HASAN KALYONCU UNIVERSITY
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EFFECT OF SAND COLUMN ON COMPRESSIBILITY AND SHEAR STRENGTH OF FIBROUS PEAT

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Effect of Sand Column on Compressibility and Shear Strength of Fibrous Peat

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Hozan K. YABA

ABSTRACT

EFFECT OF SAND COLUMN ON COMPRESSIBILITY AND SHEAR STRENGTH OF FIBROUS PEAT

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M. Sc. In Civil Engineering

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Peat is found in many countries throughout the world where it can be generally seen in thick layers in limited areas. Peat is an extreme form of soft soil and is considered problematic due to the low shear strength and large compressibility. This project presents laboratory finding on the compressibility and shear strength characteristics of fibrous peat with sand column. The peat used in the study is taken from Sakarya region, Turkey. It is classified as fibrous peat according to ASTM D 1997-91 and due to its low to medium degree of decomposition classified as H_1-H_4 in von Post scale. The natural water content of the peat is 236 % and its liquid limit is 119 %. In all tests, the fibrous peat used for the test passing #200 and remain on #100. The rounded sand used for making sand column is poorly graded passing from 2 mm sieve size and retaining on 0.075 mm sieve size. The tests focused on effect of diameter of granular column on shear strength and compressibility of the organic soil. Four different sand column diameters were used for compressibility and shear strength tests were (1.7, 2.5, 3.5and 4.7cm). All tests results showed that; when the ratio of sand column surface to organic soil surface area (S/O) increases; compression index (c_c); recompression index (c_r); and volume compressibility (m_v) decreases. Also, sand columns causes increase in (IFA) and reduction in (C) of the organic soil.

Key Words: Fibrous peat, rounded sand, consolidation of peat, shear strength of peat, soil improvement, classification of peats.

ÖZET

KUM KOLONUN FİBERLİ PEAT'LERİN KESME DAYANIMI VE SIKIŞABİLİRLİĞİ ÜZERİNE ETKİSİ

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Dünyanın bir çok ülkesinde bulunan peat, genellikle ince tabakalar halinde ve sınırlı alanlarda görülür. Peat yumuşak zeminlerin extrim halde ki formudur ve bundan dolayı düşük kesme dayanımı ve yüksek oranda oturma problemleri gösterir. Bu çalışma da içine kum kolon yerleştirilmiş fiberli peat'in kesme dayanımı ve oturma karakteristiklerinin laboratuvar bulguları sunulmuştur. Bu çalışmada kullanılan peat Türkiye'nin Sakarya bölgesinden alınmıştır. ASTM D 1997-91'a göre bu zemin fiberli peat olarak sınıflandırılmış ve Von post ölçeğinde düşük ve orta derecede çürümüş H_1-H_4 olarak tanımlanmıştır. Bu zeminin tabii su muhtevası % 236 dır ve likit limiti % 119 dur. Bütün deneylerde 200 nolu elekten geçen ve 100 nolu elek üzerinde kalan peat kullanılmıştır. Kum kolon yapımında iyi derecelenmemiş 2 mm elek çapından geçen 0,075 mm elek çapı üzerinde kalan dere kumu kullanılmıştır. Tüm deneylerde kun kolon çapının organik zeminin kesme dayınımı ve sıkaşabilirliği üzerinde ki etkisi üzerine odaklanılmıştır. Sıkışabilirlik ve kesme dayanımı deneyleri için dört farklı çapta kum kolonlar hazırlanmıştır (1.7, 2.5, 3.5 and 4.7cm)..tüm deney sonuçları göstermiştir ki; kum kolon yüzey alanı oranının organik zemin yüzey alanına oranı (S/O) arttığında , sıkışma indeksleri C_c,C_r ve hacimsel sıkışma (m_v) azalmıştır. Ayrıca kum kolonlar organik zeminin içsel sürtünme açısını artırırken kohezyonu ise azaltmıştır.

