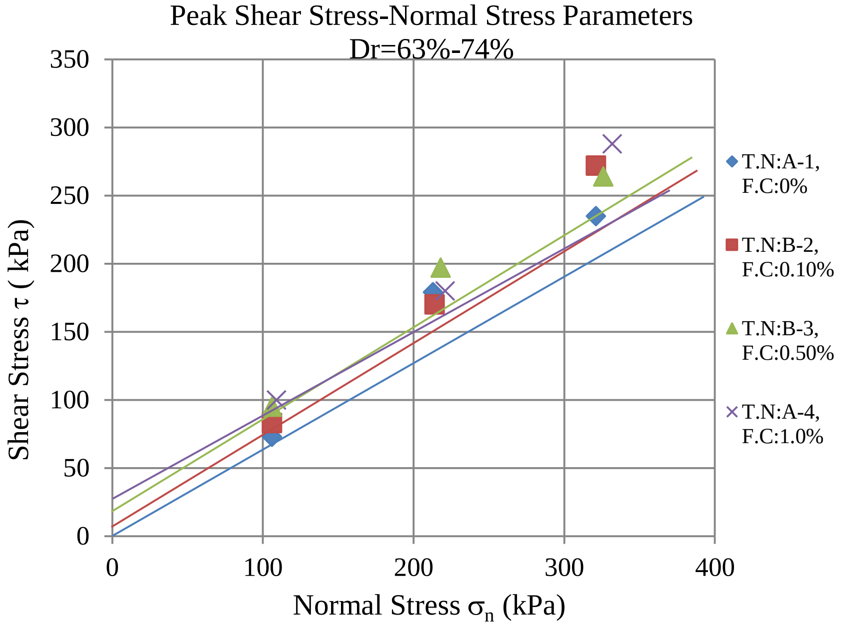

Figure 6.16 : Apparent cohesion and peak shear strength values according to fiber addition.

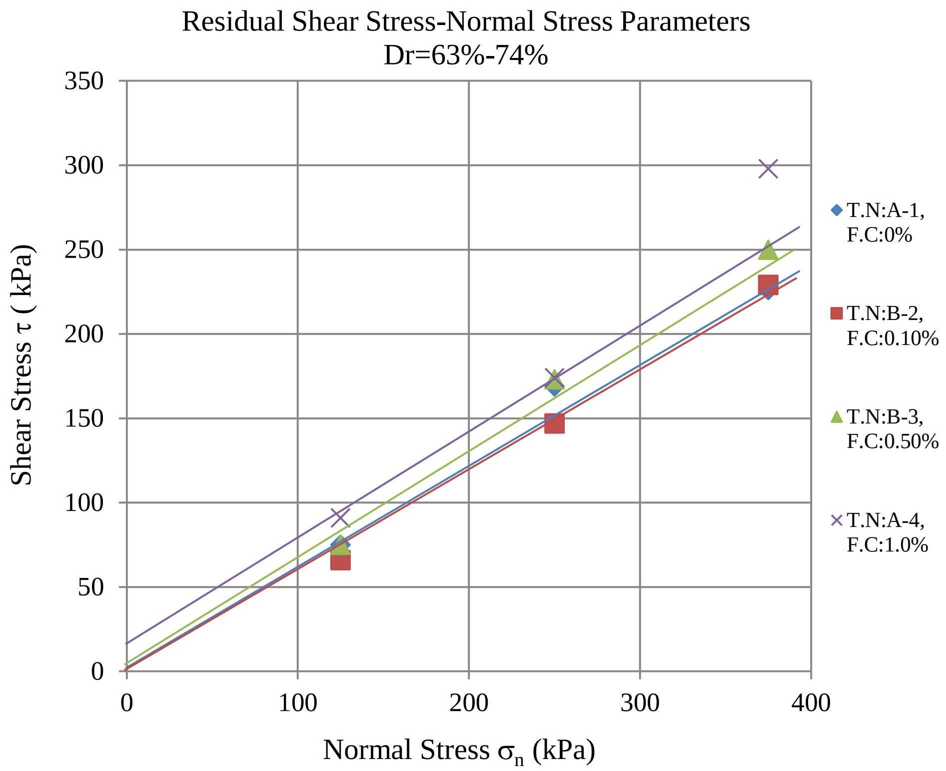

Figure 6.17 : Apparent cohesion and residual shear strength values according to fiber addition.

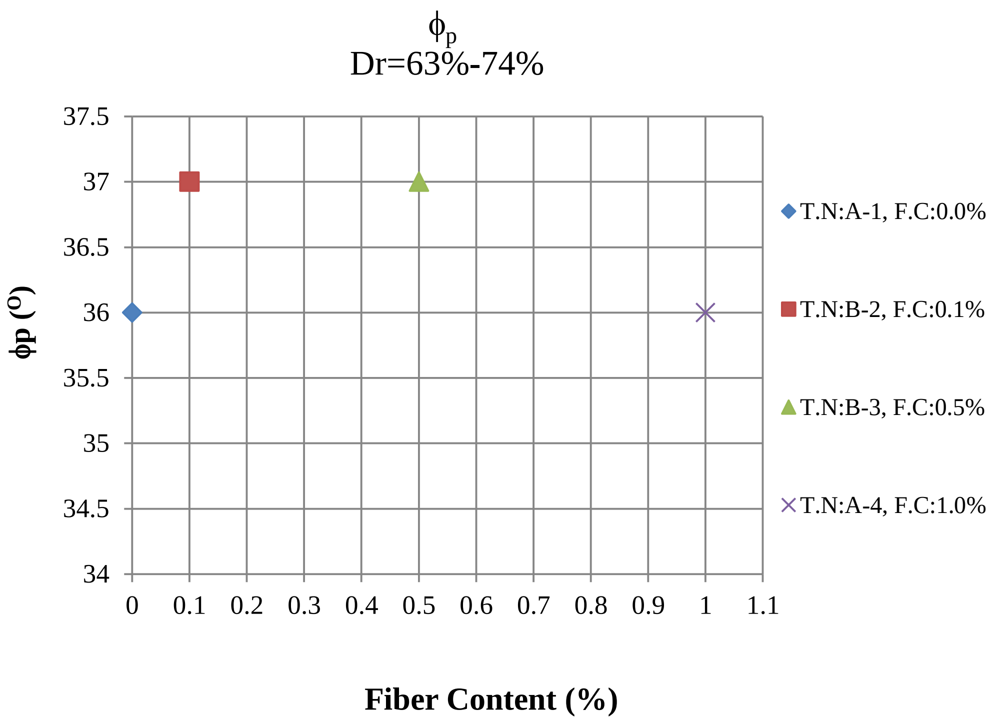

Figure 6.18 : The peak shear strength angle.

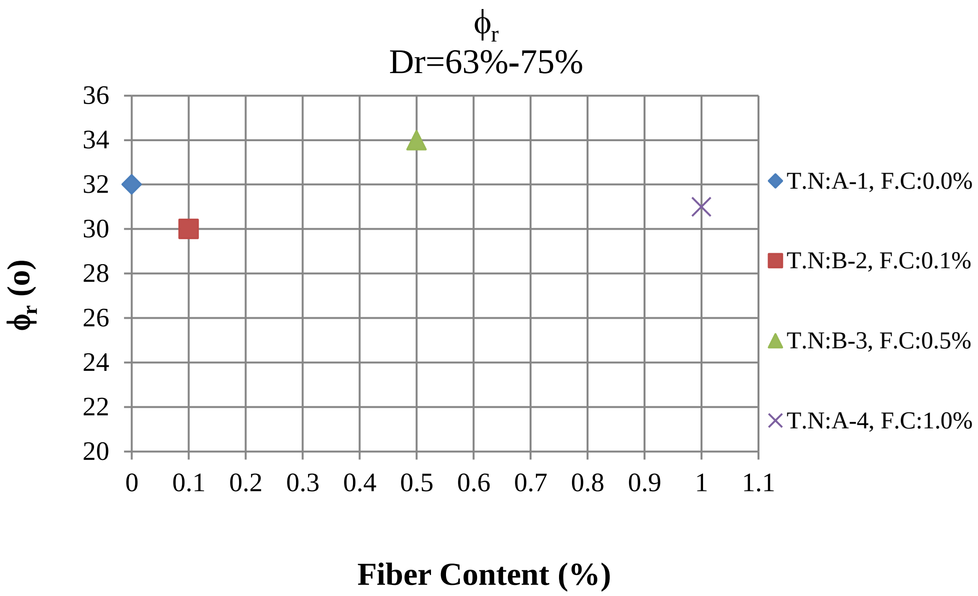

Figure 6.19 : The residual shear strength.

6.1.2 Loose sample test results

In the second part of this test the soil samples were performed with a different rang of fiber content as 0.0%, 0.1%, 0.5% and 1.0% of dry weight of sand at a low relative density around 20%-37%. The shear stress-normal stress graph for the unrein-forced and reinforced sand for this test are shown in Figure 6.20, Figure 6.21, Figure 6.22 and Figure 6.23.

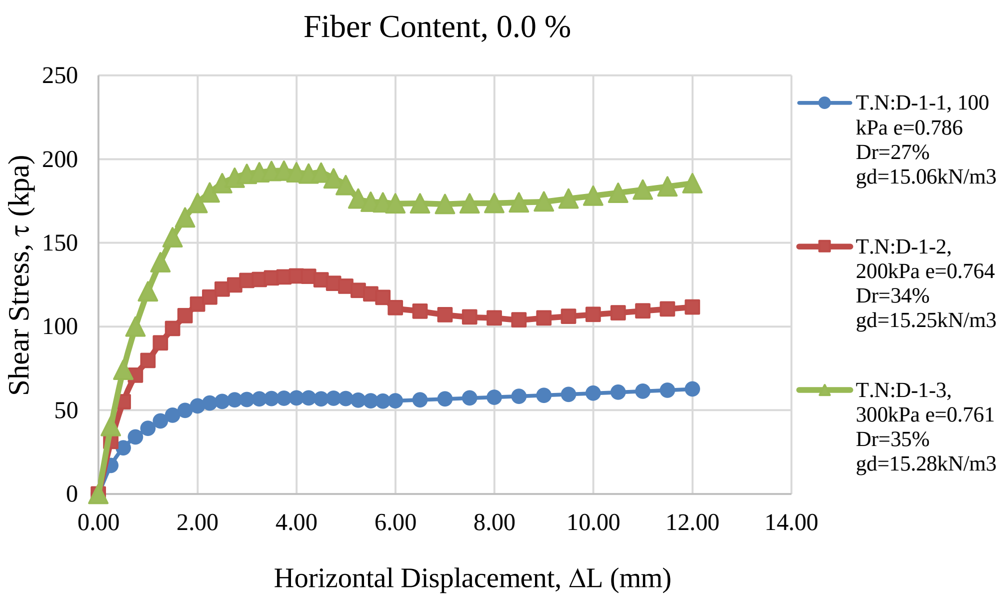

Figure 6.20 : Shear stress-horizontal displacement response for unreinforced sand samples at the normal stresses of σ_n=100, 200 and 300 kPa for at low relative density around 27%-35%.

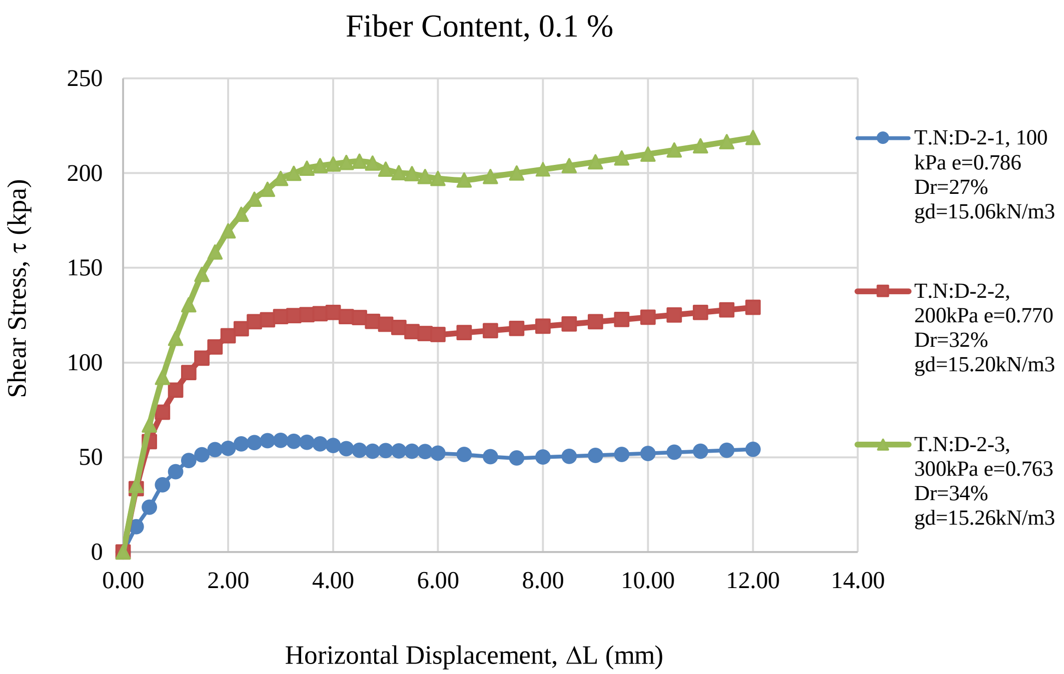

Figure 6.21 : Shear stress-horizontal displacement response for a reinforced sand samples reinforced sand with fiber content of ρ=0.1% at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density round 27%-34%.

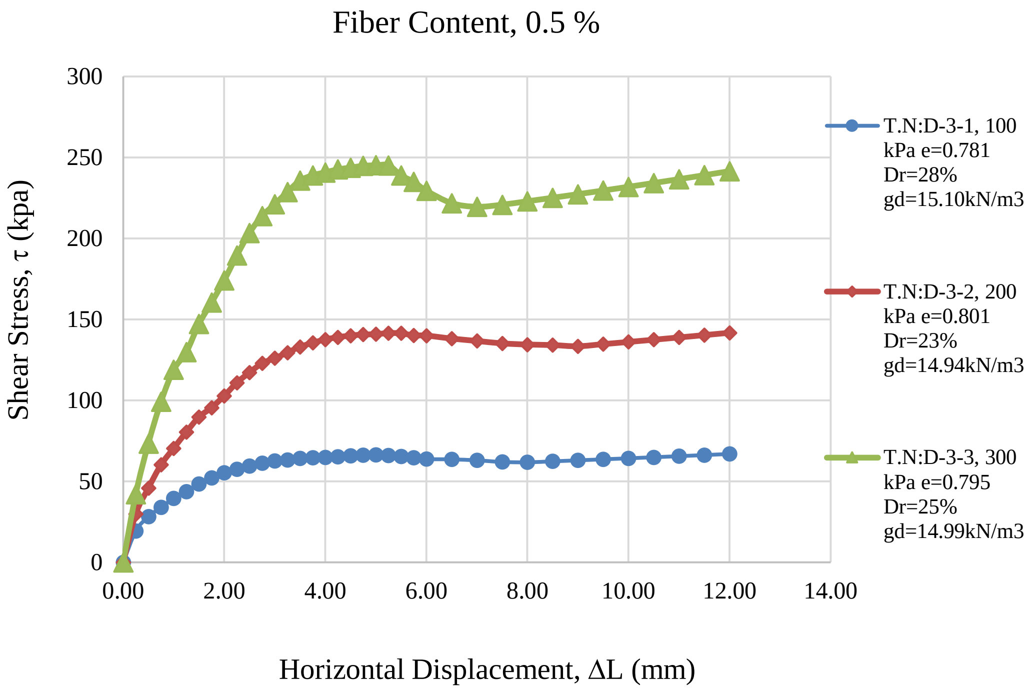

Figure 6.22 : Shear stress-horizontal displacement response for a reinforced sand samples reinforced sand with fiber content of ρ=0.5% at the normal stresses of σn=100, 200 and 300 kPa for at a low relative density around 27%-34%.

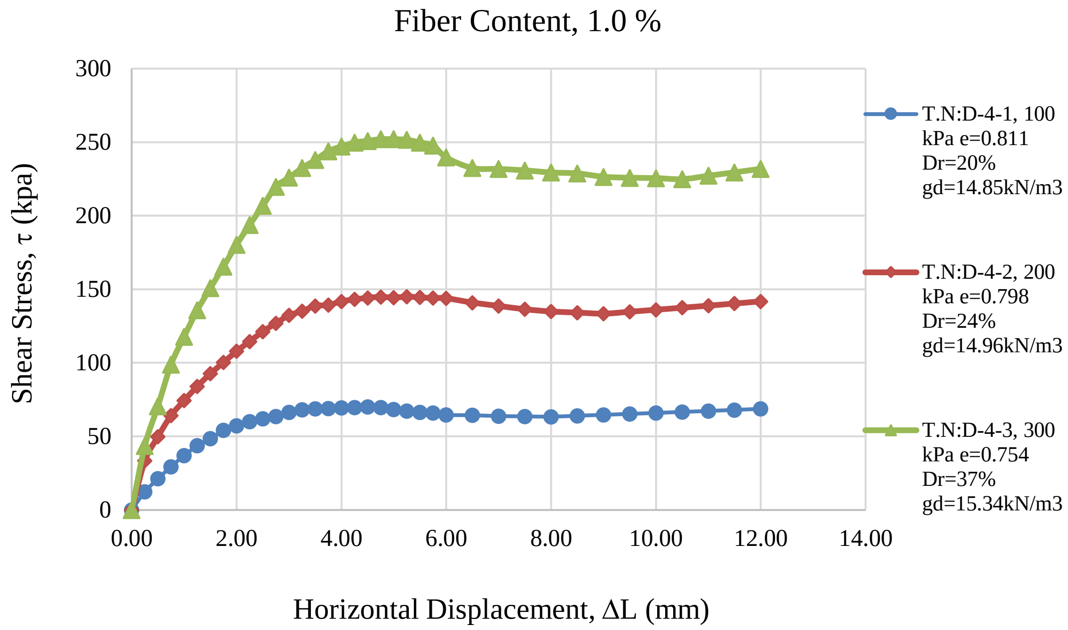

Figure 6.23 : Comparison stress-horizontal displacement response for a reinforced sand samples reinforced sand with fiber content of ρ=1.0% at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density around 27%-34% density.

Vertical displacement and effect of fiber on compression and dilation curve are shown in Figure 6.24, Figure 6.25, Figure 6.26 and Figure 6.27.

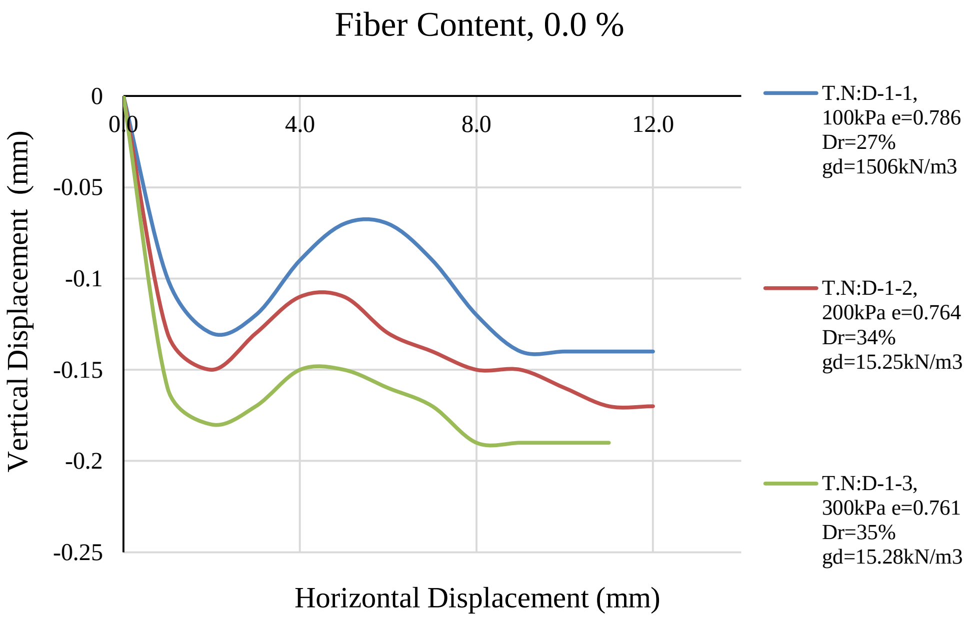

Figure 6.24 : Shear stress-horizontal displacement versus vertical displacements for the unreinforced sand samples at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density around 27%-35%.

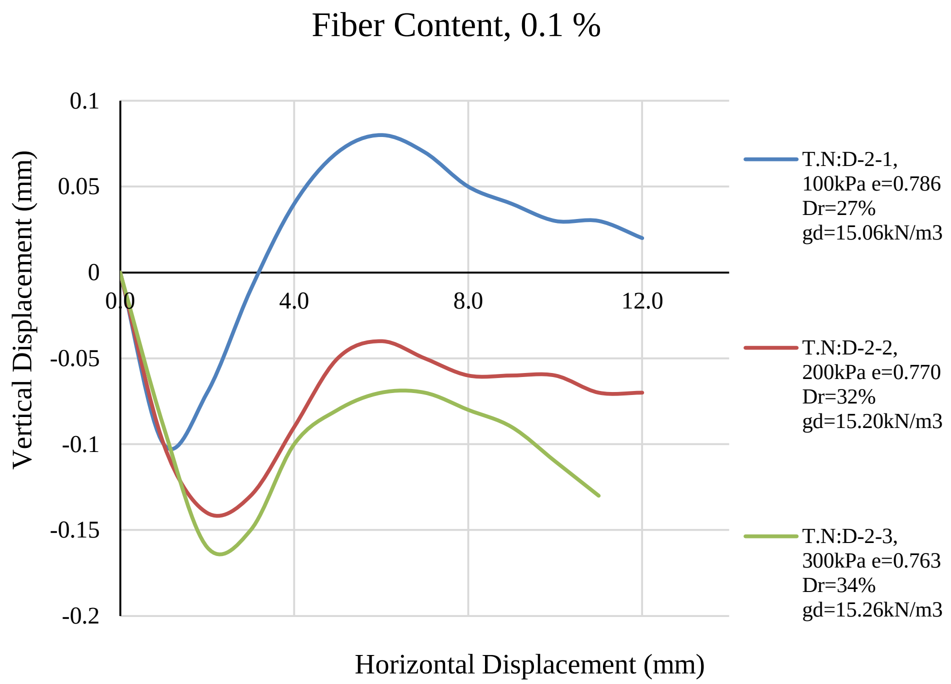

Figure 6.25 : Shear stress-horizontal displacement versus vertical displacements for the unreinforced sand samples at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density around 27%-35%.

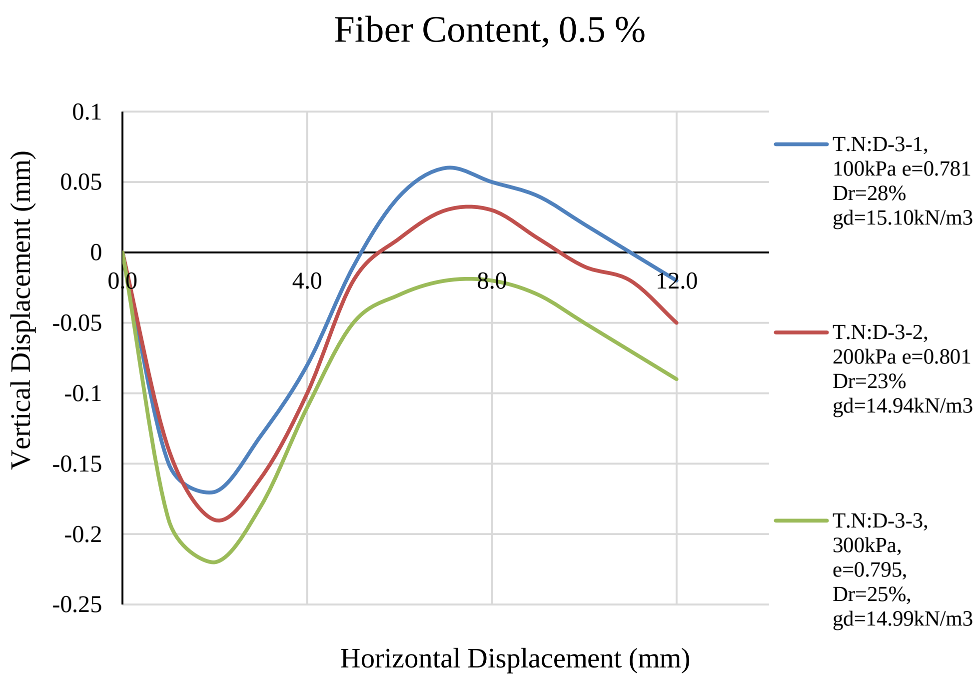

Figure 6.26 : Shear stress-horizontal displacement versus vertical displacements response for reinforced sand samples with fiber content of ρ=0.5% at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density around 23%-28%.

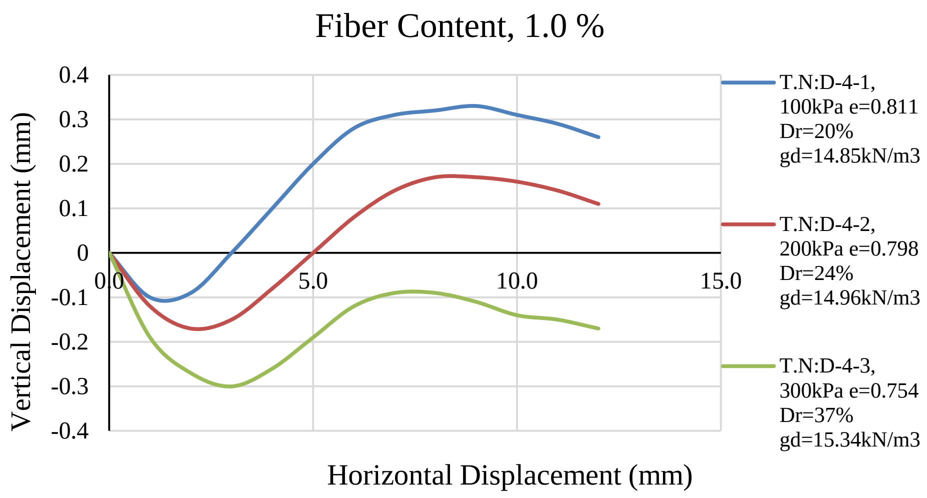

Figure 6.27 : Shear stress-horizontal displacement versus vertical displacements response for reinforced sand samples with fiber content of ρ=1.0% at the normal stresses of σ_n=100, 200 and 300 kPa for at a low relative density around 20%-37%.