Anahtar Kelimeler: Fiberli peat, Dere kumu , peat'in konsolidasyonu, Peat'in kesme dayanımı, Zemini iyileştirme, Peat'lerin sınıflandırılması.

TO MY BELOVED FAMILY

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I would like to deeply praise the **God** for allowing me passing all of this moment.

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LIST OF SYMBOLS/ABBREVIATIONS

A	Cross sectional area of specimen in shear box test
a	Primary compressibility (based on Rheological model)
AC	Ash content
a_v	Coefficient of axial compressibility, coefficient of volume Compressibility
b	Coefficient of secondary compressibility (based on Rheological model)
c	Cohesion value of soil
c_c	Compression index
CD	Consolidated drained test
Cf	Function of clay content
c_r	Recompression index
c_u	Undrained shear strength
CU	Consolidated undrained
c_α	Rate of secondary compression; slop, coefficient of secondary compression
d	Diameter of sample
D_r	Relative density
e	Overall void ratio of soil
e_0	Initial void ratio
e_{cv}	Critical void ratio
e_{op}	Void ratio at the beginning of secondary compression

FC	Fiber content
G_s	Specific gravity
h_0	Initial height
I_p	Plasticity index
LIR	Load increment ratio
LL	Liquid limit
m	Secondary compression factor
m_v	Coefficient of volume compressibility
N	Normal load
OC	Organic content
p_h	Horizontal load
PL	Plastic limit
p_v	Vertical load
r	Radius of sample
s_s	Consolidation settlement
s_c	Secondary compression
s_u	Undrained shear strength
t	Time
t_0	Beginning of secondary compression
t_p	Beginning of secondary compression; end of primary consolidation; time of primary consolidation
t_f	Time for the secondary compression settlement
UU	Unconsolidated undrained
μ_e	Excess pore water pressure
ω_0; ω	Natural water content

X	Difference in the dial reading
Δe	Change of void ratio from t_p to t_f
ΔH	Consolidation settlement
ΔV	Change in volume
Δp	Pressure difference
γ	Unit weight
γ_w	Unit weight of water
σ	Effective stress
σ'_v	Effective vertical stress
σ'_p	In-situ effective vertical stress
ϕ	Internal friction angle of soil
ϕ_u	Undrained internal friction angle of soil
τ	Shear strength of soil
τ_f	Shear strength at failure of soil
τ_u	Undrained shear stress
ε	Strain
ε_v	Vertical strain
δ	Total compression
δ_p	Primary consolidation settlement
δ_s	Secondary compression
ψ	Angle of dilation

CHAPTER I

INTRODUCTION

1.1 Background

In civil engineering, there are many problems linked with construction on soft soil especially associated with construction on soft soil especially peat. The main construction problems related to structure on peat are large compressibility and low shear strength, especially, because of low dry density high water content and low shear strength occurs in organic soil exceptionally. In addition, since the decomposition is still going on in organic soil, any structure of the stability constructed on peat soil could be affected by the mostly change of peat soil with time. Therefore, the construction build on peat deposit may cause excessive settlement and bearing capacity failure. Because of the low bearing capacity and hence the low shear strength, a surface foundation must be improved with respect to peat soil before construction works can begin. Suitable solution could be thought as replacing the poor soil by suitable soil using for fill. However this application may be very expensive. In addition, since waste excavated soil can be removed within an economically acceptable haul distance has to be needed (Jarret, 1997).This method also need maintenance work with respect to horizontal removing and long term consolidation (Magnan, 1994).

Approaches have been developed to address the problems associated with construction over peat deposits (Lea and Browner, 1963; Berry, 1983; Hansbo, 1991). There are alternative construction and stabilization methods such as surface reinforcement, preloading, chemical stabilization, sand or stone column, pre-fabricated vertical drains, and the use of piles. The selection of the most appropriate method should be based on the examination of the index and engineering characteristics of the soil. The knowledge on the shear strength and compression behavior is essential as it enables designers to understand the response of the soil to load and to suggest proper engineering solutions to overcome the problem.

Peat is found in many countries throughout the world. In the US peat is found in 42 states with a total acreage of 30 million hectares. Canada and Russia are the two countries with a large area of peat, 170 and 150 million hectares respectively (Hartlen and Wolski 1996). In Malaysia, some 3 million hectares of land is covered with peat (Hobbs 1986). While Turkey has limited areas of peat land (56,000 ha, TUSIAD 2009). There are two types of peat deposit; the shallow deposit usually is less than 3 m thick while the thickness of deep peat exceeds 5 m. The underlying materials is usually consists of marine clay (Muttalib et al., 1991).

In general, peat is grouped into two categories; amorphous peat and fibrous peat. Amorphous peat is the peat soil with fiber content less than 20 % (ASTM D4427). It contains mostly particles of colloidal size (less than 2 microns), and the pore water is absorbed around the particle surface. The compressibility behavior of the amorphous peat is known to be similar with clay soil which can be evaluated based on Terzaghi’s theory of consolidation. Fibrous peat is peat with high organic and fiber content with low degree of humification. The behavior of fibrous peat is different from mineral soil because of different phase properties and microstructure (Edil, 2003), thus Terzaghi’s theory of consolidation cannot be applied to predict the compression behavior of fibrous peat. Generally the peats are fibrous at shallow depth and become amorphous as they extend to some 8 m depth.

Fiber orientation is identified as a dominant factor in the structure of fibrous peat. The application of consolidation pressure may induce a rearrangement of fiber orientation and drastically reduces the void, causing a significant reduction in the vertical permeability. Moreover, fiber content appears to be a major compositional factor in determining the way in which peat soils behave (Dhowian and Edil, 1980). The higher the fiber content, the more the peat will differ from an inorganic soil in its behavior. In order to develop a visual appreciation of the fiber content and orientation, the microstructure of the peat was examined under a Scanning Electron Microscope (SEM).

Many researchers (Berry and Poskitt, 1972; Ajlouni, 2000; Robinson, 2003) have examined fibrous peat from different parts of the world and their findings are quite different from one another due to different content of peat soils. The properties of peat soils such as natural water content, acidity, degree of humification, fiber content, shear strength, and compressibility are affected by the formation of peat deposit. This indicates that in term of content, fibrous peat is different from one location to another location and detailed soil investigations need to be conducted for fibrous peat at a particular site where a building is intended to be constructed. The difference becomes particularly apparent especially under low vertical stresses or shallow depth. Thus, assessment on the response of peat deposit to loading should be made before any construction has to take place at a particular site.

Most of the methods to predict compressibility characteristics of soil are developed based on the results of laboratory consolidation test. Several test methods have been used to study the compressibility of different type of soil including peat. The oldest and the most popular one is the conventional Oedometer test. This test is still used as a standard consolidation test method in Turkey as well as in many parts of the world.

The compression behavior of fibrous peat consists of two phases i.e.: primary consolidation and secondary compression. The primary consolidation of fibrous peat is much larger than that of other soils due to high initial water content, while the secondary compression occurs due to not only compression of solid particles, but also the plastic yielding (buckling, bending, and squeezing) of the particles (Samson and La Rochelle, 1972). The magnitude of secondary compression takes more significant part of the compression of peat and plays an important role in determining the total settlement of the peat because the secondary compression occurs during the design life of a structure after the rapid primary consolidation. Tertiary compression was reported by several researchers (e.g. Candler and Chartres, 1988; Fox et al., 1992; Mitchell, 1993), but other researchers (e.g. Edil and Dhowian, 1979; Hansbo, 1991; Fox and Edil, 1994) argued that this part of compression can be neglected because it generally started after the design life of structure.

The method used to assess the shear strength of peat is not well defined yet. For a fibrous peat, the shear strength can be determined in laboratory by the direct shear test which is undrained test. Most peat is considered frictional or non-cohesive material (Adam, 1965) due to the fiber content, thus the shear strength of peat is determined based on drained condition as: $\tau_f = \sigma_n \tan \phi$. Direct shear and triaxial have been used to determine the shear strength of peat soil although the results of triaxial test on fibrous peat are difficult to interpret because fiber often act as horizontal reinforcement, so failure is seldom obtained in a drained test. In addition, triaxial test in drained condition may take several weeks for peat with low permeability.