The shear stress parameters for the unreinforced and reinforced sands for loose specimens at a low relative density (20%-37%) are shown in Figure 6.28, Figure 6.29, Figure 6.30 and Figure 6.31.

In the unreinforced sample while the apparent cohesion intercept is calculated as zero, the peak shear strength angle is 31^o and the residual shear strength angle is 25^o.

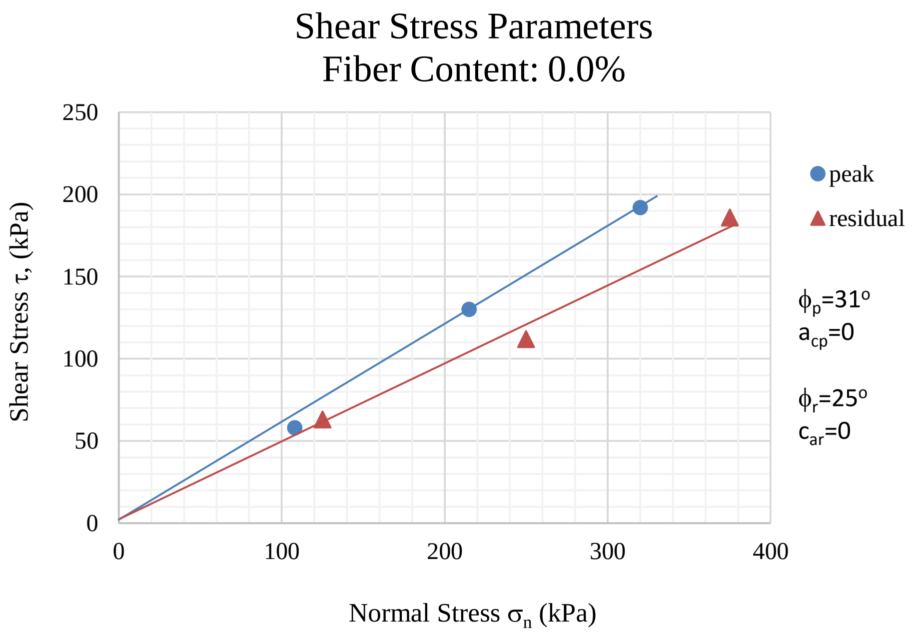

Figure 6.28 : Shear stress-Normal stress graph for unreinforced sand samples at a relative density around 27%-35%.

The shear stress-normal stress graph for the sand reinforced with fiber content of 0.1% is shown in Figure 6.25. While the apparent cohesion is calculated as zero the shear strength angle is calculated 32° and an apparent cohesion value of zero for residual point with a 32° of shear strength angle. The addition of fiber increased both of parameters. On shear stress-normal stress graph for the sand reinforced with the fiber content of 0.5% is shown in Figure 6.30. While the cohesion is calculated as 5 kPa for the peak point, the shear strength angle is calculated 33° and for residual point, apparent cohesion is calculated zero the shear strength is calculated 30°.

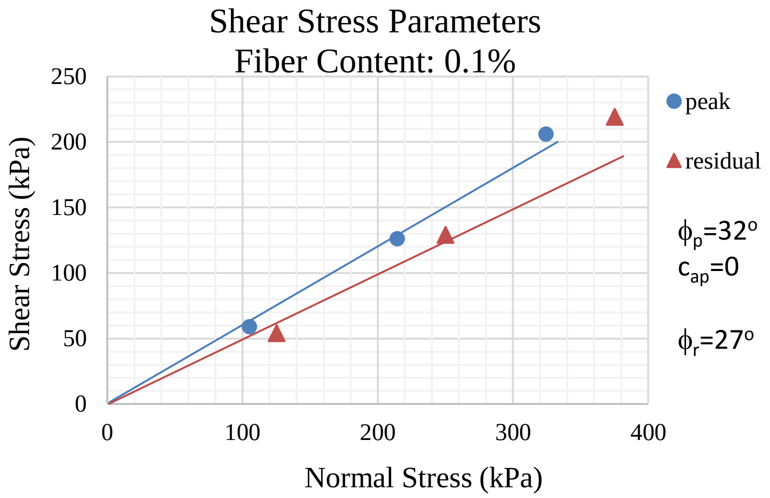

Figure 6.29 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.1% at a relative density around 27%-34%.

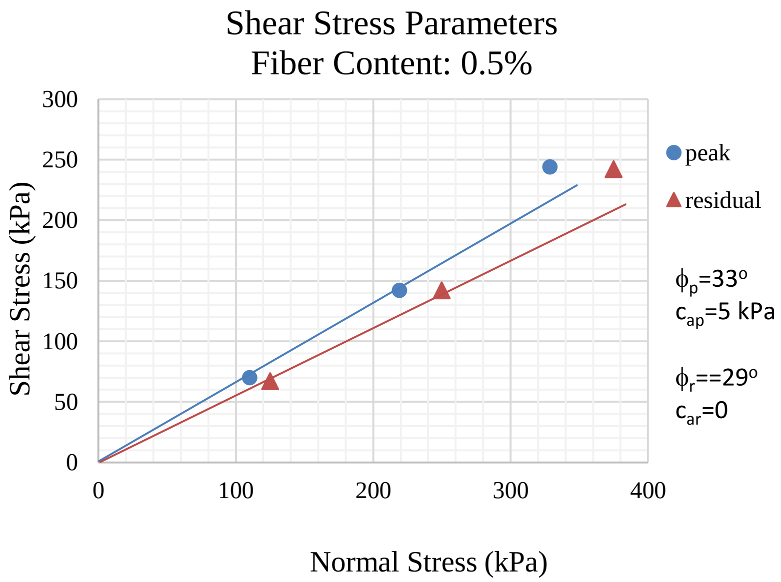

Figure 6.30 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.5% at a relative density around 23%-28%.

The shear stress-normal stress graph for the sand reinforced with fiber content of 1.0% is shown in Figure 6.31. While the apparent cohesion is calculated as zero the shear strength angle is calculated 32^o and an appering cohesion value of 0 kPa for residual point with a 29^o of shear strength angle. The addition of fiber increased both of parameters.

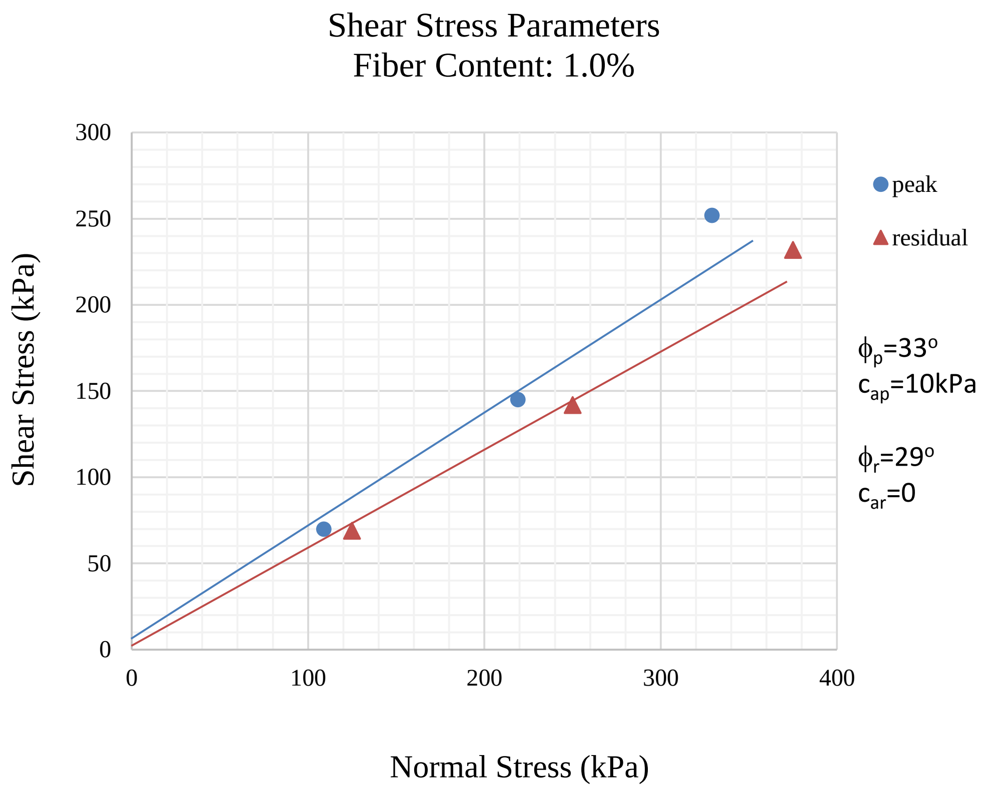

Figure 6.31 : Shear stress-Normal stress graph for sand samples reinforced with fiber content of ρ=0.5% at a relative density around 20%-37%.

The shear stress-horizontal displacement curves and comparison between unreinforced and reinforced sand specimens under the same normal stresses according to fiber content are presented in Figure 6.32, Figure 6.33 and Figure 6.34.

In these tests, all samples (unreinforceds and reinforceds) prepared at the dry condition and the same normal stresses of 100 kPa, 200 kPa, and 300 kPa. According to the Figure 6.32 for samples at the same normal stress of 100 kPa amount of the shear strength are close to each other but with increasing the stresses in tests with 200 kPa and 300 kPa normal stresses there are a increasing in shear strength of soil samples with a 0.5% and 1.0% of fiber contents.

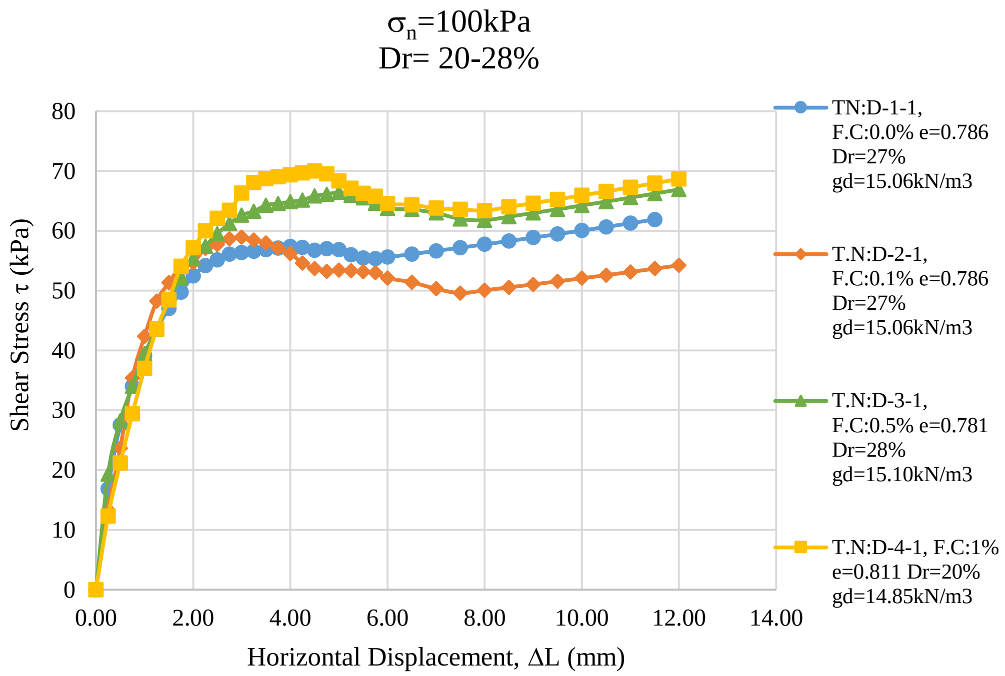

Figure 6.32 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=100 kappa for loose samples.

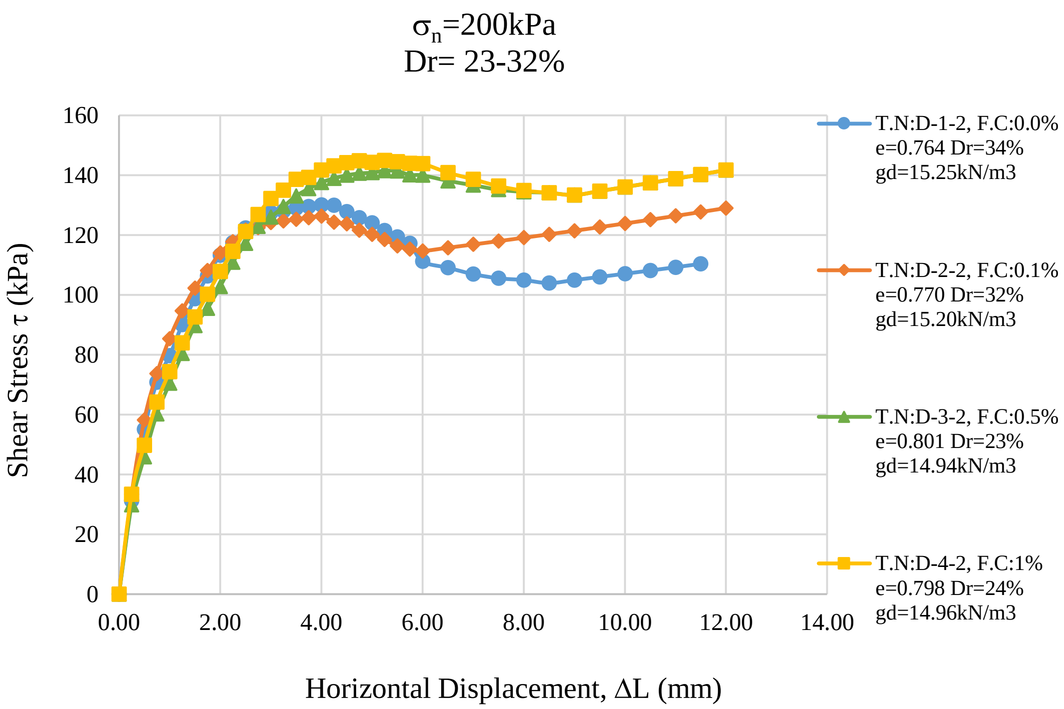

Figure 6.33 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=200 kPa for loose samples.

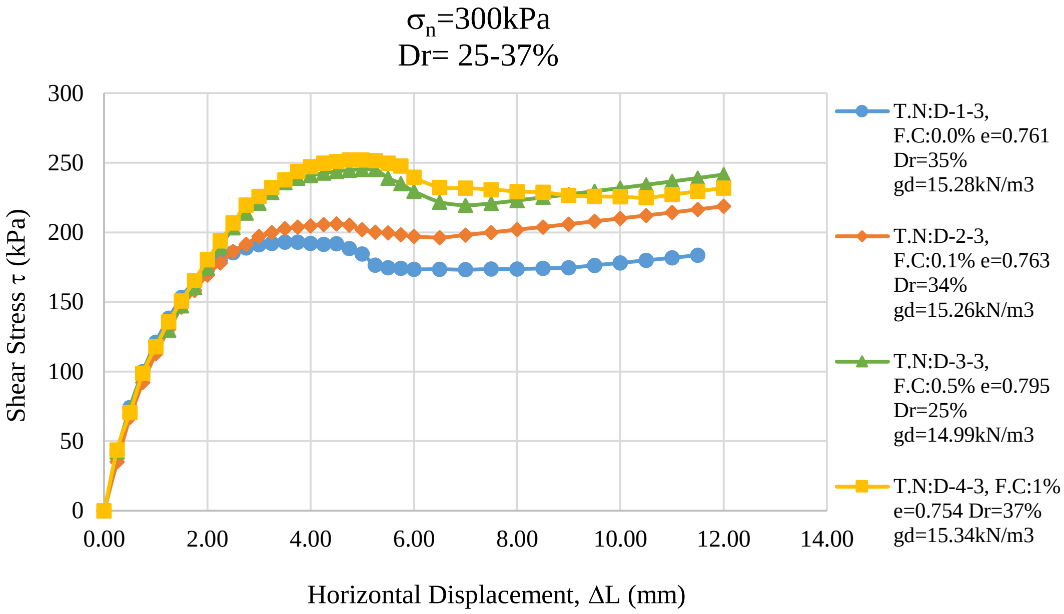

Figure 6.34 : Shear stress-horizontal displacement response for unreinforced and reinforced sand samples at the normal stress of σ_n=200 kPa for loose samples.

The summary of the direct shear test at low relative density are presented in Table 6.3 and Table 6.4. It is resulted that randomly distributed fiber addition affected the shear strength parameters of sand.

Table 6.3 : Direct Shear Test Result at residual point ΔL=12mm for sample at a high relative density around 20%-27%.

Loos Samples, D_r: 20%-37% at Peak Point								
Fiber Content (%)	Test Number	e	D_r (%)	γ_d	σ_p (kPa)	τ_p (kPa)	ϕ_p (o)	c (kPa)
0	D-1-1	0.786	27	15.06	109	57	31	0
	D-1-2	0.764	34	15.25	215	130		
	D-1-3	0.761	35	15.28	320	193		
0.1	D-2-1	0.786	27	15.06	108	62	32	0
	D-2-2	0.77	32	15.2	214	126		
	D-2-3	0.763	34	15.26	324	206		
0.5	D-3-1	0.781	28	15.1	110	70	33	5
	D-3-2	0.801	23	14.94	219	142		
	D-3-3	0.795	25	14.99	329	244		
1	D-4-1	0.811	20	14.85	109	70	33	10
	D-4-2	0.798	24	14.96	219	145		
	D-4-3	0.754	37	15.34	339	245		

Table 6.4 : Direct Shear Test Result at peak point for sample at a low relative density around 23%-37%.

Loos Samples, D_r: 20%-37% at Residual Point								
Fiber Content(%)	Test Number	e	D_r(%)	γ_d	σ_p (kPa)	τ_r (kPa)	ϕ_r (o)	c (kPa)
0	D-1-1	0.786	27	15.06	125	63	25	0
	D-1-2	0.764	34	15.25	250	112		
	D-1-3	0.761	35	15.28	375	186		
0.1	D-2-1	0.786	27	15.06	125	62	27	0
	D-2-2	0.77	32	15.2	250	129		
	D-2-3	0.763	34	15.26	375	219		
0.5	D-3-1	0.781	28	15.1	125	67	29	0
	D-3-2	0.801	23	14.94	250	142		
	D-3-3	0.795	25	14.99	375	242		
1	D-4-1	0.811	20	14.85	125	69	29	5
	D-4-2	0.798	24	14.96	250	142		
	D-4-3	0.754	37	15.34	375	232		

In this test, Figure 6.35 and Figure 6.36 show the apparent cohesion and shear strength values calculated for unreinforced and reinforced sand specimens by performing Direct Shear Test on samples. The peak and residual shear strength angle (ϕ_p) and (ϕ_r) are calculated and are presented in Figure 6.37 and Figure 6.38.

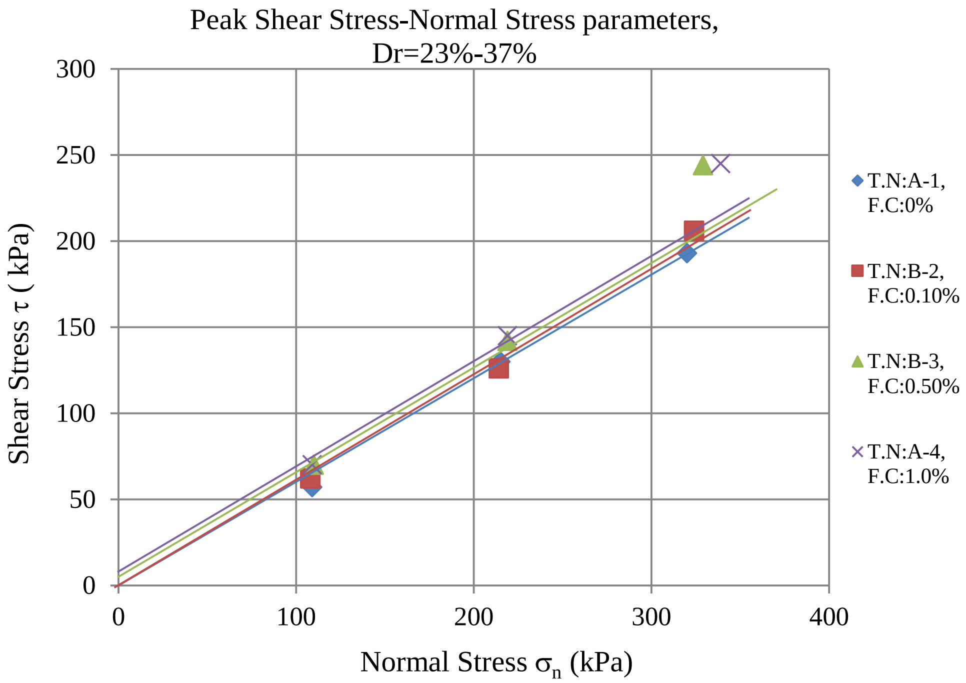

Figure 6.35 : Apparent cohesion and peak shear strength values according to fiber addition.

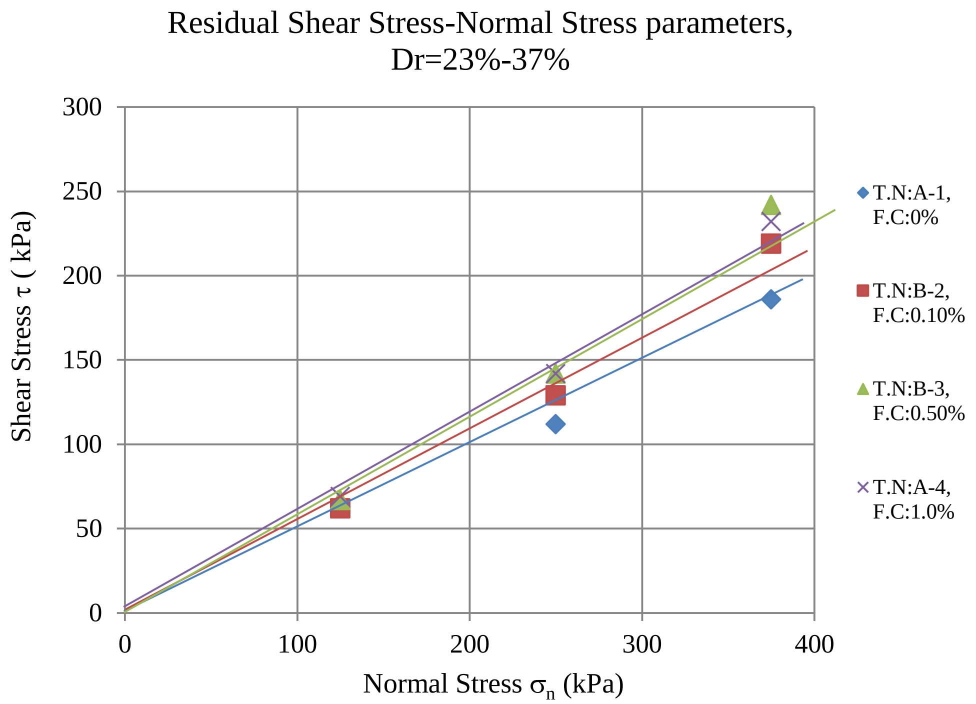

Figure 6.36 : Apparent cohesion and residual shear strength values according to fiber addition.

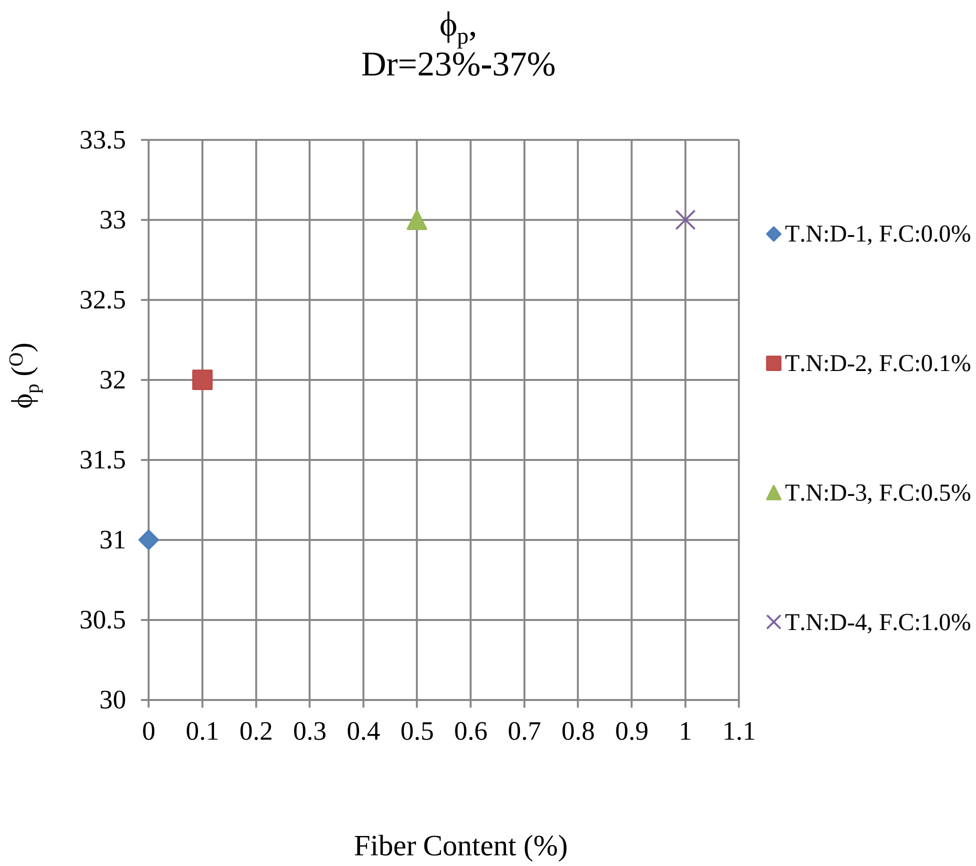

Figure 6.37 : The peak shear strength angle.

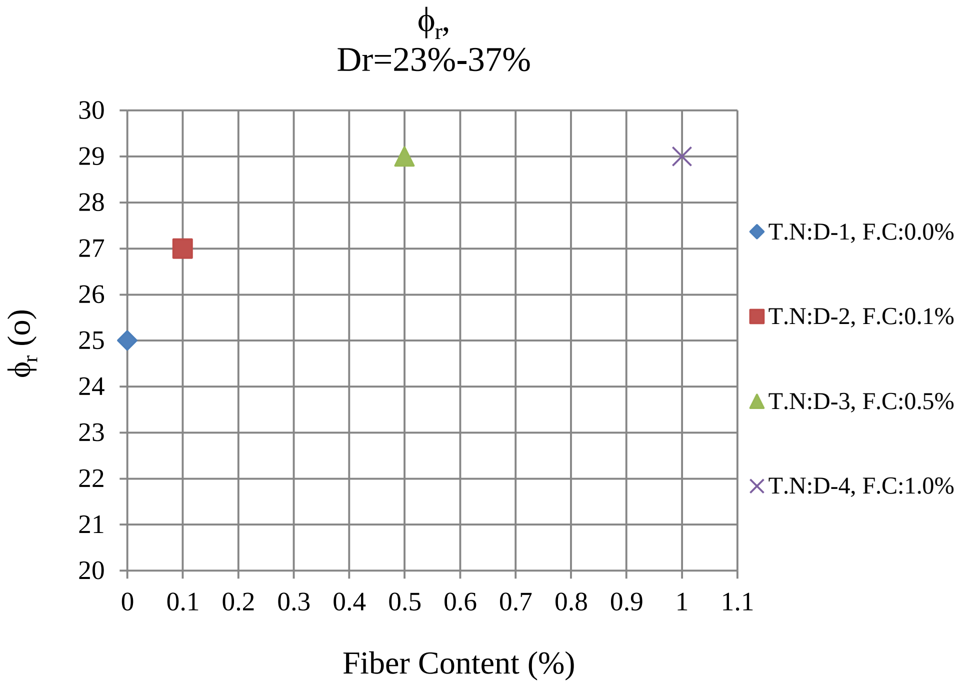

Figure 6.38 : Shear stress-Normal stress graph for unreinforced and reinforced samples according to residual value at 20%-35% relative densities.

6.1.3 The comparison between two different samples with high and low relative densities.

The results of two different direct shear tests are compared in figure 6.39 to figure 6.40. It is resulted that the randomly distributed fiber addition affected the shear strength parameters of sand. The peak shear strength angles (ϕ_p) for direct shear test in dense condition at a range of relative densities of around 60%-74% was 36^o for normal stress of unreinforced samples, 37^o for 0.1% fiber content, 37^o for 0.5% fiber content and 35^o for 1.0% fiber content so as a result the shear strength angle increased. For direct shear test in loose condition at a range of relative densities of 20% - 35% it were 31^o for normal stress of unreinforced samples, 32^o for 0.1% fiber content, 33^o for 0.5% fiber content and 33^o for 1.0% as the fiber content increased from 0.0% to 0.5. Applying the tests in dry condition, the apparent cohesion value is zero for test with a dense condition for unreinforced sample, zero for 0.1% fiber content, 20 kPa for sample with an amount of 0.5% fiber content and 27 kPa. When value of apparent cohesion (c_a) value is zero for both unreinforced and 0.1% fibrillated sample this value is 5kPa for sample with a fiber content of 0.5% and 1.0% fiber content for unreinforced and reinforced sand samples. The shear stress-normal stress graphs are presented in Appendix.

According to experimental results, it can be concluded that the shear strength values under the same normal stresses increases with increasing relative density. These results and comparison between them are shown in Figure 6.39, Figure 6.40, and Figure 6.41 for unreinforced samples under the 100 kPa normal stress. Moreover, Figure 6.42, Figure 6.43, Figure 6.44 for samples with 0.1% fiber content.

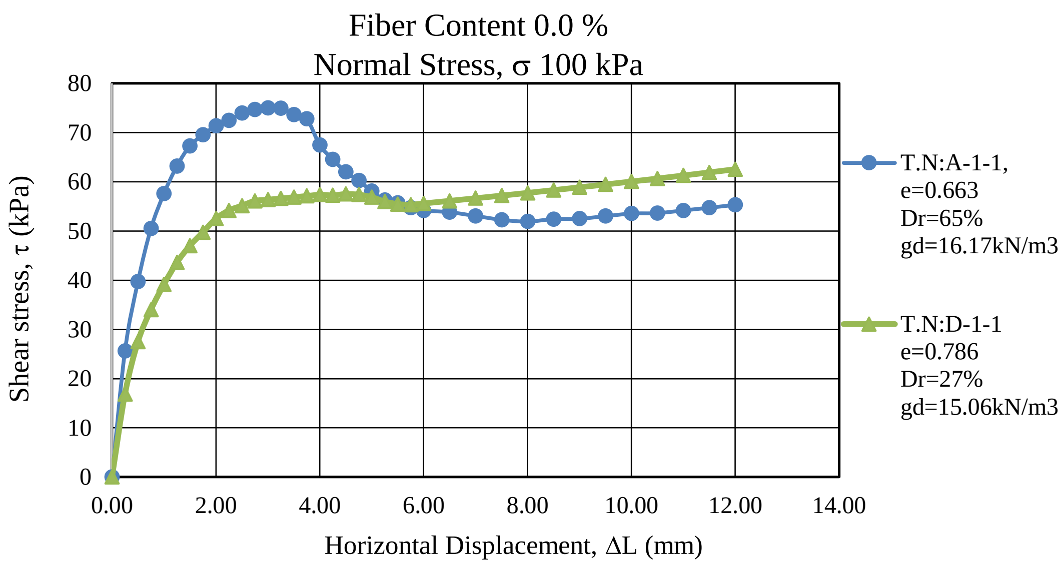

Figure 6.39 : Comparison Shear stress-horizontal displacement response for unreinforced sand samples at the normal stress of σ_n=100 kPa according to high and low relative density.

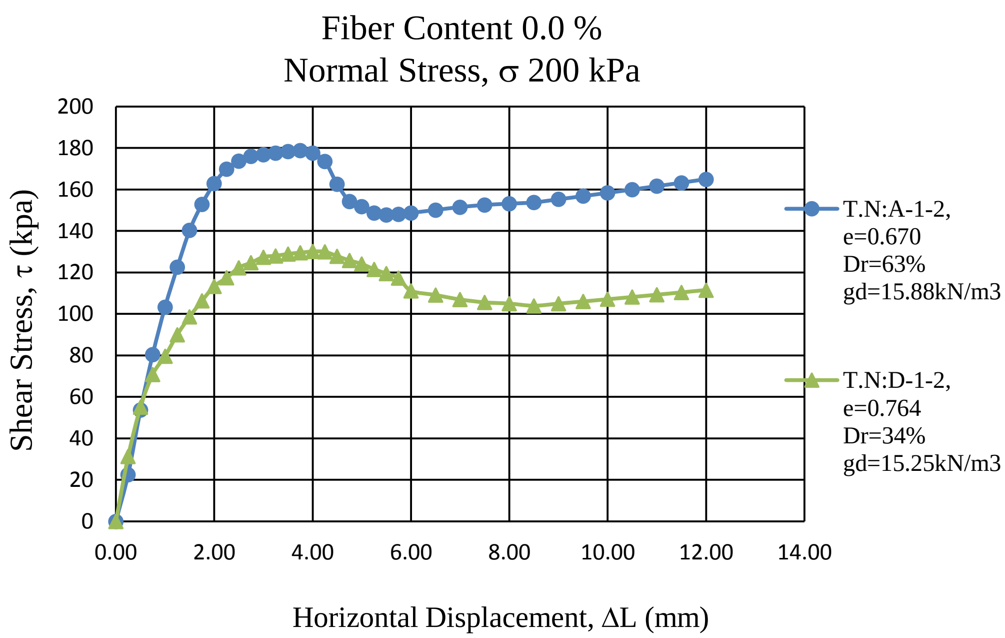

Figure 6.40 : Comparison Shear stress-horizontal displacement response for unreinforced sand samples at the normal stress of σ_n=200 kPa according to high and low relative density.

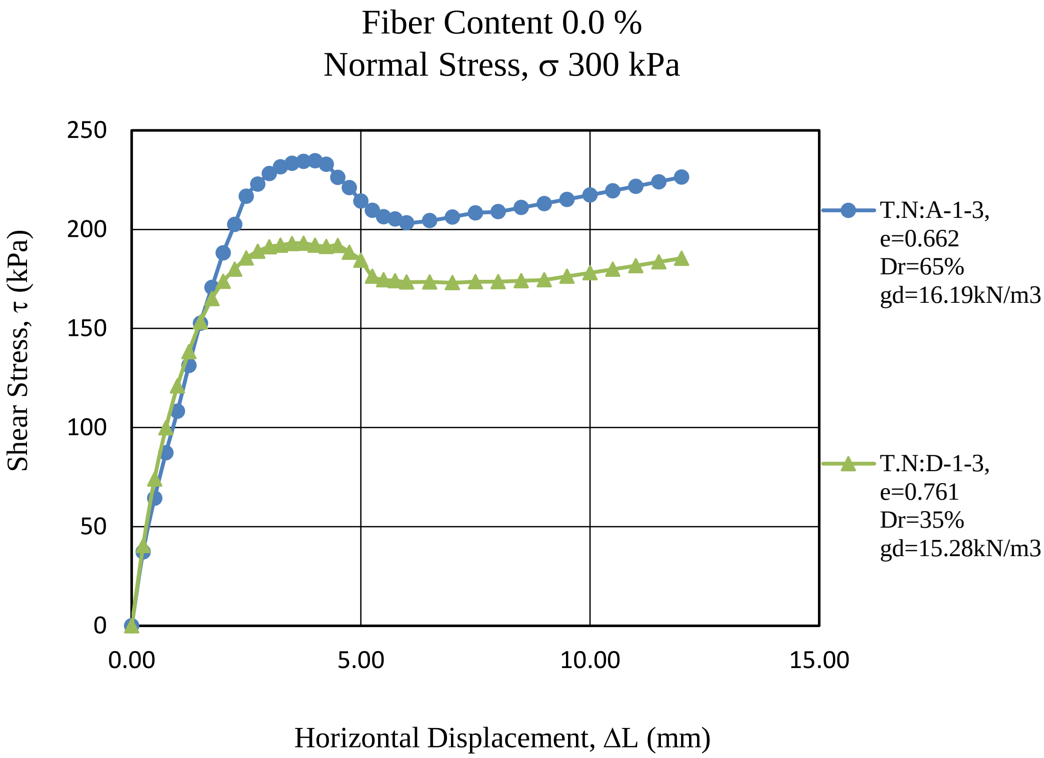

Figure 6.41 : Comparison Shear stress-horizontal displacement response for unreinforced sand samples at the normal stress of σ_n=300 kPa according to high and low relative density.

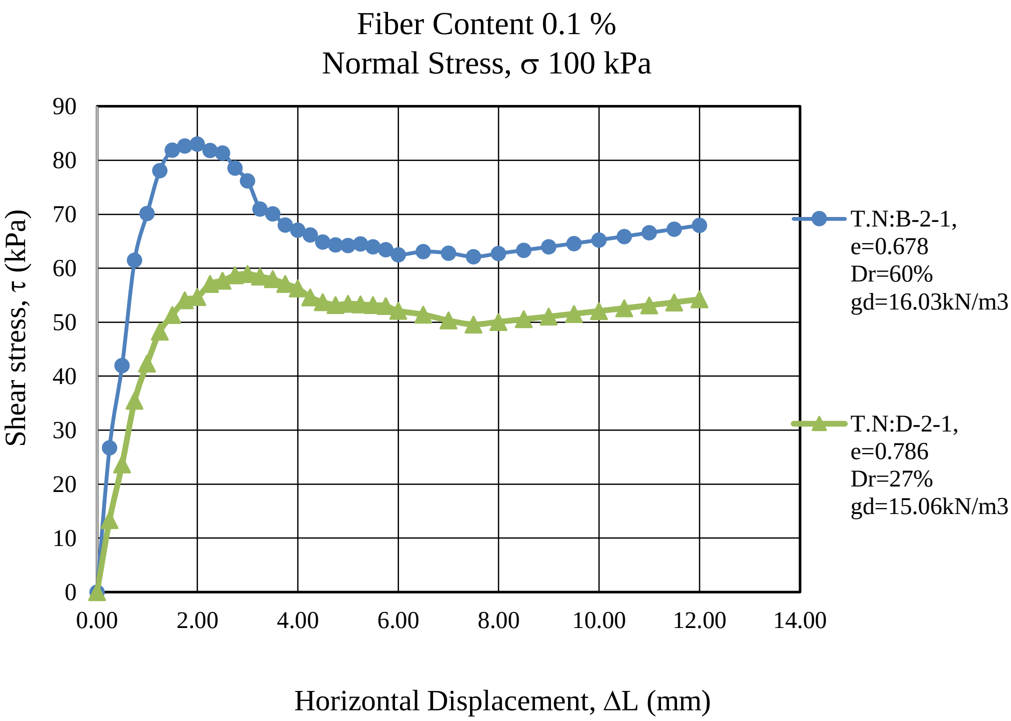

Figure 6.42 : Shear stress-horizontal displacement response for reinforced samples with fiber content of ρ=0.1% fibrillated sand samples at the normal stress of σ_n=100 kPa according to high and low relative density.

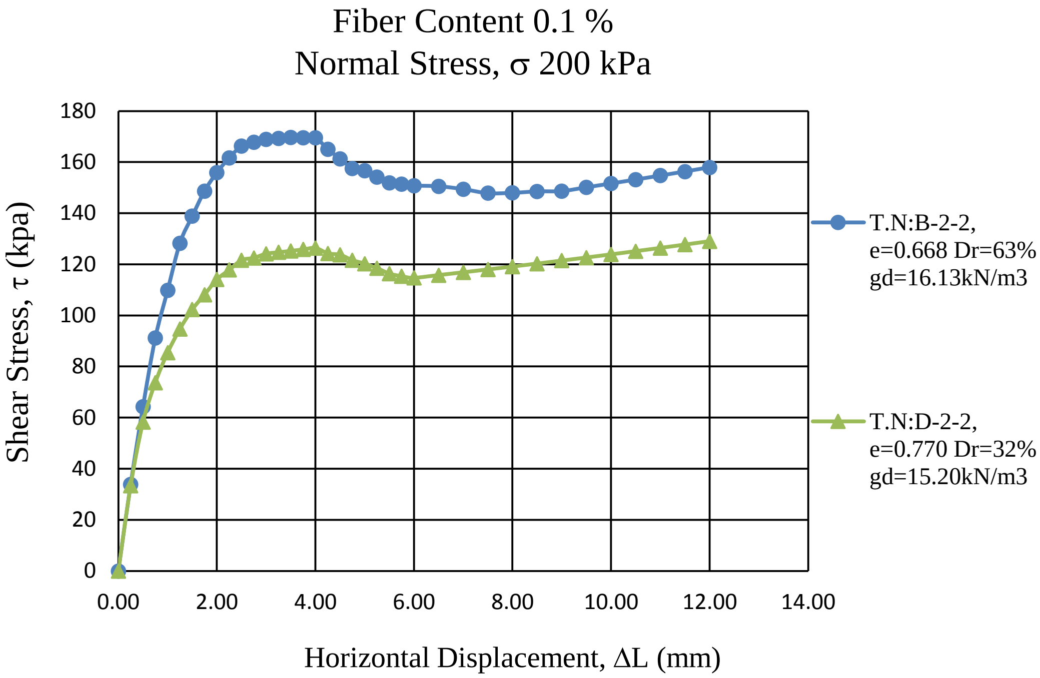

Figure 6.43 : Comparison Shear stress-horizontal displacement response for reinforced samples with fiber content of ρ=0.1% fibrillated sand samples at the normal stress of σ_n=200 kPa according to high and low relative densities.

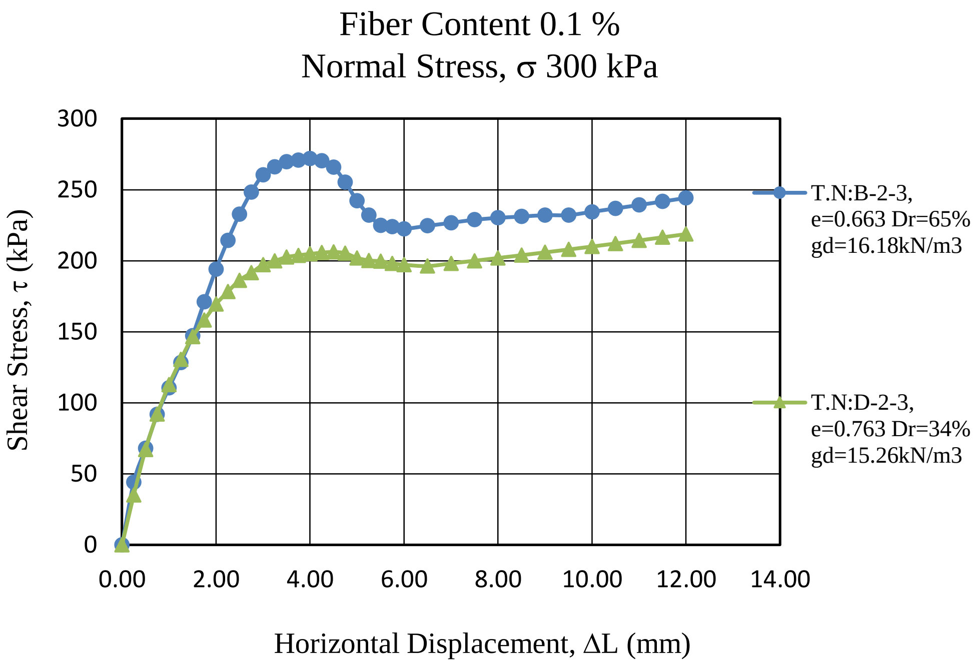

Figure 6.44 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of ρ=0.1% fibrillated sand samples at the normal stress of σ_n=300 kPa according to high and low relative densities.

The comparison between the two different test results of dense samples and loose samples are shown in Figure 6.45, Figure 6.46, for samples with a 0.5% fiber content under the same normal stress of 200 kPa.

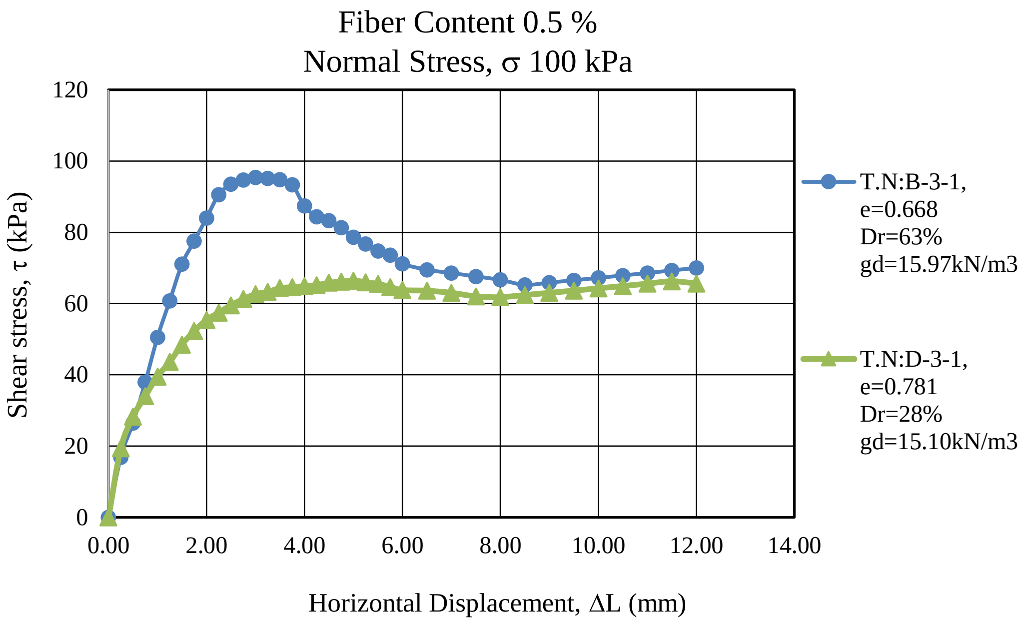

Figure 6.45 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of ρ=0.5% fibrillated sand samples at the normal stress of σ_n=100 kPa according to high and low relative densities.

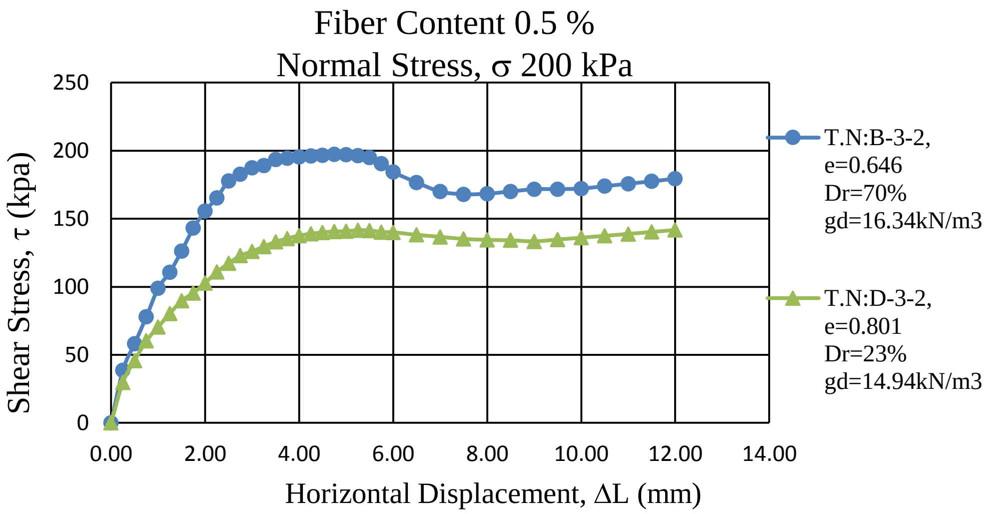

Figure 6.46 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of r=0.1% fibrillated sand samples at the normal stress of σ_n=200 kPa according to high and low relative densities.

The comparison between two different tests results with a high relative density and low relative density are shown in Figure 6.47, Figure 6.48 and Figure 4.49 for samples with a 1.0% fiber content under the same normal stress of 100 kPa, 200 kPa and 300 kPa. According to the results, when the maximum shear strength is 100 kPa for samples at a high relative density the maximum shear strength is around 100 kPa and for sample at a low relative density is around 70 kPa. These values are also 180 kPa and 145 kPa for samples under a normal stress of 200 kPa and 288 kPa and 245 kpa for respectively for samples at high and low relative densities under the 300 kPa normal stress.

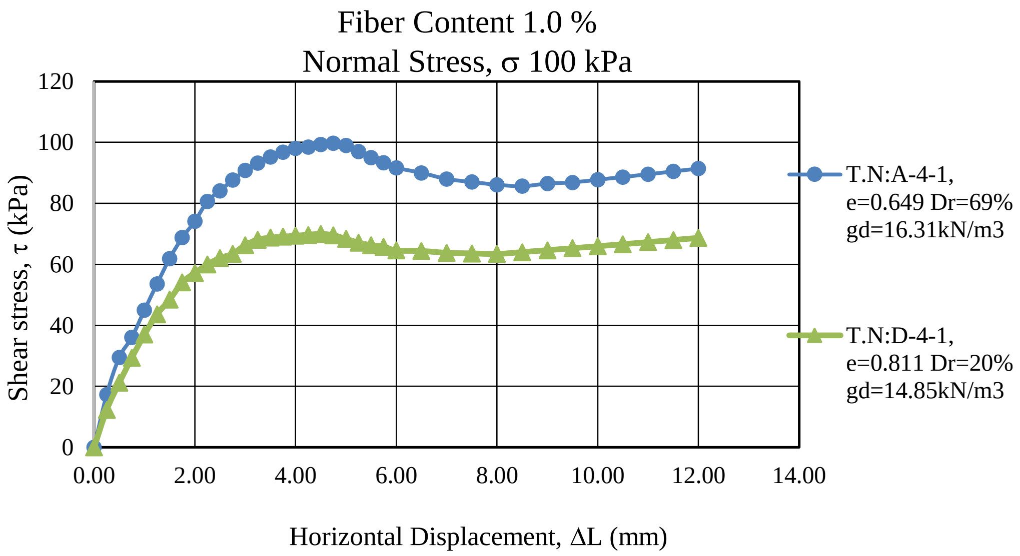

Figure 6.47 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of r=1.0% fibrillated sand samples at the normal stress of σ_n=100 kPa according to high and low relative densities.

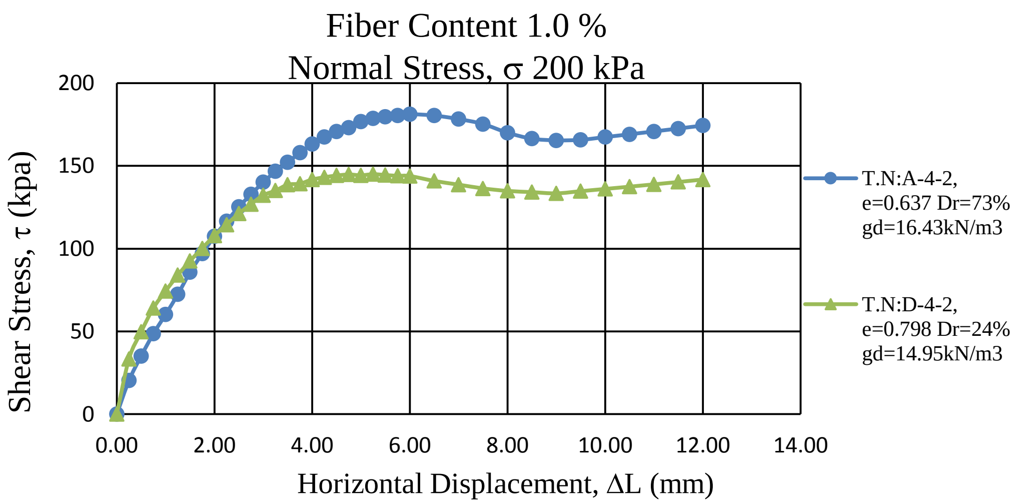

Figure 6.48 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of r=1.0% fibrillated sand samples at the normal stress of σ_n=200 kPa according to high and low relative densities.

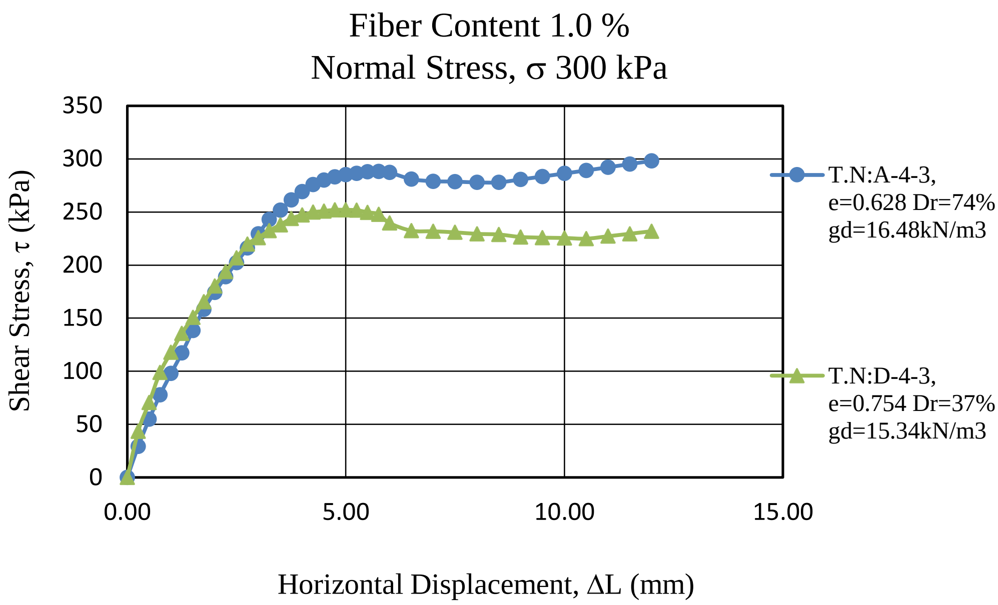

Figure 6.49 : Comparison shear stress-horizontal displacement response for reinforced samples with fiber content of r=1.0% fibrillated sand samples at the normal stress of σ_n=300 kPa according to high and low relative densities.

6.1.4 The effect of fiber ratio on angle of friction.

In the direct shear test , the peak shear strength angle (ϕ_p) and apparent cohesion (c_a) parameters are calculated by leaner regression analyses with correlation coefficients that are approximately equal to unity (R2=0.97-0.99). The peak shear strength angle (ϕ_p) calculated from direct shear tests at high and low relative densities are presented in table 6.5. It is presented that the shear strength angle increases due to fiber addition in both testes and in samples at a high relative density amount of these parameters are higher than test at a low relative density. According to the Table 6.5, there is a variation on shear strength angle in samples at high relative density but also there is an increasing in samples at loos condition.

Table 6.5 : Comparison angle of friction between two different low and high relative density.

Fiber Content (%)	ϕ_p			
	Test Number	D_r=60%-75%	Test Number	Dr=20%-37%
0	A-1	36°	D-1	31°
0.1	A-2	37°	D-2	32°
0.5	A-3	37°	D-3	33°
1.0	A-4	35°	D-4	33°

6.1.5 The effect of fiber ratio on shear strength

The shear stress-fiber content curves of unreinforced and reinforced sand specimens are presented in Figure 6.50 for samples at a high relative density and Figure 6.51 for samples at a low relative density. In the reinforced specimens, the recorded shear stress values increase of 100 kPa, 200 kPa and 300 kPa by fiber addition. These increments are shown by percent of increase in below figures. According to the Figure 6.50 and Figure 6.51 there are a significant improvement in shear strength values from unreinforced samples to samples with 1.0% fiber content.

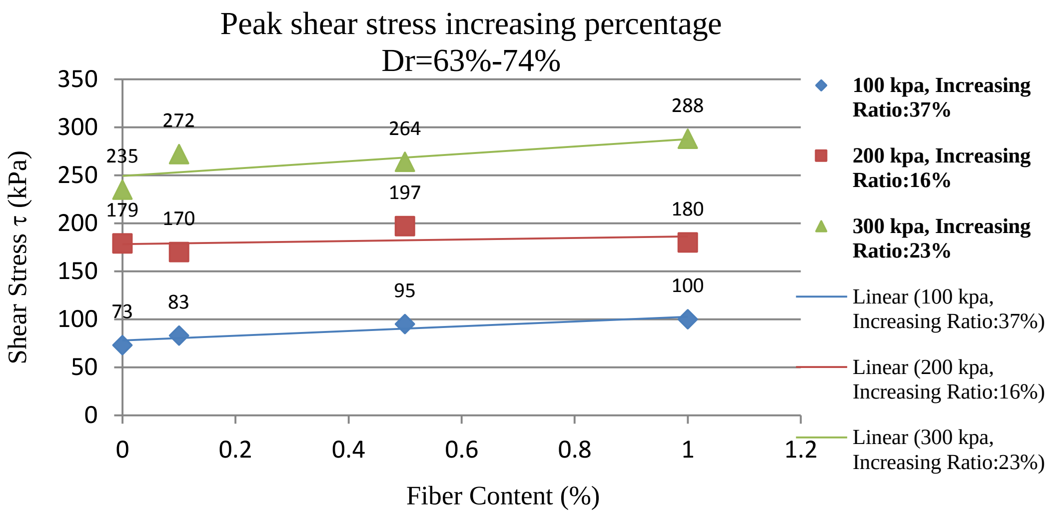

Figure 6.50 : Effect of fibers on shear strength of unreinforced and reinforced sand sample obtained from direct shear tests at a high relative density (60%-70%).

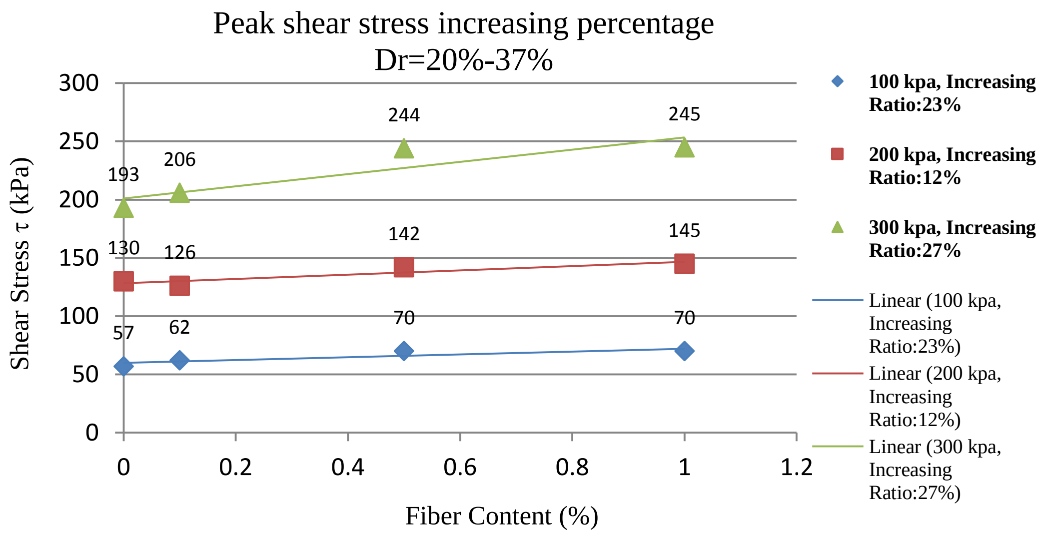

Figure 6.51 : Effect of fibers on shear strength of unreinforced and reinforced sand sample obtained from direct shear tests at a low relative density (20%-37%).

On the other hand, the shear strength calculated from Direct Shear Test increased due to fiber addition. The peak and residual shear strength angle parameters for both low and high relative densities samples are calculated from Direct Shear Tests that are performed on unsaturated specimens.

6.2 Permeability Test Results

In order to determine the effect of fibers on permeability of sand samples, Constant Head test was employed on unreinforced and reinforced specimens at relative density around 55%. The samples are prepared at the different contents of fiber as 0.0%, 0.1%, 0.5% and 1.0% of dry weight of sand. This test were repeated for three times and results of this test is shown in Figure 6.52. This figure shows that there is a decrease in permeability due to fiber addition from unreinforced sand until 0.5% fiber content samples but it can see after 0.5% fiber content there is a decreasing in permeability of these soils. According to Figure 6.25, permeability of samples first reached to the peak at sample with a 0.1% of fiber content and then it starts to decrease in other samples.

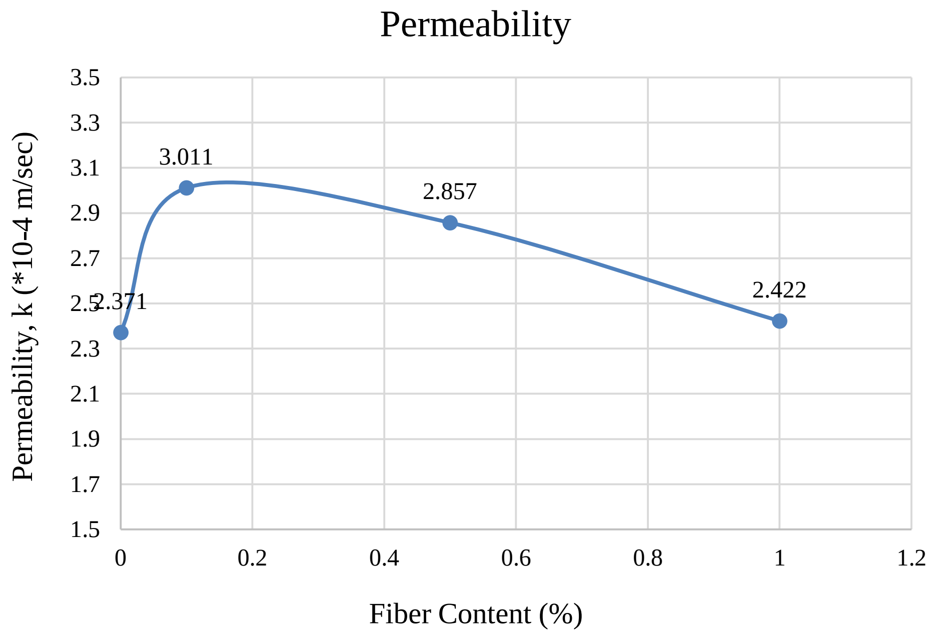

Figure 6.52 : Permeability of unreinforced and reinforced sand according to fiber.

In the other part of permeability test the effect of fiber on amount of water volume into the soil sample was studied. As same as the constant head the samples are prepared at the different contents of fiber as 0.0%, 0.1%, 0.5% and 1.0% of dry weight of sand, a vacuum pressure of 0.01MPa and 1.0 bar CO_2 it was applied in two different relative density of 60% and 70%. The results of these tests are shown in Figure 6.31. The water volume ratio of reinforced samples to the unreinforced sample was compared. According to the Figure 6.53, it can conclude that there is a decreasing in Vf.r/Vpur according to the fiber content increasing.

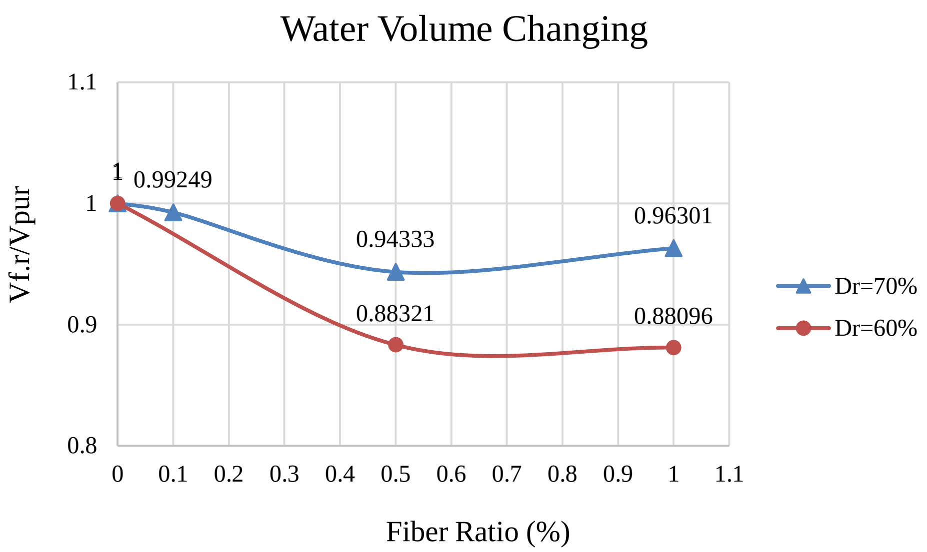

Figure 6.53 : Ratio of water volume of fibrillated samples to pure samples.

7. CONCLUSIONS AND RECOMMENDATIONS

In Geotechnical Engineering, there are some methods for improvement engineering properties of soils. In fact, soil improvement with addition of substances is a common ground treatment. Usage of synthetics fiber in geotechnical engineering is feasible in road construction, retaining walls, slope stability, railway embankments and earth-quake engineering. Previous researches on randomly distributed reinforced show that fiber inclusion improve the static and cyclic response depending on some factors such as soil type, particle size, fiber type, aspect ratio and size and amount of fiber.

In scope of this thesis, the effects of VHP (Virgin Homopolymer Polypropylene) fiber on the static behavior of a kind of sand are searched by performing laboratory experiments. The fibers are mixed into poorly graded sand. The fibers contents are determined as the percentage of the dry weight of sand. The samples are prepared at relative densities of around 50%-60% for a high relative density and a 20%-35% for a low relative density to compare effect of fiber and relative density on shear strength parameters of sand.

The materials of fibers used in this research is Virgin Homopolymer Polypropylene as a short name of VHP, which is widely, used addition in the laboratory testing of soil reinforcement. In the experimental part of the thesis, first Constant Head test is used to determine permeability parameters of fibrillated sand samples and effect of fibers on permeability and water volume changing. According the results of in this test as the fiber content increased, permeability decreased.

In the second part of the experimental study, direct shear tests are performed. The samples are prepared at the relative density around 55% and fiber contents of 0.1%, 0.5%, and 1.0% by dry weight of sand. The direct shear tests is performed with a strain rate of 0.12 mm/min. The shear stress-horizontal displacement curves for the normal stresses of 100 kPa, 200 kPa and 300 kPa showed that while the unreinforced samples showed the strain softening behavior, the fiber reinforced samples showed strain hardening behavior. The shear strength parameters are calculated for all samples. As the shear strength angle decreases due to increasing fiber content, the apparent cohesion value increased in a considerable amount. It can be observed that while the apparent cohesion value increases due to fiber addition in this testing method, the change in the shear strength angle showed an increasing in those angles.

In conclusion, the randomly distributed VHP fiber inclusion improved the strength of Akpınar sand considerably. The results obtained from the experimental study are also consistent with the previous researches that are conducted on randomly distributed polypropylene fiber reinforced sands. The method of randomly distributed fiber inclusions can be preferred for landfills, pavements and slope stabilization. The experimental study can also be advanced with an analytical model or additional laboratory tests.

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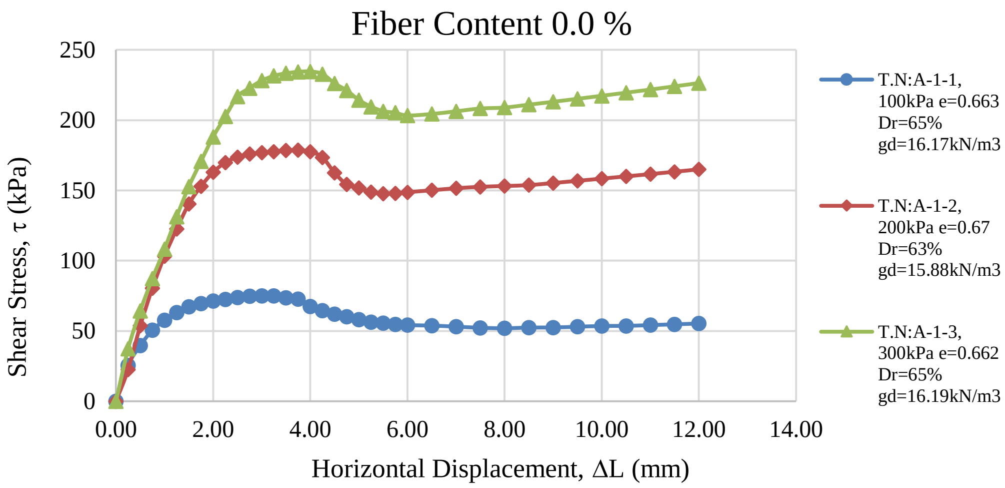
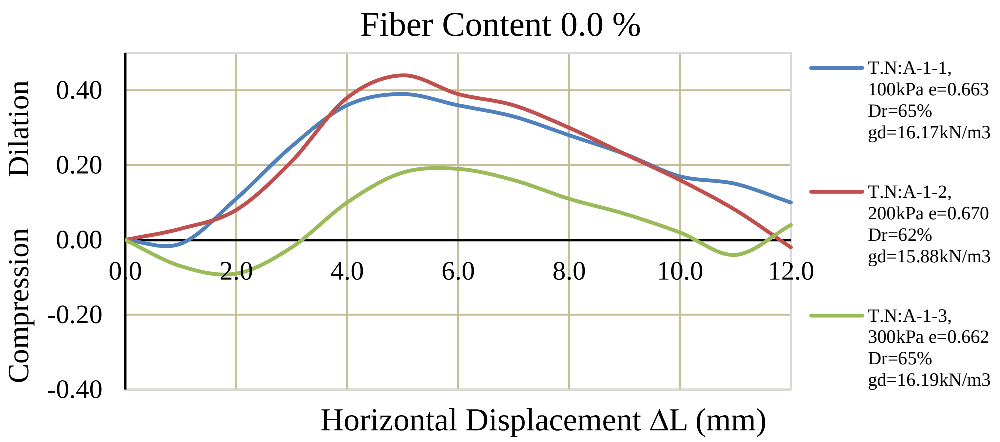
APPENDICES

APPENDIX A: Direct Shear Tets Results

APPENDIX B: Permeability Test Results

APPENDIX A





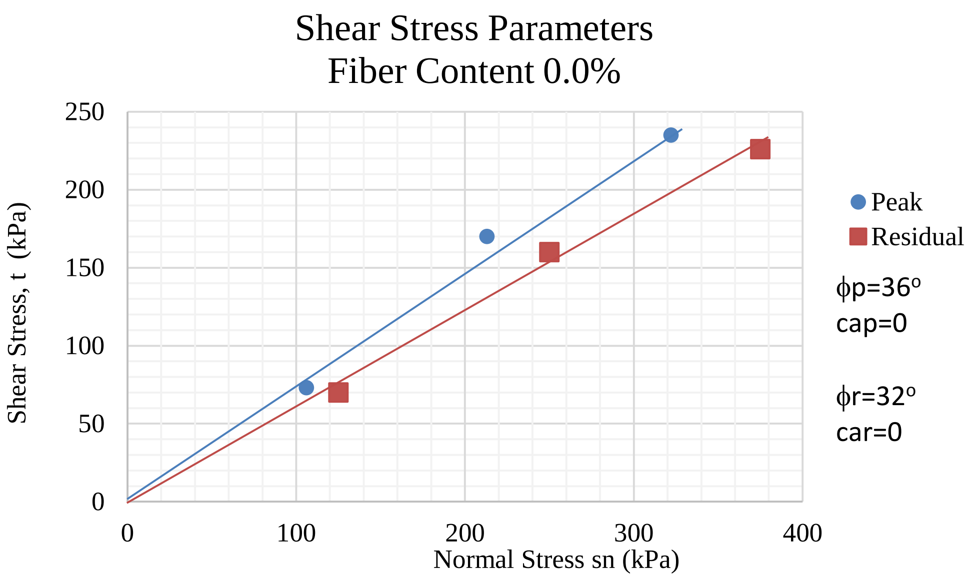


Figure A.1 : Tets Results for Tets No A-1.

Table A.1 : Direct Shear Test Results- A-1-1

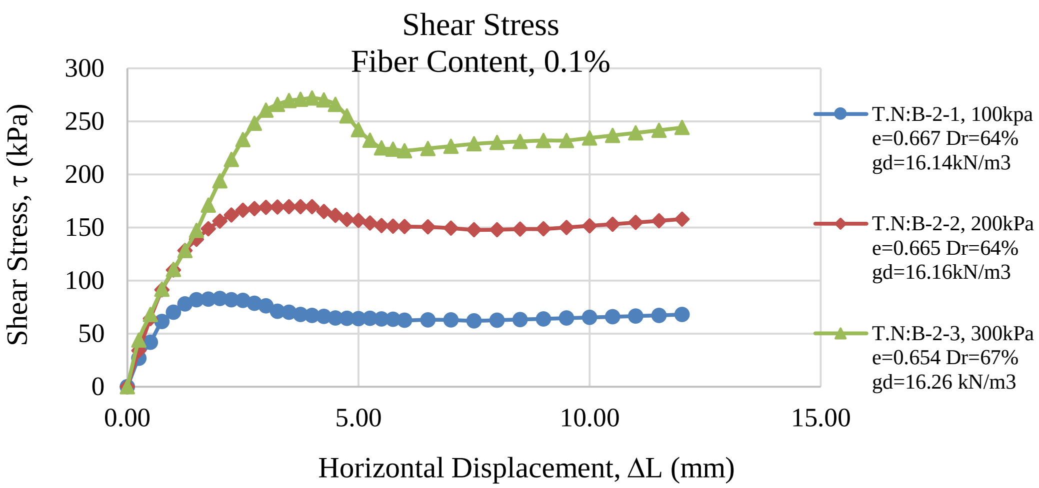
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σ_n (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	1.0	36.00	0.00	0.00
25	0.25	68.0			1.0	35.85	9.18	0.26
50	0.50	105.0			1.0	35.70	14.18	0.40
75	0.75	133.0			1.0	35.55	17.96	0.51
100	1.00	151.0	-44.0	-0.01	1.0	35.40	20.39	0.58
125	1.25	165.0			1.0	35.25	22.28	0.63
150	1.50	175.0			1.0	35.10	23.63	0.67
175	1.75	180.0			1.0	34.95	24.30	0.70
200	2.00	184.0	-32.0	0.11	1.0	34.80	24.84	0.71
225	2.25	186.0			1.0	34.65	25.11	0.72
250	2.50	189.0			1.0	34.50	25.52	0.74
275	2.75	190.0			1.0	34.35	25.65	0.75
300	3.00	190.0	-18.0	0.25	1.0	34.20	25.65	0.75
325	3.25	189.0			1.0	34.05	25.52	0.75
350	3.50	185.0			1.0	33.90	24.98	0.74
375	3.75	182.0			1.0	33.75	24.57	0.73
400	4.00	168.0	-7.0	0.36	1.0	33.60	22.68	0.67
425	4.25	160.0			1.0	33.45	21.60	0.65
450	4.50	153.0			1.0	33.30	20.66	0.62
475	4.75	148.0			1.0	33.15	19.98	0.60
500	5.00	142.0	-4.0	0.39	1.0	33.00	19.17	0.58
525	5.25	137.0			1.0	32.85	18.50	0.56
550	5.50	135.0			1.0	32.70	18.23	0.56
575	5.75	132.0			1.0	32.55	17.82	0.55
600	6.00	130.0	-4.0	0.39	1.0	32.40	17.55	0.54
650	6.50	128.0			1.0	32.10	17.28	0.54
700	7.00	125.0	-12.0	0.31	1.0	31.80	16.88	0.53
750	7.50	122.0			1.0	31.50	16.47	0.52
800	8.00	120.0	-18.0	0.25	1.0	31.20	16.20	0.52
850	8.50	120.0			1.0	30.90	16.20	0.52
900	9.00	119.0	-24.0	0.19	1.0	30.60	16.07	0.52
950	9.50	119.0			1.0	30.30	16.07	0.53
1000	10.00	119.0	-30.0	0.13	1.0	30.00	16.07	0.54
1050	10.50	118.0			1.0	29.70	15.93	0.54
1100	11.00	118.0	-37.0	0.06	1.0	29.40	15.93	0.54
1150	11.50	118.0			1.0	29.10	15.93	0.55
1200	12.00	118.0	-46.0	-0.03	1.0	28.80	15.93	0.55

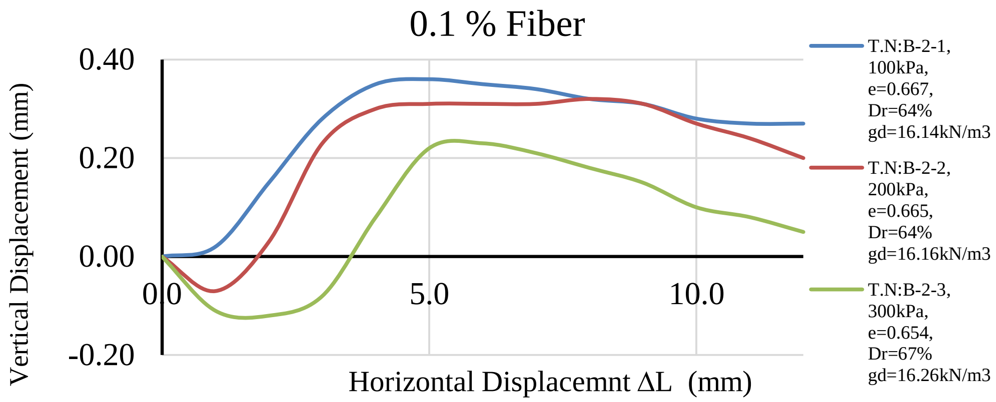
Table A.2 : Direct Shear Test Results- A-1-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	2.0	36.00	0.00	0.00
25	0.25	60			2.0	35.85	8.10	0.23
50	0.50	142.0			2.0	35.70	19.17	0.54
75	0.75	212.0			2.0	35.55	28.62	0.81
100	1.00	271.0	-41.0	0.03	2.0	35.40	36.59	1.03
125	1.25	320.0			2.0	35.25	43.20	1.23
150	1.50	365.0			2.0	35.10	49.28	1.40
175	1.75	396.0			2.0	34.95	53.46	1.53
200	2.00	420.0	-36.0	0.08	2.0	34.80	56.70	1.63
225	2.25	436.0			2.0	34.65	58.86	1.70
250	2.50	444.0			2.0	34.50	59.94	1.74
275	2.75	448.0			2.0	34.35	60.48	1.76
300	3.00	448.0	-23.0	0.21	2.0	34.20	60.48	1.77
325	3.25	448.0			2.0	34.05	60.48	1.78
350	3.50	448.0			2.0	33.90	60.48	1.78
375	3.75	447.0			2.0	33.75	60.35	1.79
400	4.00	442.0	-6.0	0.38	2.0	33.60	59.67	1.78
425	4.25	430.0			2.0	33.45	58.05	1.74
450	4.50	401.0			2.0	33.30	54.14	1.63
475	4.75	379.0			2.0	33.15	51.17	1.54
500	5.00	371.0	0.0	0.44	2.0	33.00	50.09	1.52
525	5.25	362.0			2.0	32.85	48.87	1.49
550	5.50	358.0			2.0	32.70	48.33	1.48
575	5.75	357.0			2.0	32.55	48.20	1.48
600	6.00	357.0	-5.0	0.39	2.0	32.40	48.20	1.49
650	6.50	357.0			2.0	32.10	48.20	1.50
700	7.00	357.0	-8.0	0.36	2.0	31.80	48.20	1.52
750	7.50	356.0			2.0	31.50	48.06	1.53
800	8.00	354.0	-14.0	0.3	2.0	31.20	47.79	1.53
850	8.50	352.0			2.0	30.90	47.52	1.54
900	9.00	352.0	-21.0	0.23	2.0	30.60	47.52	1.55
950	9.50	352.0			2.0	30.30	47.52	1.57
1000	10.00	352.0	-28.0	0.16	2.0	30.00	47.52	1.58
1050	10.50	352.0			2.0	29.70	47.52	1.60
1100	11.00	352.0	-36.0	0.08	2.0	29.40	47.52	1.62
1150	11.50	352.0			2.0	29.10	47.52	1.63
1200	12.00	352.0	-46.0	-0.02	2.0	28.80	47.52	1.65

Table A.3 : Direct Shear Test Results- A-1-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σ_n (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	-46	-0.46	3.0	36.00	0.00	0.00
25	0.25	99			3.0	35.85	13.37	0.37
50	0.50	170.0			3.0	35.70	22.95	0.64
75	0.75	230.0			3.0	35.55	31.05	0.87
100	1.00	284.0	-53	-0.53	3.0	35.40	38.34	1.08
125	1.25	343.0			3.0	35.25	46.31	1.31
150	1.50	397.0			3.0	35.10	53.60	1.53
175	1.75	442.0			3.0	34.95	59.67	1.71
200	2.00	485.0	-55	-0.55	3.0	34.80	65.48	1.88
225	2.25	520.0			3.0	34.65	70.20	2.03
250	2.50	554.0			3.0	34.50	74.79	2.17
275	2.75	567.0			3.0	34.35	76.55	2.23
300	3.00	578.0	-48	-0.48	3.0	34.20	78.03	2.28
325	3.25	584.0			3.0	34.05	78.84	2.32
350	3.50	586.0			3.0	33.90	79.11	2.33
375	3.75	586.0			3.0	33.75	79.11	2.34
400	4.00	584.0	-36	-0.36	3.0	33.60	78.84	2.35
425	4.25	577.0			3.0	33.45	77.90	2.33
450	4.50	558.0			3.0	33.30	75.33	2.26
475	4.75	543.0			3.0	33.15	73.31	2.21
500	5.00	524.0	-28	-0.28	3.0	33.00	70.74	2.14
525	5.25	510.0			3.0	32.85	68.85	2.10
550	5.50	500.0			3.0	32.70	67.50	2.06
575	5.75	495.0			3.0	32.55	66.83	2.05
600	6.00	488.0	-27	-0.27	3.0	32.40	65.88	2.03
650	6.50	486.0			3.0	32.10	65.61	2.04
700	7.00	486.0	-30	-0.3	3.0	31.80	65.61	2.06
750	7.50	486.0			3.0	31.50	65.61	2.08
800	8.00	483.0	-35	-0.35	3.0	31.20	65.21	2.09
850	8.50	483.0			3.0	30.90	65.21	2.11
900	9.00	483.0	-39	-0.39	3.0	30.60	65.21	2.13
950	9.50	483.0			3.0	30.30	65.21	2.15
1000	10.00	483.0	-44	-0.44	3.0	30.00	65.21	2.17
1050	10.50	483.0			3.0	29.70	65.21	2.20
1100	11.00	483.0	-50	-0.5	3.0	29.40	65.21	2.22
1150	11.50	483.0			3.0	29.10	65.21	2.24
1200	12.00	483.0	-42	-0.42	3.0	28.80	65.21	2.26





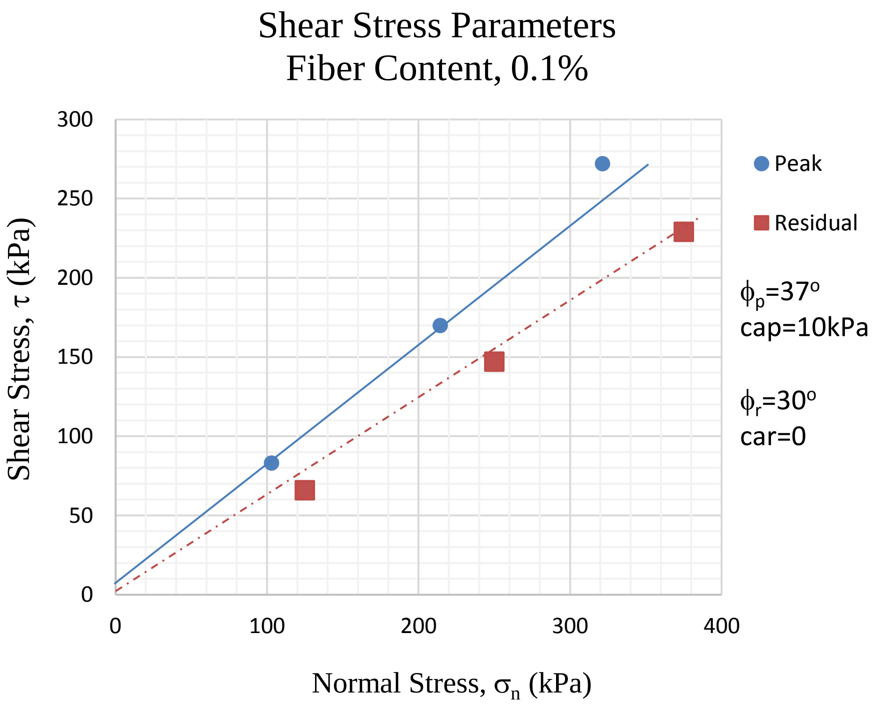


Figure A.2 : Test Results for Test No A-2.

Table A.4 : Direct Shear Test Results- B-2-1

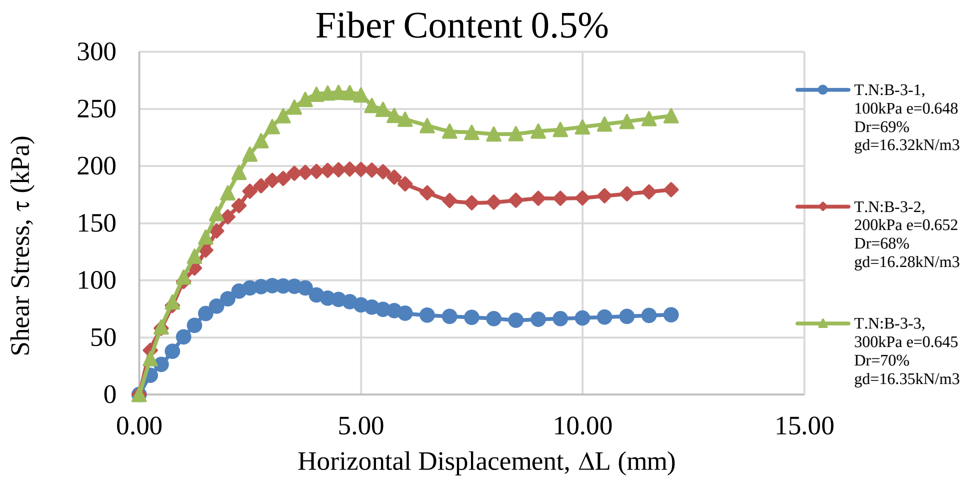
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	1.0	36.00	0.00	0.00
25	0.25	71.0			1.0	35.85	9.59	0.27
50	0.50	111.0			1.0	35.70	14.99	0.42
75	0.75	162.0			1.0	35.55	21.87	0.62
100	1.00	184.0	-35.0	-0.35	1.0	35.40	24.84	0.70
125	1.25	204.0			1.0	35.25	27.54	0.78
150	1.50	213.0			1.0	35.10	28.76	0.82
175	1.75	214.0			1.0	34.95	28.89	0.83
200	2.00	214.0	-22.0	-0.22	1.0	34.80	28.89	0.83
225	2.25	210.0			1.0	34.65	28.35	0.82
250	2.50	208.0			1.0	34.50	28.08	0.81
275	2.75	200.0			1.0	34.35	27.00	0.79
300	3.00	193.0	-9.0	-0.09	1.0	34.20	26.06	0.76
325	3.25	179.0			1.0	34.05	24.17	0.71
350	3.50	176.0			1.0	33.90	23.76	0.70
375	3.75	170.0			1.0	33.75	22.95	0.68
400	4.00	167.0	-7.0	-0.07	1.0	33.60	22.55	0.67
425	4.25	164.0			1.0	33.45	22.14	0.66
450	4.50	160.0			1.0	33.30	21.60	0.65
475	4.75	158.0			1.0	33.15	21.33	0.64
500	5.00	157.0	-6.0	-0.06	1.0	33.00	21.20	0.64
525	5.25	157.0			1.0	32.85	21.20	0.65
550	5.50	155.0			1.0	32.70	20.93	0.64
575	5.75	153.0			1.0	32.55	20.66	0.63
600	6.00	150.0	-6.0	-0.06	1.0	32.40	20.25	0.62
650	6.50	150.0			1.0	32.10	20.25	0.63
700	7.00	148.0	-6.0	-0.06	1.0	31.80	19.98	0.63
750	7.50	145.0			1.0	31.50	19.58	0.62
800	8.00	145.0	-6.0	-0.06	1.0	31.20	19.58	0.63
850	8.50	145.0			1.0	30.90	19.58	0.63
900	9.00	145.0	-7.0	-0.07	1.0	30.60	19.58	0.64
950	9.50	145.0			1.0	30.30	19.58	0.65
1000	10.00	145.0	-8.0	-0.08	1.0	30.00	19.58	0.65
1050	10.50	145.0			1.0	29.70	19.58	0.66
1100	11.00	145.0	-9.0	-0.09	1.0	29.40	19.58	0.67
1150	11.50	145.0			1.0	29.10	19.58	0.67
1200	12.00	145.0	-10.0	-0.1	1.0	28.80	19.58	0.68

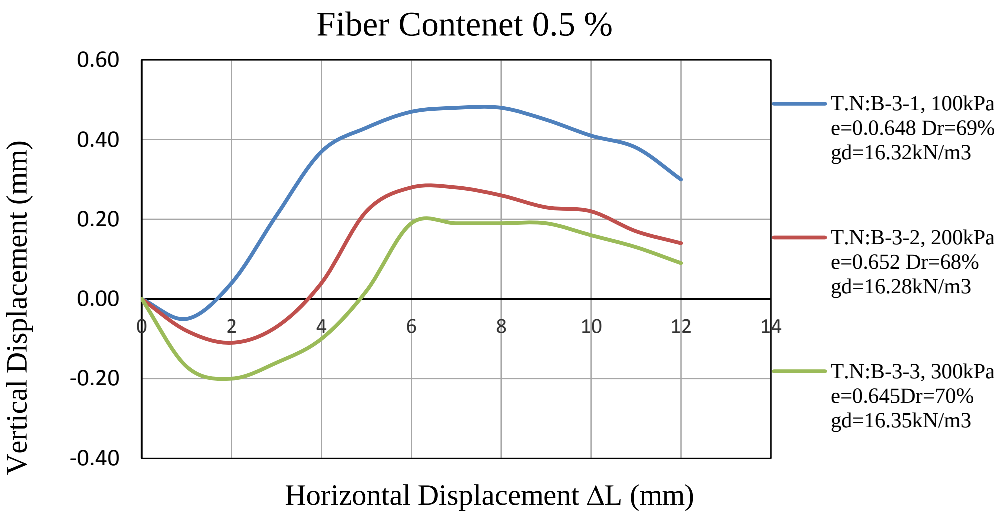
Table A.5 : Direct Shear Test Results- B-2-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm²)	Düzeltilmiş Alan (A_L) (cm²)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm²)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	2.0	36.00	0.00	0.00
25	0.25	90			2.0	35.85	12.15	0.34
50	0.50	170.0			2.0	35.70	22.95	0.64
75	0.75	240.0			2.0	35.55	32.40	0.91
100	1.00	288.0	-47.0	-0.47	2.0	35.40	38.88	1.10
125	1.25	335.0			2.0	35.25	45.23	1.28
150	1.50	361.0			2.0	35.10	48.74	1.39
175	1.75	385.0			2.0	34.95	51.98	1.49
200	2.00	402.0	-37.0	-0.37	2.0	34.80	54.27	1.56
225	2.25	415.0			2.0	34.65	56.03	1.62
250	2.50	425.0			2.0	34.50	57.38	1.66
275	2.75	427.0			2.0	34.35	57.65	1.68
300	3.00	428.0	-17.0	-0.17	2.0	34.20	57.78	1.69
325	3.25	427.0			2.0	34.05	57.65	1.69
350	3.50	426.0			2.0	33.90	57.51	1.70
375	3.75	424.0			2.0	33.75	57.24	1.70
400	4.00	422.0	-10.0	-0.1	2.0	33.60	56.97	1.70
425	4.25	409.0			2.0	33.45	55.22	1.65
450	4.50	398.0			2.0	33.30	53.73	1.61
475	4.75	387.0			2.0	33.15	52.25	1.58
500	5.00	383.0	0.0	0	2.0	33.00	51.71	1.57
525	5.25	375.0			2.0	32.85	50.63	1.54
550	5.50	368.0			2.0	32.70	49.68	1.52
575	5.75	365.0			2.0	32.55	49.28	1.51
600	6.00	362.0	0.0	0	2.0	32.40	48.87	1.51
650	6.50	358.0			2.0	32.10	48.33	1.51
700	7.00	352.0	-3.0	-0.03	2.0	31.80	47.52	1.49
750	7.50	345.0			2.0	31.50	46.58	1.48
800	8.00	342.0	-5.0	-0.05	2.0	31.20	46.17	1.48
850	8.50	340.0			2.0	30.90	45.90	1.49
900	9.00	337.0	-9.0	-0.09	2.0	30.60	45.50	1.49
950	9.50	337.0			2.0	30.30	45.50	1.50
1000	10.00	337.0	-13.0	-0.13	2.0	30.00	45.50	1.52
1050	10.50	337.0			2.0	29.70	45.50	1.53
1100	11.00	337.0	-16.0	-0.16	2.0	29.40	45.50	1.55
1150	11.50	337.0			2.0	29.10	45.50	1.56
1200	12.00	337.0	-20.0	-0.2	2.0	28.80	45.50	1.58

Table A.6 : Drect Shear Test Results- B-2-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σ_n (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	3.0	36.00	0.00	0.00
25	0.25	117			3.0	35.85	15.80	0.44
50	0.50	180.0			3.0	35.70	24.30	0.68
75	0.75	242.0			3.0	35.55	32.67	0.92
100	1.00	290.0	-63	-0.63	3.0	35.40	39.15	1.11
125	1.25	335.0			3.0	35.25	45.23	1.28
150	1.50	383.0			3.0	35.10	51.71	1.47
175	1.75	443.0			3.0	34.95	59.81	1.71
200	2.00	500.0	-67	-0.67	3.0	34.80	67.50	1.94
225	2.25	550.0			3.0	34.65	74.25	2.14
250	2.50	595.0			3.0	34.50	80.33	2.33
275	2.75	632.0			3.0	34.35	85.32	2.48
300	3.00	660.0	-63	-0.63	3.0	34.20	89.10	2.61
325	3.25	671.0			3.0	34.05	90.59	2.66
350	3.50	677.0			3.0	33.90	91.40	2.70
375	3.75	677.0			3.0	33.75	91.40	2.71
400	4.00	677.0	-47	-0.47	3.0	33.60	91.40	2.72
425	4.25	670.0			3.0	33.45	90.45	2.70
450	4.50	656.0			3.0	33.30	88.56	2.66
475	4.75	627.0			3.0	33.15	84.65	2.55
500	5.00	592.0	-33	-0.33	3.0	33.00	79.92	2.42
525	5.25	565.0			3.0	32.85	76.28	2.32
550	5.50	545.0			3.0	32.70	73.58	2.25
575	5.75	540.0			3.0	32.55	72.90	2.24
600	6.00	534.0	-32	-0.32	3.0	32.40	72.09	2.23
650	6.50	534.0			3.0	32.10	72.09	2.25
700	7.00	534.0	-34	-0.34	3.0	31.80	72.09	2.27
750	7.50	534.0			3.0	31.50	72.09	2.29
800	8.00	532.0	-37	-0.37	3.0	31.20	71.82	2.30
850	8.50	529.0			3.0	30.90	71.42	2.31
900	9.00	526.0	-40	-0.4	3.0	30.60	71.01	2.32
950	9.50	521.0			3.0	30.30	70.34	2.32
1000	10.00	521.0	-45	-0.45	3.0	30.00	70.34	2.34
1050	10.50	521.0			3.0	29.70	70.34	2.37
1100	11.00	521.0	-47	-0.47	3.0	29.40	70.34	2.39
1150	11.50	521.0			3.0	29.10	70.34	2.42
1200	12.00	521.0	-50	-0.5	3.0	28.80	70.34	2.44





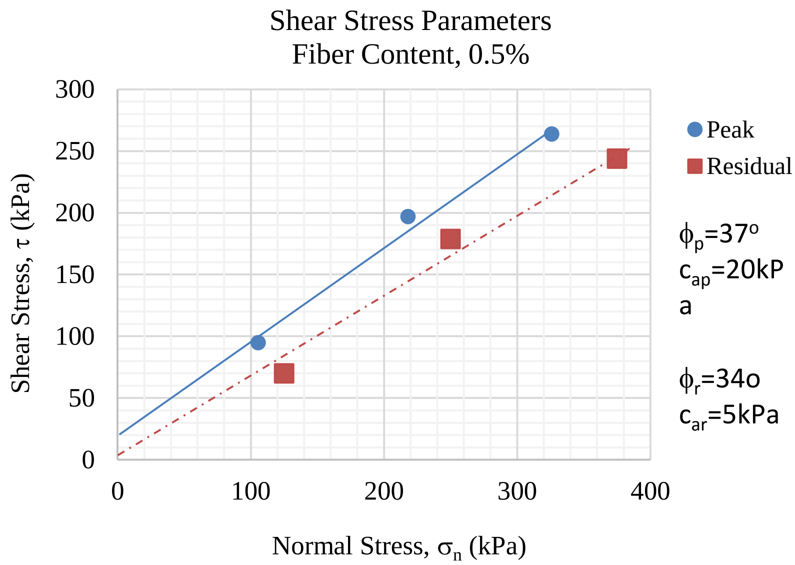


Figure A.3 : Test Results for test No A-3.

Table A.7 : Direct Shear Test Results- B-3-1

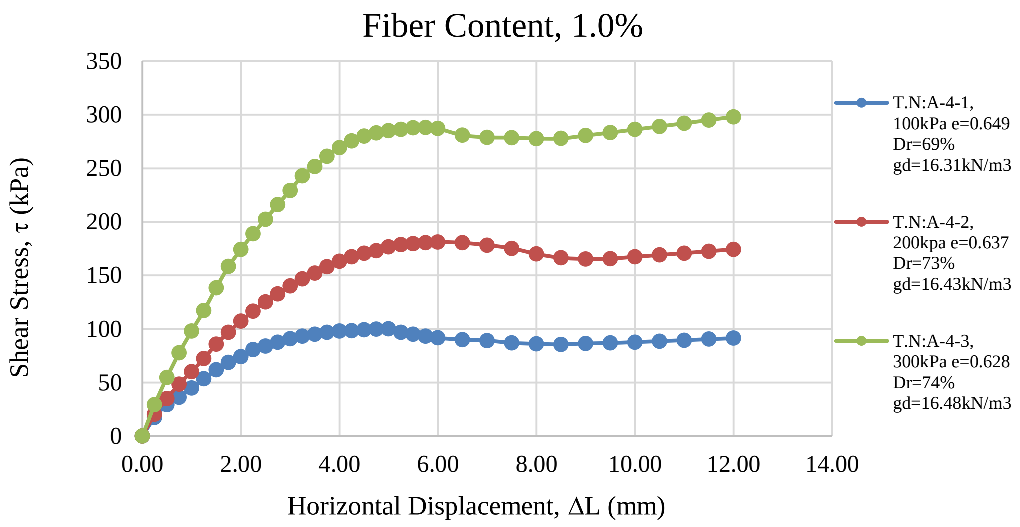
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	-54.0	-0.54	1.0	36.00	0.00	0.00
25	0.25	48.0			1.0	35.85	6.05	0.17
50	0.50	75.0			1.0	35.70	9.45	0.26
75	0.75	107.0			1.0	35.55	13.48	0.38
100	1.00	142.0	-74.0	-0.74	1.0	35.40	17.89	0.51
125	1.25	170.0			1.0	35.25	21.42	0.61
150	1.50	198.0			1.0	35.10	24.95	0.71
175	1.75	215.0			1.0	34.95	27.09	0.78
200	2.00	232.0	-68.0	-0.68	1.0	34.80	29.23	0.84
225	2.25	249.0			1.0	34.65	31.37	0.91
250	2.50	256.0			1.0	34.50	32.26	0.93
275	2.75	258.0			1.0	34.35	32.51	0.95
300	3.00	259.0	-49.0	-0.49	1.0	34.20	32.63	0.95
325	3.25	257.0			1.0	34.05	32.38	0.95
350	3.50	255.0			1.0	33.90	32.13	0.95
375	3.75	250.0			1.0	33.75	31.50	0.93
400	4.00	233.0	-24.0	-0.24	1.0	33.60	29.36	0.87
425	4.25	224.0			1.0	33.45	28.22	0.84
450	4.50	220.0			1.0	33.30	27.72	0.83
475	4.75	214.0			1.0	33.15	26.96	0.81
500	5.00	206.0	-30.0	-0.3	1.0	33.00	25.96	0.79
525	5.25	200.0			1.0	32.85	25.20	0.77
550	5.50	194.0			1.0	32.70	24.44	0.75
575	5.75	190.0			1.0	32.55	23.94	0.74
600	6.00	183.0	-30.0	-0.3	1.0	32.40	23.06	0.71
650	6.50	177.0			1.0	32.10	22.30	0.69
700	7.00	173.0	-30.0	-0.3	1.0	31.80	21.80	0.69
750	7.50	169.0			1.0	31.50	21.29	0.68
800	8.00	165.0	-30.0	-0.3	1.0	31.20	20.79	0.67
850	8.50	160.0			1.0	30.90	20.16	0.65
900	9.00	160.0	-33.0	-0.33	1.0	30.60	20.16	0.66
950	9.50	160.0			1.0	30.30	20.16	0.67
1000	10.00	160.0	-34.0	-0.34	1.0	30.00	20.16	0.67
1050	10.50	160.0			1.0	29.70	20.16	0.68
1100	11.00	160.0	-37.0	-0.37	1.0	29.40	20.16	0.69
1150	11.50	160.0			1.0	29.10	20.16	0.69
1200	12.00	160.0	-40.0	-0.4	1.0	28.80	20.16	0.70

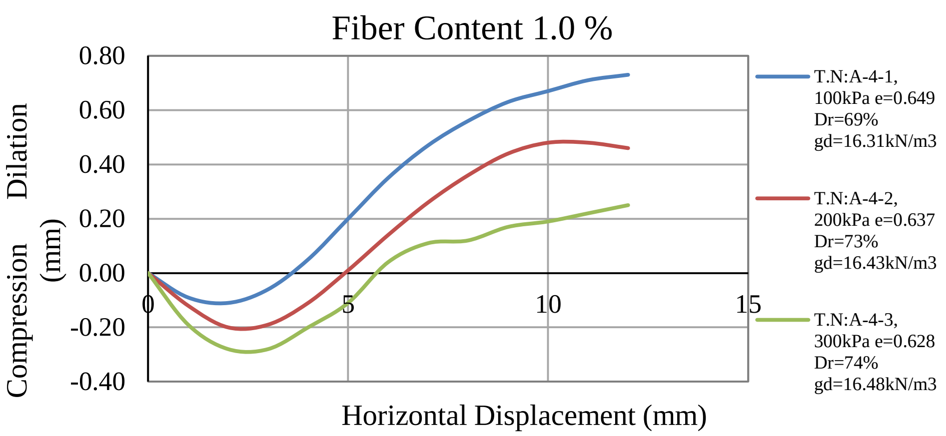
Table A.8 : Direct Shear Test Results- B-3-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm²)	Düzeltilmiş Alan (A_L) (cm²)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm²)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	-48.0	-0.48	2.0	36.00	0.00	0.00
25	0.25	110			2.0	35.85	13.86	0.39
50	0.50	165.0			2.0	35.70	20.79	0.58
75	0.75	220.0			2.0	35.55	27.72	0.78
100	1.00	278.0	-60.0	-0.6	2.0	35.40	35.03	0.99
125	1.25	310.0			2.0	35.25	39.06	1.11
150	1.50	352.0			2.0	35.10	44.35	1.26
175	1.75	397.0			2.0	34.95	50.02	1.43
200	2.00	430.0	-58.0	-0.58	2.0	34.80	54.18	1.56
225	2.25	455.0			2.0	34.65	57.33	1.65
250	2.50	487.0			2.0	34.50	61.36	1.78
275	2.75	498.0			2.0	34.35	62.75	1.83
300	3.00	509.0	-43.0	-0.43	2.0	34.20	64.13	1.88
325	3.25	511.0			2.0	34.05	64.39	1.89
350	3.50	521.0			2.0	33.90	65.65	1.94
375	3.75	521.0			2.0	33.75	65.65	1.95
400	4.00	521.0	-22.0	-0.22	2.0	33.60	65.65	1.95
425	4.25	521.0			2.0	33.45	65.65	1.96
450	4.50	520.0			2.0	33.30	65.52	1.97
475	4.75	519.0			2.0	33.15	65.39	1.97
500	5.00	516.0	0.0	0	2.0	33.00	65.02	1.97
525	5.25	512.0			2.0	32.85	64.51	1.96
550	5.50	506.0			2.0	32.70	63.76	1.95
575	5.75	492.0			2.0	32.55	61.99	1.90
600	6.00	474.0	16.0	0.16	2.0	32.40	59.72	1.84
650	6.50	450.0			2.0	32.10	56.70	1.77
700	7.00	429.0	25.0	0.25	2.0	31.80	54.05	1.70
750	7.50	420.0			2.0	31.50	52.92	1.68
800	8.00	417.0	25.0	0.25	2.0	31.20	52.54	1.68
850	8.50	417.0			2.0	30.90	52.54	1.70
900	9.00	417.0	25.0	0.25	2.0	30.60	52.54	1.72
950	9.50	413.0			2.0	30.30	52.04	1.72
1000	10.00	410.0	22.0	0.22	2.0	30.00	51.66	1.72
1050	10.50	410.0			2.0	29.70	51.66	1.74
1100	11.00	410.0	22.0	0.22	2.0	29.40	51.66	1.76
1150	11.50	410.0			2.0	29.10	51.66	1.78
1200	12.00	410.0	20.0	0.2	2.0	28.80	51.66	1.79

Table A.9 : Direct Shear Test Results- B-3-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	-58	-0.58	3.0	36.00	0.00	0.00
25	0.25	90			3.0	35.85	11.34	0.32
50	0.50	168.0			3.0	35.70	21.17	0.59
75	0.75	229.0			3.0	35.55	28.85	0.81
100	1.00	289.0	-68	-0.68	3.0	35.40	36.41	1.03
125	1.25	339.0			3.0	35.25	42.71	1.21
150	1.50	384.0			3.0	35.10	48.38	1.38
175	1.75	439.0			3.0	34.95	55.31	1.58
200	2.00	488.0	-73	-0.73	3.0	34.80	61.49	1.77
225	2.25	535.0			3.0	34.65	67.41	1.95
250	2.50	576.0			3.0	34.50	72.58	2.10
275	2.75	606.0			3.0	34.35	76.36	2.22
300	3.00	637.0	-70	-0.7	3.0	34.20	80.26	2.35
325	3.25	660.0			3.0	34.05	83.16	2.44
350	3.50	677.0			3.0	33.90	85.30	2.52
375	3.75	692.0			3.0	33.75	87.19	2.58
400	4.00	701.0	-60	-0.6	3.0	33.60	88.33	2.63
425	4.25	701.0			3.0	33.45	88.33	2.64
450	4.50	699.0			3.0	33.30	88.07	2.64
475	4.75	695.0			3.0	33.15	87.57	2.64
500	5.00	687.0	-50	-0.5	3.0	33.00	86.56	2.62
525	5.25	660.0			3.0	32.85	83.16	2.53
550	5.50	648.0			3.0	32.70	81.65	2.50
575	5.75	631.0			3.0	32.55	79.51	2.44
600	6.00	620.0	-40	-0.4	3.0	32.40	78.12	2.41
650	6.50	600.0			3.0	32.10	75.60	2.36
700	7.00	582.0	-37	-0.37	3.0	31.80	73.33	2.31
750	7.50	574.0			3.0	31.50	72.32	2.30
800	8.00	565.0	-37	-0.37	3.0	31.20	71.19	2.28
850	8.50	560.0			3.0	30.90	70.56	2.28
900	9.00	560.0	-40	-0.4	3.0	30.60	70.56	2.31
950	9.50	558.0			3.0	30.30	70.31	2.32
1000	10.00	558.0	-42	-0.42	3.0	30.00	70.31	2.34
1050	10.50	558.0			3.0	29.70	70.31	2.37
1100	11.00	558.0	-45	-0.45	3.0	29.40	70.31	2.39
1150	11.50	558.0			3.0	29.10	70.31	2.42
1200	12.00	558.0	-52	-0.52	3.0	28.80	70.31	2.44





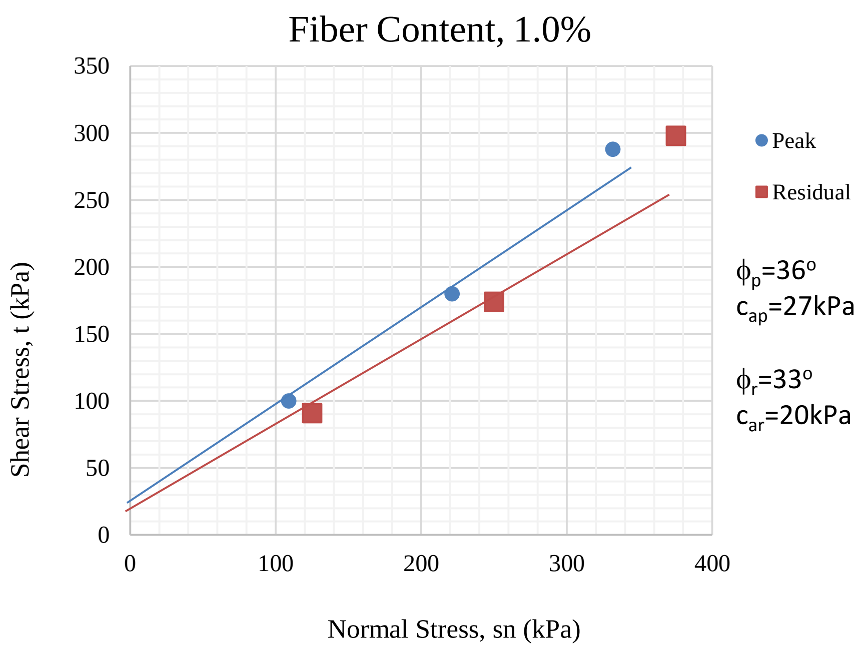


Figure A.4 : Test results for Test No A-4.

Table A.10 : Direct Shear Test Results- A-4-1

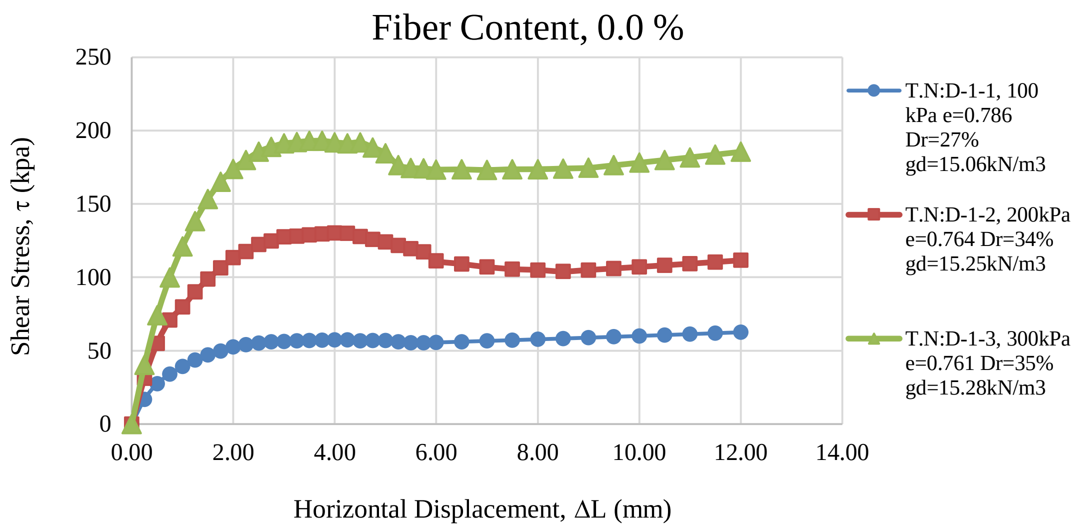
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	-40.0	-0.4	1.0	36.00	0.00	0.00
25	0.25	46			1.0	35.85	6.21	0.17
50	0.50	78.0			1.0	35.70	10.53	0.29
75	0.75	95.0			1.0	35.55	12.83	0.36
100	1.00	118.0	-49.0	-0.49	1.0	35.40	15.93	0.45
125	1.25	140.0			1.0	35.25	18.90	0.54
150	1.50	161.0			1.0	35.10	21.74	0.62
175	1.75	178.0			1.0	34.95	24.03	0.69
200	2.00	191.0	-51.0	-0.51	1.0	34.80	25.79	0.74
225	2.25	207.0			1.0	34.65	27.95	0.81
250	2.50	215.0			1.0	34.50	29.03	0.84
275	2.75	223.0			1.0	34.35	30.11	0.88
300	3.00	230.0	-42.0	-0.42	1.0	34.20	31.05	0.91
325	3.25	235.0			1.0	34.05	31.73	0.93
350	3.50	239.0			1.0	33.90	32.27	0.95
375	3.75	242.0			1.0	33.75	32.67	0.97
400	4.00	244.0	-35.0	-0.35	1.0	33.60	32.94	0.98
425	4.25	244.0			1.0	33.45	32.94	0.98
450	4.50	245.0			1.0	33.30	33.08	0.99
475	4.75	245.0			1.0	33.15	33.08	1.00
500	5.00	245.0	-20.0	-0.2	1.0	33.00	33.08	1.00
525	5.25	236.0			1.0	32.85	31.86	0.97
550	5.50	230.0			1.0	32.70	31.05	0.95
575	5.75	225.0			1.0	32.55	30.38	0.93
600	6.00	220.0	-5.0	-0.05	1.0	32.40	29.70	0.92
650	6.50	214.0			1.0	32.10	28.89	0.90
700	7.00	210.0	7.0	0.07	1.0	31.80	28.35	0.89
750	7.50	203.0			1.0	31.50	27.41	0.87
800	8.00	199.0	16.0	0.16	1.0	31.20	26.87	0.86
850	8.50	196.0			1.0	30.90	26.46	0.86
900	9.00	196.0	23.0	0.23	1.0	30.60	26.46	0.86
950	9.50	195.0			1.0	30.30	26.33	0.87
1000	10.00	195.0	27.0	0.27	1.0	30.00	26.33	0.88
1050	10.50	195.0			1.0	29.70	26.33	0.89
1100	11.00	195.0	31.0	0.31	1.0	29.40	26.33	0.90
1150	11.50	195.0			1.0	29.10	26.33	0.90
1200	12.00	195.0	33.0	0.33	1.0	28.80	26.33	0.91

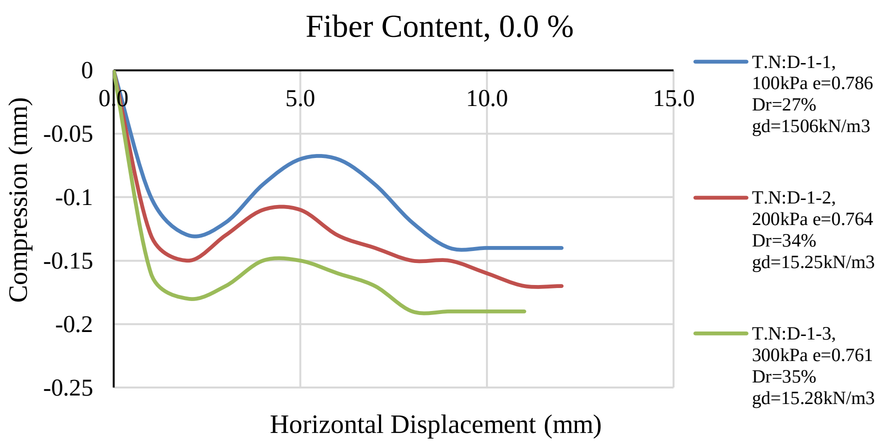
Table A.11 : Direct Shear Test Results- A-4-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	2.0	36.00	0.00	0.00
25	0.25	54			2.0	35.85	7.29	0.20
50	0.50	93.0			2.0	35.70	12.56	0.35
75	0.75	128.0			2.0	35.55	17.28	0.49
100	1.00	158.0	-70.0	-0.7	2.0	35.40	21.33	0.60
125	1.25	189.0			2.0	35.25	25.52	0.72
150	1.50	223.0			2.0	35.10	30.11	0.86
175	1.75	251.0			2.0	34.95	33.89	0.97
200	2.00	277.0	-78.0	-0.78	2.0	34.80	37.40	1.07
225	2.25	299.0			2.0	34.65	40.37	1.16
250	2.50	320.0			2.0	34.50	43.20	1.25
275	2.75	338.0			2.0	34.35	45.63	1.33
300	3.00	355.0	-77.0	-0.77	2.0	34.20	47.93	1.40
325	3.25	370.0			2.0	34.05	49.95	1.47
350	3.50	382.0			2.0	33.90	51.57	1.52
375	3.75	395.0			2.0	33.75	53.33	1.58
400	4.00	406.0	-69.0	-0.69	2.0	33.60	54.81	1.63
425	4.25	415.0			2.0	33.45	56.03	1.67
450	4.50	421.0			2.0	33.30	56.84	1.71
475	4.75	425.0			2.0	33.15	57.38	1.73
500	5.00	432.0	-57.0	-0.57	2.0	33.00	58.32	1.77
525	5.25	435.0			2.0	32.85	58.73	1.79
550	5.50	435.0			2.0	32.70	58.73	1.80
575	5.75	435.0			2.0	32.55	58.73	1.80
600	6.00	435.0	-44.0	-0.44	2.0	32.40	58.73	1.81
650	6.50	429.0			2.0	32.10	57.92	1.80
700	7.00	420.0	-32.0	-0.32	2.0	31.80	56.70	1.78
750	7.50	409.0			2.0	31.50	55.22	1.75
800	8.00	393.0	-22.0	-0.22	2.0	31.20	53.06	1.70
850	8.50	381.0			2.0	30.90	51.44	1.66
900	9.00	375.0	-14.0	-0.14	2.0	30.60	50.63	1.65
950	9.50	372.0			2.0	30.30	50.22	1.66
1000	10.00	372.0	-10.0	-0.1	2.0	30.00	50.22	1.67
1050	10.50	372.0			2.0	29.70	50.22	1.69
1100	11.00	372.0	-10.0	-0.1	2.0	29.40	50.22	1.71
1150	11.50	372.0			2.0	29.10	50.22	1.73
1200	12.00	372.0	-12.0	-0.12	2.0	28.80	50.22	1.74

Table A.12 : Direct Shear Test Results- A-4-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0.0	0	3.0	36.00	0.00	0.00
25	0.25	78			3.0	35.85	10.53	0.29
50	0.50	145.0			3.0	35.70	19.58	0.55
75	0.75	205.0			3.0	35.55	27.68	0.78
100	1.00	257.0	-84.0	-0.84	3.0	35.40	34.70	0.98
125	1.25	306.0			3.0	35.25	41.31	1.17
150	1.50	360.0			3.0	35.10	48.60	1.38
175	1.75	410.0			3.0	34.95	55.35	1.58
200	2.00	449.0	-93.0	-0.93	3.0	34.80	60.62	1.74
225	2.25	485.0			3.0	34.65	65.48	1.89
250	2.50	517.0			3.0	34.50	69.80	2.02
275	2.75	550.0			3.0	34.35	74.25	2.16
300	3.00	581.0	-93.0	-0.93	3.0	34.20	78.44	2.29
325	3.25	613.0			3.0	34.05	82.76	2.43
350	3.50	632.0			3.0	33.90	85.32	2.52
375	3.75	653.0			3.0	33.75	88.16	2.61
400	4.00	670.0	-85.0	-0.85	3.0	33.60	90.45	2.69
425	4.25	683.0			3.0	33.45	92.21	2.76
450	4.50	691.0			3.0	33.30	93.29	2.80
475	4.75	695.0			3.0	33.15	93.83	2.83
500	5.00	697.0	-84.0	-0.84	3.0	33.00	94.10	2.85
525	5.25	697.0			3.0	32.85	94.10	2.86
550	5.50	697.0			3.0	32.70	94.10	2.88
575	5.75	695.0			3.0	32.55	93.83	2.88
600	6.00	689.0	-61.0	-0.61	3.0	32.40	93.02	2.87
650	6.50	668.0			3.0	32.10	90.18	2.81
700	7.00	657.0	-54.0	-0.54	3.0	31.80	88.70	2.79
750	7.50	650.0			3.0	31.50	87.75	2.79
800	8.00	642.0	-53.0	-0.53	3.0	31.20	86.67	2.78
850	8.50	636.0			3.0	30.90	85.86	2.78
900	9.00	636.0	-48.0	-0.48	3.0	30.60	85.86	2.81
950	9.50	636.0			3.0	30.30	85.86	2.83
1000	10.00	636.0	-46.0	-0.46	3.0	30.00	85.86	2.86
1050	10.50	636.0			3.0	29.70	85.86	2.89
1100	11.00	636.0	-43.0	-0.43	3.0	29.40	85.86	2.92
1150	11.50	636.0			3.0	29.10	85.86	2.95
1200	12.00	636.0	-40.0	-0.4	3.0	28.80	85.86	2.98





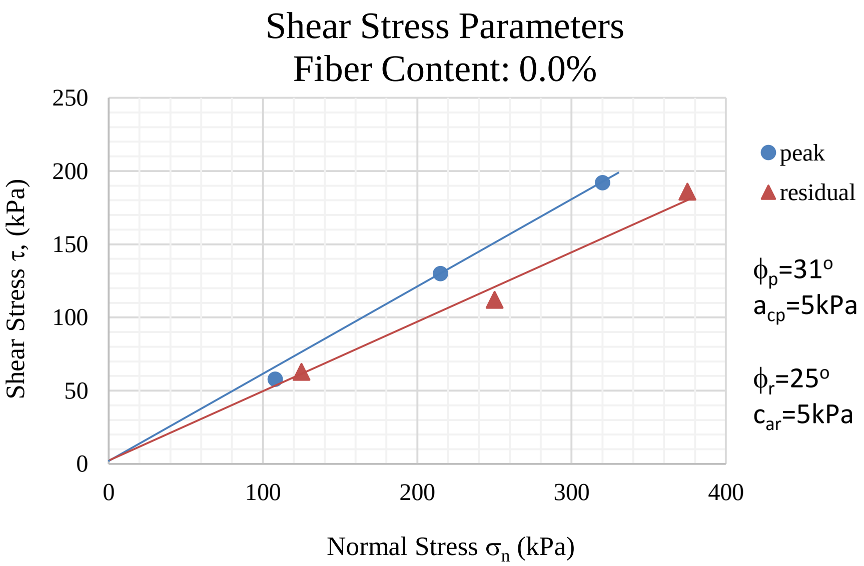


Figure A.5 : Test Results for Test No D-1.

Table A.13 : Direct Shear Test Results- D-1-1

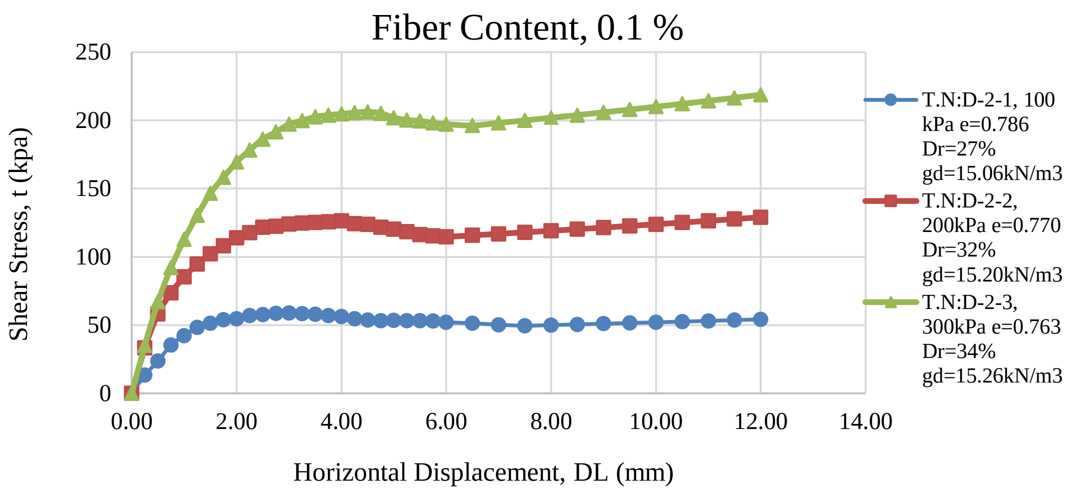
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	1.0	36.00	0.00	0.00
25	0.25	48			1.0	35.85	6.05	0.17
50	0.50	78.0			1.0	35.70	9.83	0.28
75	0.75	96.0			1.0	35.55	12.10	0.34
100	1.00	110.0	-56	-0.14	1.0	35.40	13.86	0.39
125	1.25	122.0			1.0	35.25	15.37	0.44
150	1.50	131.0			1.0	35.10	16.51	0.47
175	1.75	138.0			1.0	34.95	17.39	0.50
200	2.00	145.0	-58	-0.16	1.0	34.80	18.27	0.53
225	2.25	149.0			1.0	34.65	18.77	0.54
250	2.50	151.0			1.0	34.50	19.03	0.55
275	2.75	153.0			1.0	34.35	19.28	0.56
300	3.00	153.0	-55	-0.13	1.0	34.20	19.28	0.56
325	3.25	153.0			1.0	34.05	19.28	0.57
350	3.50	153.0			1.0	33.90	19.28	0.57
375	3.75	153.0			1.0	33.75	19.28	0.57
400	4.00	153.0	-52	-0.1	1.0	33.60	19.28	0.57
425	4.25	152.0			1.0	33.45	19.15	0.57
450	4.50	150.0			1.0	33.30	18.90	0.57
475	4.75	150.0			1.0	33.15	18.90	0.57
500	5.00	149.0	-51	-0.09	1.0	33.00	18.77	0.57
525	5.25	146.0			1.0	32.85	18.40	0.56
550	5.50	144.0			1.0	32.70	18.14	0.55
575	5.75	143.0			1.0	32.55	18.02	0.55
600	6.00	143.0	-52	-0.1	1.0	32.40	18.02	0.56
650	6.50	143.0			1.0	32.10	18.02	0.56
700	7.00	143.0	-57	-0.15	1.0	31.80	18.02	0.57
750	7.50	143.0			1.0	31.50	18.02	0.57
800	8.00	143.0	-64	-0.22	1.0	31.20	18.02	0.58
850	8.50	143.0			1.0	30.90	18.02	0.58
900	9.00	143.0	-70	-0.28	1.0	30.60	18.02	0.59
950	9.50	143.0			1.0	30.30	18.02	0.59
1000	10.00	143.0	-75	-0.33	1.0	30.00	18.02	0.60
1050	10.50	143.0			1.0	29.70	18.02	0.61
1100	11.00	143.0	-78	-0.36	1.0	29.40	18.02	0.61
1150	11.50	143.0			1.0	29.10	18.02	0.62
1200	12.00	143.0	-80	-0.38	1.0	28.80	18.02	0.63

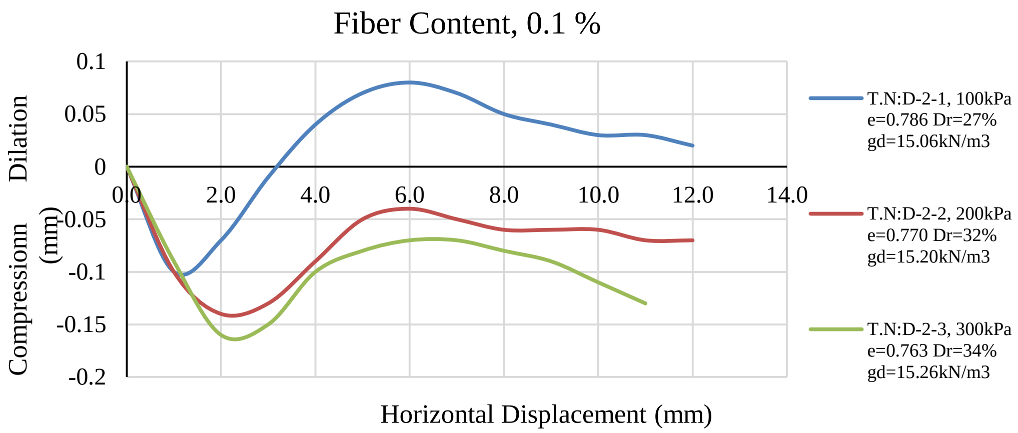
Table A.14 : Direct Shear Test Results- D-1-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	2.0	36.00	0.00	0.00
25	0.25	89			2.0	35.85	11.21	0.31
50	0.50	156.0			2.0	35.70	19.66	0.55
75	0.75	200.0			2.0	35.55	25.20	0.71
100	1.00	224.0	-78	-0.13	2.0	35.40	28.22	0.80
125	1.25	252.0			2.0	35.25	31.75	0.90
150	1.50	275.0			2.0	35.10	34.65	0.99
175	1.75	295.0			2.0	34.95	37.17	1.06
200	2.00	313.0	-85	-0.2	2.0	34.80	39.44	1.13
225	2.25	323.0			2.0	34.65	40.70	1.17
250	2.50	335.0			2.0	34.50	42.21	1.22
275	2.75	340.0			2.0	34.35	42.84	1.25
300	3.00	346.0	-85	-0.2	2.0	34.20	43.60	1.27
325	3.25	346.0			2.0	34.05	43.60	1.28
350	3.50	347.0			2.0	33.90	43.72	1.29
375	3.75	347.0			2.0	33.75	43.72	1.30
400	4.00	347.0	-83	-0.18	2.0	33.60	43.72	1.30
425	4.25	345.0			2.0	33.45	43.47	1.30
450	4.50	338.0			2.0	33.30	42.59	1.28
475	4.75	331.0			2.0	33.15	41.71	1.26
500	5.00	325.0	-82	-0.17	2.0	33.00	40.95	1.24
525	5.25	317.0			2.0	32.85	39.94	1.22
550	5.50	310.0			2.0	32.70	39.06	1.19
575	5.75	303.0			2.0	32.55	38.18	1.17
600	6.00	286.0	-85	-0.2	2.0	32.40	36.04	1.11
650	6.50	278.0			2.0	32.10	35.03	1.09
700	7.00	270.0	-93	-0.28	2.0	31.80	34.02	1.07
750	7.50	264.0			2.0	31.50	33.26	1.06
800	8.00	260.0	-93	-0.28	2.0	31.20	32.76	1.05
850	8.50	255.0			2.0	30.90	32.13	1.04
900	9.00	255.0	-93	-0.28	2.0	30.60	32.13	1.05
950	9.50	255.0			2.0	30.30	32.13	1.06
1000	10.00	255.0	-93	-0.28	2.0	30.00	32.13	1.07
1050	10.50	255.0			2.0	29.70	32.13	1.08
1100	11.00	255.0	-94	-0.29	2.0	29.40	32.13	1.09
1150	11.50	255.0			2.0	29.10	32.13	1.10
1200	12.00	255.0	-97	-0.32	2.0	28.80	32.13	1.12

Table A.15 : Direct Shear Test Results- D-1-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	3.0	36.00	0.00	0.00
25	0.25	115			3.0	35.85	14.49	0.40
50	0.50	210.0			3.0	35.70	26.46	0.74
75	0.75	282.0			3.0	35.55	35.53	1.00
100	1.00	340.0	-87	-0.07	3.0	35.40	42.84	1.21
125	1.25	387.0			3.0	35.25	48.76	1.38
150	1.50	427.0			3.0	35.10	53.80	1.53
175	1.75	458.0			3.0	34.95	57.71	1.65
200	2.00	480.0	-92	-0.12	3.0	34.80	60.48	1.74
225	2.25	495.0			3.0	34.65	62.37	1.80
250	2.50	508.0			3.0	34.50	64.01	1.86
275	2.75	515.0			3.0	34.35	64.89	1.89
300	3.00	519.0	-89	-0.09	3.0	34.20	65.39	1.91
325	3.25	519.0			3.0	34.05	65.39	1.92
350	3.50	519.0			3.0	33.90	65.39	1.93
375	3.75	517.0			3.0	33.75	65.14	1.93
400	4.00	512.0	-85	-0.05	3.0	33.60	64.51	1.92
425	4.25	508.0			3.0	33.45	64.01	1.91
450	4.50	507.0			3.0	33.30	63.88	1.92
475	4.75	496.0			3.0	33.15	62.50	1.89
500	5.00	483.0	-85	-0.05	3.0	33.00	60.86	1.84
525	5.25	460.0			3.0	32.85	57.96	1.76
550	5.50	453.0			3.0	32.70	57.08	1.75
575	5.75	450.0			3.0	32.55	56.70	1.74
600	6.00	446.0	-88	-0.08	3.0	32.40	56.20	1.73
650	6.50	442.0			3.0	32.10	55.69	1.73
700	7.00	437.0	-97	-0.17	3.0	31.80	55.06	1.73
750	7.50	434.0			3.0	31.50	54.68	1.74
800	8.00	430.0	-97	-0.17	3.0	31.20	54.18	1.74
850	8.50	427.0			3.0	30.90	53.80	1.74
900	9.00	424.0	-99	-0.19	3.0	30.60	53.42	1.75
950	9.50	424.0			3.0	30.30	53.42	1.76
1000	10.00	424.0	-99	-0.19	3.0	30.00	53.42	1.78
1050	10.50	424.0			3.0	29.70	53.42	1.80
1100	11.00	424.0	-99	-0.19	3.0	29.40	53.42	1.82
1150	11.50	424.0			3.0	29.10	53.42	1.84
1200	12.00	424.0	-99	-0.19	3.0	28.80	53.42	1.86





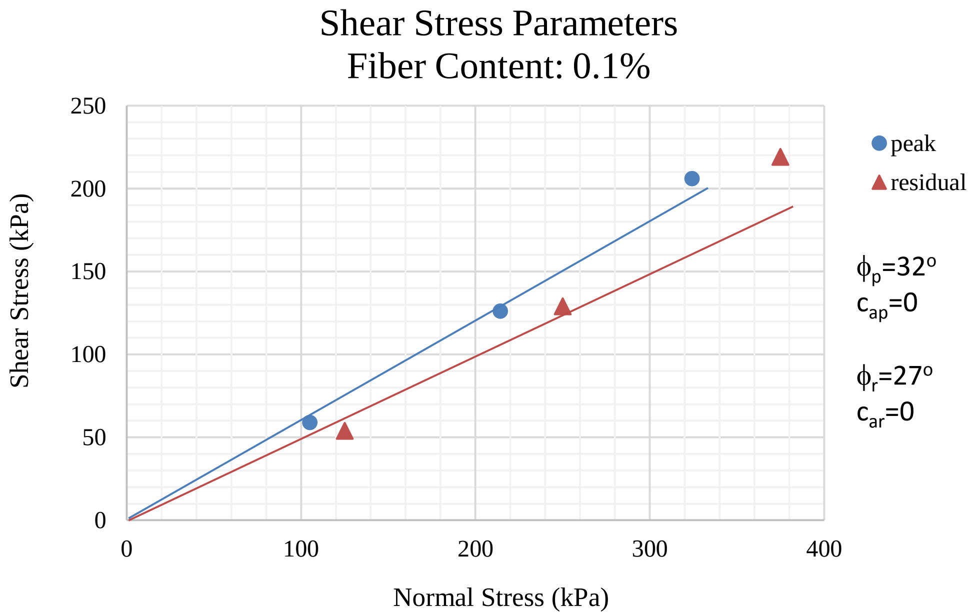


Figure A.6 : Test Results for Test No D-2

Table A.16 : Direct Shear Test Results- D-2-1

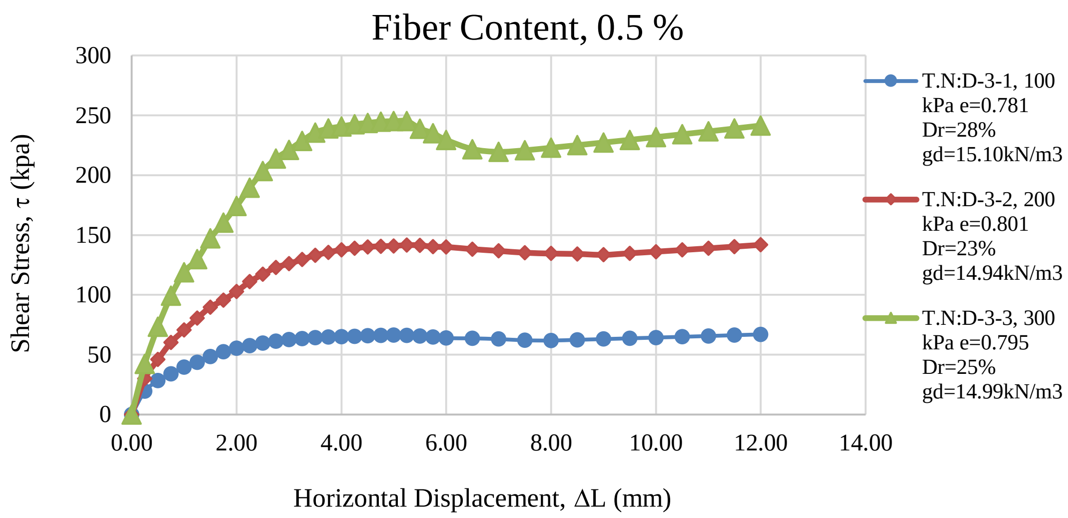
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	1.0	36.00	0.00	0.00
25	0.25	38			1.0	35.85	4.79	0.13
50	0.50	67.0			1.0	35.70	8.44	0.24
75	0.75	100.0			1.0	35.55	12.60	0.35
100	1.00	119.0	-53	-0.1	1.0	35.40	14.99	0.42
125	1.25	135.0			1.0	35.25	17.01	0.48
150	1.50	143.0			1.0	35.10	18.02	0.51
175	1.75	150.0			1.0	34.95	18.90	0.54
200	2.00	151.0	-50	-0.07	1.0	34.80	19.03	0.55
225	2.25	157.0			1.0	34.65	19.78	0.57
250	2.50	158.0			1.0	34.50	19.91	0.58
275	2.75	160.0			1.0	34.35	20.16	0.59
300	3.00	160.0	-44	-0.01	1.0	34.20	20.16	0.59
325	3.25	158.0			1.0	34.05	19.91	0.58
350	3.50	156.0			1.0	33.90	19.66	0.58
375	3.75	153.0			1.0	33.75	19.28	0.57
400	4.00	150.0	-40	0.03	1.0	33.60	18.90	0.56
425	4.25	145.0			1.0	33.45	18.27	0.55
450	4.50	142.0			1.0	33.30	17.89	0.54
475	4.75	140.0			1.0	33.15	17.64	0.53
500	5.00	140.0	-36	0.07	1.0	33.00	17.64	0.53
525	5.25	139.0			1.0	32.85	17.51	0.53
550	5.50	138.0			1.0	32.70	17.39	0.53
575	5.75	137.0			1.0	32.55	17.26	0.53
600	6.00	134.0	-36	0.07	1.0	32.40	16.88	0.52
650	6.50	131.0			1.0	32.10	16.51	0.51
700	7.00	127.0	-38	0.05	1.0	31.80	16.00	0.50
750	7.50	124.0			1.0	31.50	15.62	0.50
800	8.00	124.0	-41	0.02	1.0	31.20	15.62	0.50
850	8.50	124.0			1.0	30.90	15.62	0.51
900	9.00	124.0	-41	0.02	1.0	30.60	15.62	0.51
950	9.50	124.0			1.0	30.30	15.62	0.52
1000	10.00	124.0	-41	0.02	1.0	30.00	15.62	0.52
1050	10.50	124.0			1.0	29.70	15.62	0.53
1100	11.00	124.0	-41	0.02	1.0	29.40	15.62	0.53
1150	11.50	124.0			1.0	29.10	15.62	0.54
1200	12.00	124.0	-41	0.02	1.0	28.80	15.62	0.54

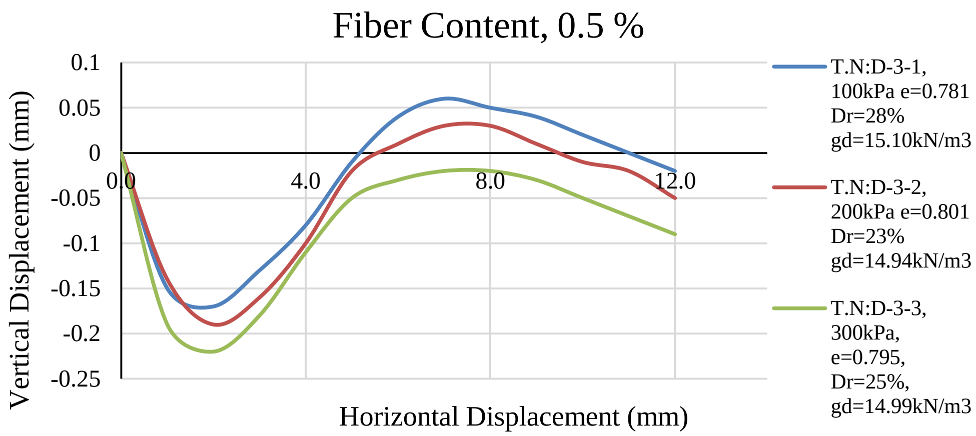
Table A.17 : Direct Shear Test Results- D-2-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	2.0	36.00	0.00	0.00
25	0.25	95			2.0	35.85	11.97	0.33
50	0.50	165.0			2.0	35.70	20.79	0.58
75	0.75	208.0			2.0	35.55	26.21	0.74
100	1.00	240.0	-76	-0.1	2.0	35.40	30.24	0.85
125	1.25	265.0			2.0	35.25	33.39	0.95
150	1.50	285.0			2.0	35.10	35.91	1.02
175	1.75	300.0			2.0	34.95	37.80	1.08
200	2.00	315.0	-80	-0.14	2.0	34.80	39.69	1.14
225	2.25	324.0			2.0	34.65	40.82	1.18
250	2.50	333.0			2.0	34.50	41.96	1.22
275	2.75	334.0			2.0	34.35	42.08	1.23
300	3.00	337.0	-77	-0.11	2.0	34.20	42.46	1.24
325	3.25	337.0			2.0	34.05	42.46	1.25
350	3.50	337.0			2.0	33.90	42.46	1.25
375	3.75	337.0			2.0	33.75	42.46	1.26
400	4.00	337.0	-75	-0.09	2.0	33.60	42.46	1.26
425	4.25	330.0			2.0	33.45	41.58	1.24
450	4.50	327.0			2.0	33.30	41.20	1.24
475	4.75	320.0			2.0	33.15	40.32	1.22
500	5.00	315.0	-75	-0.09	2.0	33.00	39.69	1.20
525	5.25	309.0			2.0	32.85	38.93	1.19
550	5.50	302.0			2.0	32.70	38.05	1.16
575	5.75	298.0			2.0	32.55	37.55	1.15
600	6.00	295.0	-80	-0.14	2.0	32.40	37.17	1.15
650	6.50	295.0			2.0	32.10	37.17	1.16
700	7.00	295.0	-85	-0.19	2.0	31.80	37.17	1.17
750	7.50	295.0			2.0	31.50	37.17	1.18
800	8.00	295.0	-85	-0.19	2.0	31.20	37.17	1.19
850	8.50	295.0			2.0	30.90	37.17	1.20
900	9.00	295.0	-85	-0.19	2.0	30.60	37.17	1.21
950	9.50	295.0			2.0	30.30	37.17	1.23
1000	10.00	295.0	-85	-0.19	2.0	30.00	37.17	1.24
1050	10.50	295.0			2.0	29.70	37.17	1.25
1100	11.00	295.0	-85	-0.19	2.0	29.40	37.17	1.26
1150	11.50	295.0			2.0	29.10	37.17	1.28
1200	12.00	295.0	-85	-0.19	2.0	28.80	37.17	1.29

Table A.18 : Direct Shear Test Results- D-2-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	3.0	36.00	0.00	0.00
25	0.25	100			3.0	35.85	12.60	0.35
50	0.50	190.0			3.0	35.70	23.94	0.67
75	0.75	260.0			3.0	35.55	32.76	0.92
100	1.00	317.0	-85	-0.09	3.0	35.40	39.94	1.13
125	1.25	365.0			3.0	35.25	45.99	1.30
150	1.50	408.0			3.0	35.10	51.41	1.46
175	1.75	439.0			3.0	34.95	55.31	1.58
200	2.00	468.0	-92	-0.16	3.0	34.80	58.97	1.69
225	2.25	490.0			3.0	34.65	61.74	1.78
250	2.50	510.0			3.0	34.50	64.26	1.86
275	2.75	522.0			3.0	34.35	65.77	1.91
300	3.00	535.0	-91	-0.15	3.0	34.20	67.41	1.97
325	3.25	540.0			3.0	34.05	68.04	2.00
350	3.50	545.0			3.0	33.90	68.67	2.03
375	3.75	546.0			3.0	33.75	68.80	2.04
400	4.00	546.0	-84	-0.08	3.0	33.60	68.80	2.05
425	4.25	546.0			3.0	33.45	68.80	2.06
450	4.50	545.0			3.0	33.30	68.67	2.06
475	4.75	540.0			3.0	33.15	68.04	2.05
500	5.00	529.0	-80	-0.04	3.0	33.00	66.65	2.02
525	5.25	522.0			3.0	32.85	65.77	2.00
550	5.50	518.0			3.0	32.70	65.27	2.00
575	5.75	512.0			3.0	32.55	64.51	1.98
600	6.00	507.0	-80	-0.04	3.0	32.40	63.88	1.97
650	6.50	500.0			3.0	32.10	63.00	1.96
700	7.00	500.0	-85	-0.09	3.0	31.80	63.00	1.98
750	7.50	500.0			3.0	31.50	63.00	2.00
800	8.00	500.0	-92	-0.16	3.0	31.20	63.00	2.02
850	8.50	500.0			3.0	30.90	63.00	2.04
900	9.00	500.0	-92	-0.16	3.0	30.60	63.00	2.06
950	9.50	500.0			3.0	30.30	63.00	2.08
1000	10.00	500.0	-92	-0.16	3.0	30.00	63.00	2.10
1050	10.50	500.0			3.0	29.70	63.00	2.12
1100	11.00	500.0	-92	-0.16	3.0	29.40	63.00	2.14
1150	11.50	500.0			3.0	29.10	63.00	2.16
1200	12.00	500.0	-92	-0.16	3.0	28.80	63.00	2.19





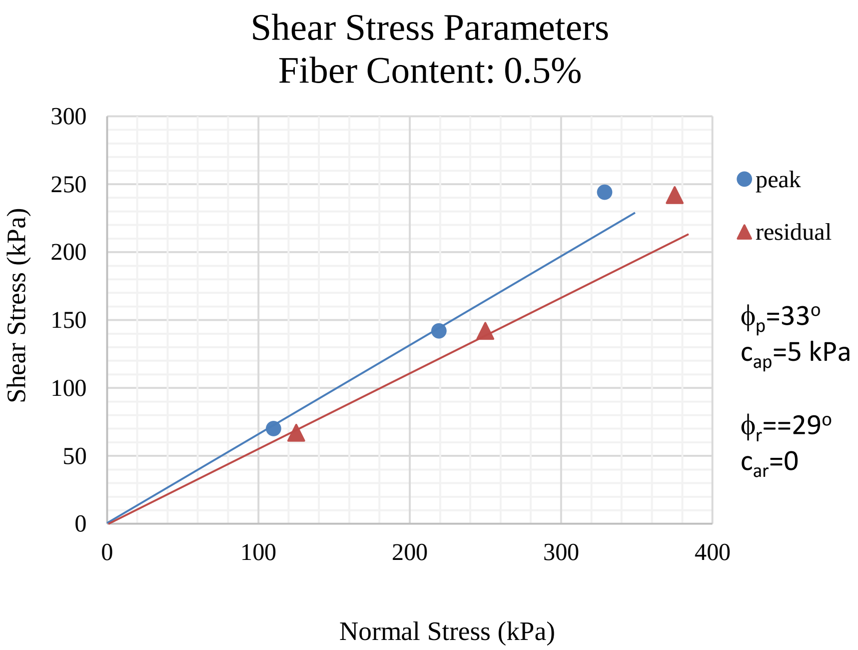


Figure A.7 : Test Results for Test No D-3

Table A.19 : Direct Shear Test Results- D-3-1

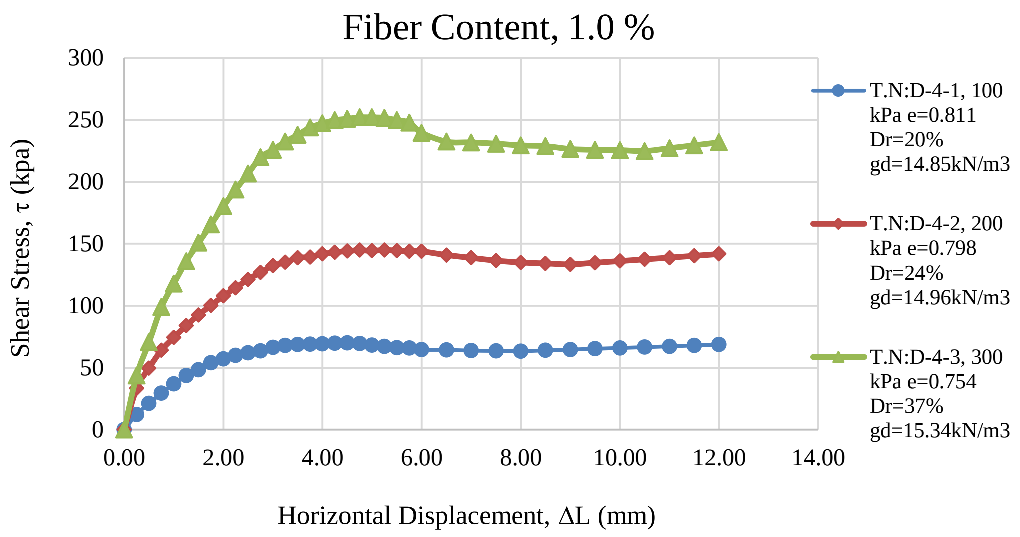
Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	1.0	36.00	0.00	0.00
25	0.25	55			1.0	35.85	6.93	0.19
50	0.50	80.0			1.0	35.70	10.08	0.28
75	0.75	96.0			1.0	35.55	12.10	0.34
100	1.00	111.0	-80	-0.15	1.0	35.40	13.99	0.40
125	1.25	122.0			1.0	35.25	15.37	0.44
150	1.50	135.0			1.0	35.10	17.01	0.48
175	1.75	145.0			1.0	34.95	18.27	0.52
200	2.00	153.0	-82	-0.17	1.0	34.80	19.28	0.55
225	2.25	158.0			1.0	34.65	19.91	0.57
250	2.50	163.0			1.0	34.50	20.54	0.60
275	2.75	167.0			1.0	34.35	21.04	0.61
300	3.00	170.0	-78	-0.13	1.0	34.20	21.42	0.63
325	3.25	171.0			1.0	34.05	21.55	0.63
350	3.50	173.0			1.0	33.90	21.80	0.64
375	3.75	173.0			1.0	33.75	21.80	0.65
400	4.00	173.0	-73	-0.08	1.0	33.60	21.80	0.65
425	4.25	173.0			1.0	33.45	21.80	0.65
450	4.50	174.0			1.0	33.30	21.92	0.66
475	4.75	174.0			1.0	33.15	21.92	0.66
500	5.00	174.0	-63	0.02	1.0	33.00	21.92	0.66
525	5.25	172.0			1.0	32.85	21.67	0.66
550	5.50	170.0			1.0	32.70	21.42	0.66
575	5.75	167.0			1.0	32.55	21.04	0.65
600	6.00	164.0	-61	0.04	1.0	32.40	20.66	0.64
650	6.50	162.0			1.0	32.10	20.41	0.64
700	7.00	159.0	-60	0.05	1.0	31.80	20.03	0.63
750	7.50	155.0			1.0	31.50	19.53	0.62
800	8.00	153.0	-60	0.05	1.0	31.20	19.28	0.62
850	8.50	153.0			1.0	30.90	19.28	0.62
900	9.00	153.0	-60	0.05	1.0	30.60	19.28	0.63
950	9.50	153.0			1.0	30.30	19.28	0.64
1000	10.00	153.0	-64	0.01	1.0	30.00	19.28	0.64
1050	10.50	153.0			1.0	29.70	19.28	0.65
1100	11.00	153.0	-67	-0.02	1.0	29.40	19.28	0.66
1150	11.50	153.0			1.0	29.10	19.28	0.66
1200	12.00	153.0	-72	-0.07	1.0	28.80	19.28	0.67

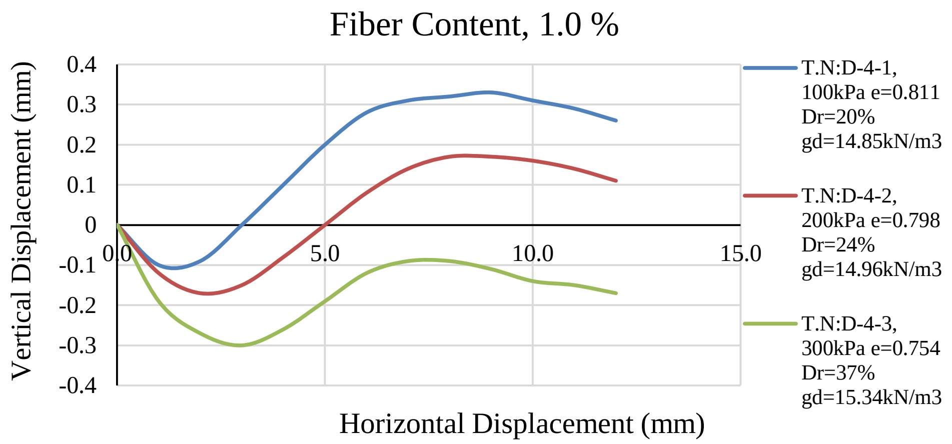
Table A.20 : Direct Shear Test Results- D-3-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	2.0	36.00	0.00	0.00
25	0.25	85			2.0	35.85	10.71	0.30
50	0.50	130.0			2.0	35.70	16.38	0.46
75	0.75	170.0			2.0	35.55	21.42	0.60
100	1.00	198.0	-85	-0.14	2.0	35.40	24.95	0.70
125	1.25	225.0			2.0	35.25	28.35	0.80
150	1.50	250.0			2.0	35.10	31.50	0.90
175	1.75	265.0			2.0	34.95	33.39	0.96
200	2.00	284.0	-90	-0.19	2.0	34.80	35.78	1.03
225	2.25	305.0			2.0	34.65	38.43	1.11
250	2.50	321.0			2.0	34.50	40.45	1.17
275	2.75	335.0			2.0	34.35	42.21	1.23
300	3.00	342.0	-87	-0.16	2.0	34.20	43.09	1.26
325	3.25	350.0			2.0	34.05	44.10	1.30
350	3.50	358.0			2.0	33.90	45.11	1.33
375	3.75	363.0			2.0	33.75	45.74	1.36
400	4.00	367.0	-81	-0.1	2.0	33.60	46.24	1.38
425	4.25	369.0			2.0	33.45	46.49	1.39
450	4.50	370.0			2.0	33.30	46.62	1.40
475	4.75	370.0			2.0	33.15	46.62	1.41
500	5.00	369.0	-73	-0.02	2.0	33.00	46.49	1.41
525	5.25	369.0			2.0	32.85	46.49	1.42
550	5.50	367.0			2.0	32.70	46.24	1.41
575	5.75	362.0			2.0	32.55	45.61	1.40
600	6.00	360.0	-68	0.03	2.0	32.40	45.36	1.40
650	6.50	352.0			2.0	32.10	44.35	1.38
700	7.00	345.0	-66	0.05	2.0	31.80	43.47	1.37
750	7.50	338.0			2.0	31.50	42.59	1.35
800	8.00	333.0	-68	0.03	2.0	31.20	41.96	1.34
850	8.50	329.0			2.0	30.90	41.45	1.34
900	9.00	324.0	-71	0	2.0	30.60	40.82	1.33
950	9.50	324.0			2.0	30.30	40.82	1.35
1000	10.00	324.0	-76	-0.05	2.0	30.00	40.82	1.36
1050	10.50	324.0			2.0	29.70	40.82	1.37
1100	11.00	324.0	-82	-0.11	2.0	29.40	40.82	1.39
1150	11.50	324.0			2.0	29.10	40.82	1.40
1200	12.00	324.0	-90	-0.19	2.0	28.80	40.82	1.42

Table A.21 : Direct Shear Test Results- D-3-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm²)	Düzeltilmiş Alan (A_L) (cm²)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm²)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	3.0	36.00	0.00	0.00
25	0.25	120			3.0	35.85	15.12	0.42
50	0.50	208.0			3.0	35.70	26.21	0.73
75	0.75	280.0			3.0	35.55	35.28	0.99
100	1.00	334.0	-87	-0.07	3.0	35.40	42.08	1.19
125	1.25	363.0			3.0	35.25	45.74	1.30
150	1.50	410.0			3.0	35.10	51.66	1.47
175	1.75	445.0			3.0	34.95	56.07	1.60
200	2.00	481.0	-90	-0.1	3.0	34.80	60.61	1.74
225	2.25	521.0			3.0	34.65	65.65	1.89
250	2.50	557.0			3.0	34.50	70.18	2.03
275	2.75	583.0			3.0	34.35	73.46	2.14
300	3.00	600.0	-85	-0.05	3.0	34.20	75.60	2.21
325	3.25	618.0			3.0	34.05	77.87	2.29
350	3.50	634.0			3.0	33.90	79.88	2.36
375	3.75	640.0			3.0	33.75	80.64	2.39
400	4.00	642.0	-74	0.06	3.0	33.60	80.89	2.41
425	4.25	644.0			3.0	33.45	81.14	2.43
450	4.50	644.0			3.0	33.30	81.14	2.44
475	4.75	644.0			3.0	33.15	81.14	2.45
500	5.00	642.0	-65	0.15	3.0	33.00	80.89	2.45
525	5.25	639.0			3.0	32.85	80.51	2.45
550	5.50	620.0			3.0	32.70	78.12	2.39
575	5.75	607.0			3.0	32.55	76.48	2.35
600	6.00	590.0	-48	0.32	3.0	32.40	74.34	2.29
650	6.50	565.0			3.0	32.10	71.19	2.22
700	7.00	554.0	-41	0.39	3.0	31.80	69.80	2.20
750	7.50	552.0			3.0	31.50	69.55	2.21
800	8.00	552.0	-41	0.39	3.0	31.20	69.55	2.23
850	8.50	552.0			3.0	30.90	69.55	2.25
900	9.00	552.0	-44	0.36	3.0	30.60	69.55	2.27
950	9.50	552.0			3.0	30.30	69.55	2.30
1000	10.00	552.0	-47	0.33	3.0	30.00	69.55	2.32
1050	10.50	552.0			3.0	29.70	69.55	2.34
1100	11.00	552.0	-50	0.3	3.0	29.40	69.55	2.37
1150	11.50	552.0			3.0	29.10	69.55	2.39
1200	12.00	552.0	-53	0.27	3.0	28.80	69.55	2.42





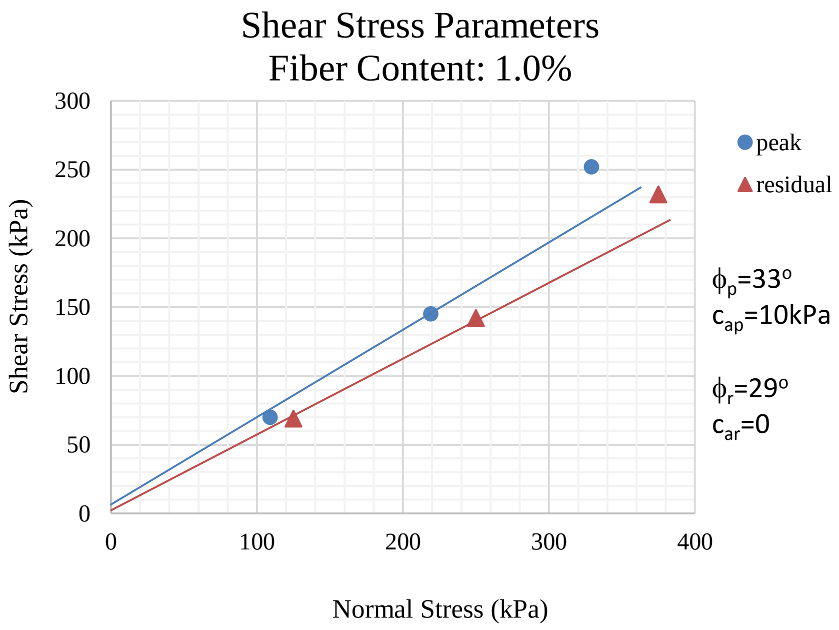


Figure A.8 : Test Results for Test No D-4.

Table A.22 : Direct Shear Test Results- D-4-1

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası Div	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	1.0	36.00	0.00	0.00
25	0.25	38			1.0	35.85	4.79	0.13
50	0.50	67.0			1.0	35.70	8.44	0.24
75	0.75	100.0			1.0	35.55	12.60	0.35
100	1.00	119.0	-53	-0.1	1.0	35.40	14.99	0.42
125	1.25	135.0			1.0	35.25	17.01	0.48
150	1.50	143.0			1.0	35.10	18.02	0.51
175	1.75	150.0			1.0	34.95	18.90	0.54
200	2.00	151.0	-50	-0.07	1.0	34.80	19.03	0.55
225	2.25	157.0			1.0	34.65	19.78	0.57
250	2.50	158.0			1.0	34.50	19.91	0.58
275	2.75	160.0			1.0	34.35	20.16	0.59
300	3.00	160.0	-44	-0.01	1.0	34.20	20.16	0.59
325	3.25	158.0			1.0	34.05	19.91	0.58
350	3.50	156.0			1.0	33.90	19.66	0.58
375	3.75	153.0			1.0	33.75	19.28	0.57
400	4.00	150.0	-40	0.03	1.0	33.60	18.90	0.56
425	4.25	145.0			1.0	33.45	18.27	0.55
450	4.50	142.0			1.0	33.30	17.89	0.54
475	4.75	140.0			1.0	33.15	17.64	0.53
500	5.00	140.0	-36	0.07	1.0	33.00	17.64	0.53
525	5.25	139.0			1.0	32.85	17.51	0.53
550	5.50	138.0			1.0	32.70	17.39	0.53
575	5.75	137.0			1.0	32.55	17.26	0.53
600	6.00	134.0	-36	0.07	1.0	32.40	16.88	0.52
650	6.50	131.0			1.0	32.10	16.51	0.51
700	7.00	127.0	-38	0.05	1.0	31.80	16.00	0.50
750	7.50	124.0			1.0	31.50	15.62	0.50
800	8.00	124.0	-41	0.02	1.0	31.20	15.62	0.50
850	8.50	124.0			1.0	30.90	15.62	0.51
900	9.00	124.0	-41	0.02	1.0	30.60	15.62	0.51
950	9.50	124.0			1.0	30.30	15.62	0.52
1000	10.00	124.0	-41	0.02	1.0	30.00	15.62	0.52
1050	10.50	124.0			1.0	29.70	15.62	0.53
1100	11.00	124.0	-41	0.02	1.0	29.40	15.62	0.53
1150	11.50	124.0			1.0	29.10	15.62	0.54
1200	12.00	124.0	-41	0.02	1.0	28.80	15.62	0.54

Table A.23 : Direct Shear Test Results- D-4-2

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL 1/100 mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100) mm	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	2.0	36.00	0.00	0.00
25	0.25	95			2.0	35.85	11.97	0.33
50	0.50	141.0			2.0	35.70	17.77	0.50
75	0.75	181.0			2.0	35.55	22.81	0.64
100	1.00	209.0	-87	-0.12	2.0	35.40	26.33	0.74
125	1.25	235.0			2.0	35.25	29.61	0.84
150	1.50	258.0			2.0	35.10	32.51	0.93
175	1.75	278.0			2.0	34.95	35.03	1.00
200	2.00	298.0	-92	-0.17	2.0	34.80	37.55	1.08
225	2.25	315.0			2.0	34.65	39.69	1.15
250	2.50	332.0			2.0	34.50	41.83	1.21
275	2.75	346.0			2.0	34.35	43.60	1.27
300	3.00	359.0	-90	-0.15	2.0	34.20	45.23	1.32
325	3.25	365.0			2.0	34.05	45.99	1.35
350	3.50	373.0			2.0	33.90	47.00	1.39
375	3.75	373.0			2.0	33.75	47.00	1.39
400	4.00	378.0	-83	-0.08	2.0	33.60	47.63	1.42
425	4.25	380.0			2.0	33.45	47.88	1.43
450	4.50	381.0			2.0	33.30	48.01	1.44
475	4.75	381.0			2.0	33.15	48.01	1.45
500	5.00	378.0	-75	0	2.0	33.00	47.63	1.44
525	5.25	378.0			2.0	32.85	47.63	1.45
550	5.50	375.0			2.0	32.70	47.25	1.44
575	5.75	372.0			2.0	32.55	46.87	1.44
600	6.00	370.0	-67	0.08	2.0	32.40	46.62	1.44
650	6.50	359.0			2.0	32.10	45.23	1.41
700	7.00	350.0	-62	0.13	2.0	31.80	44.10	1.39
750	7.50	341.0			2.0	31.50	42.97	1.36
800	8.00	334.0	-58	0.17	2.0	31.20	42.08	1.35
850	8.50	329.0			2.0	30.90	41.45	1.34
900	9.00	324.0	-58	0.17	2.0	30.60	40.82	1.33
950	9.50	324.0			2.0	30.30	40.82	1.35
1000	10.00	324.0	-58	0.17	2.0	30.00	40.82	1.36
1050	10.50	324.0			2.0	29.70	40.82	1.37
1100	11.00	324.0	-58	0.17	2.0	29.40	40.82	1.39
1150	11.50	324.0			2.0	29.10	40.82	1.40
1200	12.00	324.0	-58	0.17	2.0	28.80	40.82	1.42

Table A.24 : Direct Shear Test Results- D-4-3

Yatay Deplasman ΔL 1/100 mm	Yatay Deplasman ΔL mm	Kuvvet Halkası	Düşey Deplasman Δh (1/100)	Düşey Deplasman Δh (mm)	Normal Yük, σn (kg/cm^2)	Düzeltilmiş Alan (A_L) (cm^2)	Kesme Kuvveti (S) (kg)	Kayma Gerilmesi, τ (kg/cm^2)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	0.00	0.0	0	0	3.0	36.00	0.00	0.00
25	0.25	124			3.0	35.85	15.62	0.44
50	0.50	200.0			3.0	35.70	25.20	0.71
75	0.75	279.0			3.0	35.55	35.15	0.99
100	1.00	331.0	-160	-0.19	3.0	35.40	41.71	1.18
125	1.25	380.0			3.0	35.25	47.88	1.36
150	1.50	420.0			3.0	35.10	52.92	1.51
175	1.75	459.0			3.0	34.95	57.83	1.65
200	2.00	498.0	-168	-0.27	3.0	34.80	62.75	1.80
225	2.25	533.0			3.0	34.65	67.16	1.94
250	2.50	566.0			3.0	34.50	71.32	2.07
275	2.75	599.0			3.0	34.35	75.47	2.20
300	3.00	613.0	-171	-0.3	3.0	34.20	77.24	2.26
325	3.25	628.0			3.0	34.05	79.13	2.32
350	3.50	640.0			3.0	33.90	80.64	2.38
375	3.75	653.0			3.0	33.75	82.28	2.44
400	4.00	659.0	-167	-0.26	3.0	33.60	83.03	2.47
425	4.25	663.0			3.0	33.45	83.54	2.50
450	4.50	663.0			3.0	33.30	83.54	2.51
475	4.75	663.0			3.0	33.15	83.54	2.52
500	5.00	660.0	-160	-0.19	3.0	33.00	83.16	2.52
525	5.25	656.0			3.0	32.85	82.66	2.52
550	5.50	648.0			3.0	32.70	81.65	2.50
575	5.75	640.0			3.0	32.55	80.64	2.48
600	6.00	616.0	-153	-0.12	3.0	32.40	77.62	2.40
650	6.50	592.0			3.0	32.10	74.59	2.32
700	7.00	585.0	-150	-0.09	3.0	31.80	73.71	2.32
750	7.50	577.0			3.0	31.50	72.70	2.31
800	8.00	568.0	-150	-0.09	3.0	31.20	71.57	2.29
850	8.50	561.0			3.0	30.90	70.69	2.29
900	9.00	550.0	-150	-0.09	3.0	30.60	69.30	2.26
950	9.50	543.0			3.0	30.30	68.42	2.26
1000	10.00	537.0	-150	-0.09	3.0	30.00	67.66	2.26
1050	10.50	530.0			3.0	29.70	66.78	2.25
1100	11.00	530.0	-150	-0.09	3.0	29.40	66.78	2.27
1150	11.50	530.0			3.0	29.10	66.78	2.29
1200	12.00	530.0	-150	-0.09	3.0	28.80	66.78	2.32

APPENDIX B

Table B.1 : Permeability Results

		Fiber Content (%)			
		0.0	0.1	0.5	1.0
Permeability (m/sec)	1st	2.229×10^{-4}	3.277×10^{-4}	2.857×10^{-4}	2.371×10^{-4}
	2nd	2.371×10^{-4}	3.095×10^{-4}	2.857×10^{-4}	2.321×10^{-4}
	3rd	2.321×10^{-4}	3.012×10^{-4}	2.857×10^{-4}	2.422×10^{-4}

Table B.2 : Permeability Test Results for 0.0% Fiber Content Samples

Fiber Content (%)	0	0	0
Test Number	0-B	0.C	0.D
Gs	2.69	2.69	2.69
e max	0.876	0.876	0.876
e min	0.547	0.547	0.547
e	0.695	0.695	0.695
Dr (%)	55	55	55
γ_{dry} (gr/cm^3)	1.587	1.587	1.587
Ws (gr)	3196	3196	3196
Wf (gr)	0	0	0
Ws+Wf (gr)	3196	3196	3196
Vmold (cm^3)	2014.4	2014.4	2014.4
Settlement (cm)	1.5	0.35	0.3
Vnew (cm^3)	1924.2	1993.4	1996.4
$\gamma_{dry\ new}$ (gr/cm^3)	1.661	1.603	1.601
e new	0.620	0.678	0.680
n new	0.383	0.404	0.405
New Dr (%)	78	60	59
Q (m/sec)	200	200	200
i	2.985	2.985	2.985
A(cm2)	60.13	60.13	60.13
t (sec)	50	47	48
K(m/sec)	0.00022286	0.00023708	0.00023214

Table B.3 : Permeability Tets Results for 0.1% Fiber Content Samples

permeability			
Fiber Content (%)	*0.1*	*0.1*	*0.1*
Test Number	*0.1-A*	*0.1-B*	*0.1-C*
Gs	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*
e	*0.695*	*0.695*	*0.695*
Dr (%)	*55*	*55*	*55*
γ_{dry} *(gr/cm³)*	*1.587*	*1.587*	*1.587*
Ws (gr)	*3193*	*3193*	*3193*
Wf (gr)	*3*	*3*	*3*
Ws+Wf (gr)	*3196*	*3196*	*3196*
Vmold (cm³)	*2014.4*	*2014.4*	*2014.4*
Settlement (cm)	*0.2*	*0.25*	*0.2*
Vnew (cm³)	*2002.4*	*1999.4*	*2002.4*
$\gamma_{dry\ new}$ *(gr/cm³)*	*1.596*	*1.598*	*1.596*
e new	*0.685*	*0.683*	*0.685*
n new	*0.407*	*0.406*	*0.407*
New Dr (%)	*58*	*59*	*58*
Q (m/sec)	200	200	200
i	2.985	2.985	2.985
A(cm2)	60.13	60.13	60.13
t (sec)	34	36	37
K(m/sec)	0.00032773	0.00030952	0.00030116

Table B.4 : Permeability Tets Results for 0.5% Fiber Content Samples

permeability			
Fiber Content (%)	*0.5*	*0.5*	*0.5*
Test Number	*0.5-A*	*0.5-B*	*0.5-C*
Gs	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*
e	*0.695*	*0.695*	*0.695*
Dr (%)	*55*	*55*	*55*
γ_{dry} *(gr/cm^3)*	*1.587*	*1.587*	*1.587*
Ws (gr)	*3180*	*3180*	*3180*
Wf (gr)	*16*	*16*	*16*
Ws+Wf (gr)	*3196*	*3196*	*3196*
Vmold (cm^3)	*2014.4*	*2014.4*	*2014.4*
Settlement (cm)	*0.1*	*0.1*	*0.1*
Vnew (cm^3)	*2008.4*	*2008.4*	*2008.4*
$\gamma_{dry\ new}$ *(gr/cm^3)*	*1.591*	*1.591*	*1.591*
e new	*0.690*	*0.690*	*0.690*
n new	*0.408*	*0.408*	*0.408*
New Dr (%)	*56*	*56*	*56*
Q (m/sec)	200	200	200
i	2.985	2.985	2.985
A(cm2)	60.13	60.13	60.13
t (sec)	39	39	39
K(m/sec)	0.00028571	0.00028571	0.00028571

Table B.5 : Permeability Tets Results for 1.0% Fiber Content Samples

permeability			
Fiber Content (%)	*1*	*1*	*1*
Test Number	*1-A*	*1-B*	*1-C*
Gs	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*
e	*0.695*	*0.695*	*0.695*
Dr (%)	*55*	*55*	*55*
γ_{dry} *(gr/cm³)*	*1.587*	*1.587*	*1.587*
Ws (gr)	*3164*	*3164*	*3164*
Wf (gr)	*32*	*32*	*32*
Ws+Wf (gr)	*3196*	*3196*	*3196*
Vmold (cm³)	*2014.4*	*2014.4*	*2014.4*
Settlement (cm)	*0*	*0*	*0*
Vnew (cm³)	*2014.4*	*2014.4*	*2014.4*
$\gamma_{dry\ new}$ *(gr/cm³)*	*1.587*	*1.587*	*1.587*
e new	*0.695*	*0.695*	*0.695*
n new	*0.410*	*0.410*	*0.410*
New Dr (%)	*55*	*55*	*55*
Q (m/sec)	200	200	200
i	2.985	2.985	2.985
A(cm2)	60.13	60.13	60.13
t (sec)	47	48	46
K(m/sec)	0.00023708	0.00023214	0.00024223

Table B.6 : Permeability Test Results Test No: W-1

Vacuum and CO2 Application (0.03MPa Vacuum and 3 bar CO2)				
Fiber Content (%)	***0***	***0.1***	***0.5***	***1***
Gs	*2.69*	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*	*0.547*
Dr (%)	*55*	*55*	*55*	*55*
e	*0.695*	*0.695*	*0.695*	*0.695*
Ws (gr)	*1445*	*1445*	*1445*	*1445*
Wf (gr)	*0*	*1.45*	*7.23*	*14.45*
Ws+Wf (gr)	*1445*	*1446.45*	*1458.2*	*1459.5*
***$W_{\omega t}$** (gr)*	*422*	*423*	*406*	*398*
W_{ω} sat	359.16	368.02	398.15	390.15
γ_{dry} (gr/cm^3)	*1.585*	*1.585*	*1.585*	*1.585*
Vmold (cm^3)	*919.035*	*919.035*	*919.035*	*919.035*
Settlement (cm)	*0.8*	*0.7*	*0.1*	*0.1*
Vnew (cm^3)	*856.195*	*864.05*	*911.18*	*911.18*
W_{ω} top	62.84	54.985	7.855	7.855
$\gamma_{dry\ new}$ (gr/cm^3)	*1.688*	*1.674*	*1.600*	*1.602*
e new	*0.594*	*0.607*	*0.681*	*0.679*
n new	*0.373*	*0.378*	*0.405*	*0.405*
New Dr (%)	*86*	*82*	*59*	*60*

Table B.7 : Permeability Test Results Test No: W-2

Non Vacum Application				
Fiber Content (%)	***0***	***0.1***	***0.5***	***1***
Gs	*2.69*	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*	*0.547*
Dr (%)	*55*	*55*	*55*	*55*
e	*0.695*	*0.695*	*0.695*	*0.695*
Ws (gr)	*1445*	*1445*	*1445*	*1445*
Wf (gr)	*0*	*1.45*	*7.23*	*14.45*
Ws+Wf (gr)	*1445*	*1446.45*	*1458.2*	*1459.5*
Wωt (gr)	*418*	*411*	*393*	*388*
Wω sat	*371*	*361*	*385*	*385*
γ_{dry} (gr/cm^3)	*1.572*	*1.574*	*1.587*	*1.588*
Vmold (cm^3)	*919.035*	*919.035*	*919.035*	*919.035*
Settlement (cm)	*0.6*	*0.6*	*0.1*	*0.1*
Vnew (cm^3)	*871.905*	*871.905*	*911.18*	*911.18*
Wω top	47.13	47.13	7.855	7.855
$\gamma_{dry\ new}$ (gr/cm^3)	*1.657*	*1.659*	*1.600*	*1.602*
e new	*0.623*	*0.622*	*0.681*	*0.679*
n new	*0.384*	*0.383*	*0.405*	*0.405*
New Dr (%)	*77*	*77*	*59*	*60*

Table B.8 : Permeability Test Results Test No: W-3

Vacuum Application- Dr 60%				
Fiber Content (%)	***0***	***0.1***	***0.5-C***	***1***
Gs	*2.69*	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*	*0.547*
e	*0.694*	*0.694*	*0.766*	*0.741*
Dr (%)	*55*	*55*	*34*	*41*
γ_{dry} ***(gr/cm³)***	*1.588*	*1.588*	*1.526*	*1.545*
Ws (gr)	*1459.4*	*1458*	*1395*	*1406*
Wf (gr)	*0*	*1.4*	*7*	*14*
Ws+Wf (gr)	*1459.4*	*1459.4*	*1402*	*1420*
Wω (gr)	*424*	*419*	*432*	*428*
Wω sat	395.50	392.53	373.09	380.87
Vmold (cm³)	*919*	*919*	*919*	*919*
Settlement (cm)	*0.364*	*0.337*	*0.75*	*0.6*
Vnew (cm³)	*890.4078*	*892.5287*	*860.0875*	*871.87*
Wω top	28.5	26.47135	58.9125	47.13
$\gamma_{dry\ new}$ ***(gr/cm³)***	*1.639*	*1.635*	*1.630*	*1.629*
e new	*0.641*	*0.645*	*0.650*	*0.652*
n new	*0.391*	*0.392*	*0.394*	*0.395*
New Dr (%)	*71*	*70*	*69*	*68*
γ_{sat} ***(gr/cm³)***	*2.03*	*2.03*	*2.02*	*2.02*
γ ***(gr/cm³)***	*2.12*	*2.11*	*2.03*	*2.03*
V_{sat}/V_{tot}	0.9328	0.9368	0.8636	0.8899
V_{pur}/V_{sat}	1	1.00757	1.060073	1.165752
V_{sat}/V_{pur}	1	0.992487	0.943331	0.857815

Table B.9 : Permeability Test Results Test No: W-4

Vacuum Application- Dr 70%			
Fiber Content (%)	***0-B***	***0.5***	***1***
Gs	*2.69*	*2.69*	*2.69*
e max	*0.876*	*0.876*	*0.876*
e min	*0.547*	*0.547*	*0.547*
e	*0.711*	*0.694*	*0.694*
Dr (%)	*50*	*55*	*55*
γ_{dry} (gr/cm^3)	*1.567*	*1.588*	*1.588*
Ws (gr)	*1440*	*1452*	*1445*
Wf (gr)	*0*	*7.4*	*14.4*
Ws+Wf (gr)	*1440*	*1459.4*	*1459.4*
Wω (gr)	*471*	*400*	*399*
Wω sat	*444*	392.15	391.15
Vmold (cm^3)	*919*	*919*	*919*
Settlement (cm)	*0.3*	*0.1*	*0.1*
Vnew (cm^3)	*895.435*	*911.145*	*911.145*
Wω top	27	7.855	7.855
$\gamma_{dry\ new}$ (gr/cm^3)	*1.608*	*1.602*	*1.602*
e new	*0.673*	*0.679*	*0.679*
n new	*0.402*	*0.405*	*0.405*
New Dr (%)	*62*	*60*	*60*
γ_{sat} (gr/cm^3)	*2.01*	*2.01*	*2.01*
γ(gr/cm^3)	*2.10*	*2.04*	*2.03*
V_{sat}/V_{tot}	0.9427	0.9804	0.9803
V_{pur}/V_{sat}	1	1.132234	1.135129
V_{sat}/V_{pur}	1	0.883209	0.880957

CURRICULUM VITAE

Name Surname: Ahmad Darvishi

Place and Date of Birth: Lalecin 0.1.11.1981

Address: Maçka, Beşiktaş, İSTANBUL

E-Mail: ahmetdervisi@gmail.com

B.Sc.: Islamic Azad University of Tehran- South Branch