Based on his study, Magnan (1994) suggested a ratio of shear strength increase due to increase in overburden pressure of 0.5 for peat soil. Furthermore, Edil and Wang (2000) collected normalized undrained strength ($\mathrm{c_u}/\sigma'_{3c}$ or $\mathrm{c_u}/\sigma'_{1c}$) as a function of organic content for all peat and organic soil.

1.2 Objective of Study

The main objectives of this study are given as follows:

1) To determine engineering properties of the fibrous peat soil collected from Sakarya region, Turkey.
2) To find out the effect of different sand column diameter on the shear strength properties of the fibrous peat soil used in the study.
3) To determine the effect of different sand column diameter on the consolidation parameters of the fibrous peat soil used in the experiments.

1.3 Scope of Project

The study focuses on the effect of different sand column diameter on peat soil found in. Sakarya region, Turkey. Therefore, the interpretation of the results of the study was limited as indicated in the followings:

1. Peat soil found in. Sakarya region, Turkey.
2. Identification of index properties of soil including: specific gravity, acidity, sieve analysis, and water content.
3. Classification of peat was made based on degree of humification (von Post) as well as the fiber and organic content.
4. Evaluation of shear strength of the peat was made by direcr shear box tests (laboratory).
5. The use of the standard consolidation test (Oedometer) data to determine the range of settlement was made on a hypothetical problem.

1.4 Significance of Study

This research will enrich the knowledge on the characteristics of peat soil and understanding behavior of peat with sand column the results will be used in the development of suitable soil improvement for fibrous peat. As foundation as well as construction material.

1.5 Thesis Structure

The thesis is composed of five chapters. Chapter 1 presents general information regarding background, objectives, scope, and significance of the study, and thesis structure. Chapter 2 provides the background of the study on different topics related to the research, this chapter outlines information on the general characteristics , shear strength properties and consolidation properties of fibrous peats were given from literature review. Chapter 3 provides the overall experimental program including materials properties and laboratory tests; shear strength and consolidation properties of fibrous peats were discussed. Chapter 4 presents and discusses the results of the experimental studies. Chapter 5 presents the conclusions of major findings of this research.

CHAPTER II

LITERATURE REVIEW

2.1 Peat Soil

2.1.1 Definition

The precise definition of peat varies between soil science and engineering .as well as between countries according to the soil scientist. Peat is soil with organic content greater than 35 percent. Whilst for a geotechnical engineer all soils with organic content greater than 20 percent is known as organic soil. Based on his research (Huat. 2004) Peat is an organic soil with organic content of more than 75 percent.

The engineering definition is basically based on the mechanical properties of the soil according to the Organic Sediments Research Centre (OSRC), University of South Carolina (1989), peat is defined according to the ash content in the soil. Peat has 25 percent or less inorganic content in the condition of dry weight. Under the Unified Soil Classification System (UCS), organic soils are recognized as a separate soil entity and have a major division called Highly Organic Soil (Pt), which refers to peat, muck and highly organic soils.

Generally peat soil is defined as a mixture of fragmented organic material formed in wetlands under appropriate topographic and climatic conditions and it comes from vegetation that was chemically decomposed and fossilized (Edil and Dhowian, 1980). Peat totally or partially changes remains of dead plants which were accumulated under water for many years. Peat can be generally seen in thick layers in limited areas, has high compressive deformation and low shear strength which often causes some difficulties when construction work is doing on the deposit (Anggraini,2006).

Peat soil usually contains organic material with normal depth of 0.5 meter. Peat is known for its high organic content which exceeds 75 percent. The organic contents of peat are basically the plant remains for which rate of decay is slower than the rate of accumulation. The content of peat soil differs from a location to other location due to factors such as humidity, temperature and the origin of fiber. Decomposition involves the loss of organic matter either in solution or in gas, the vanishing of physical structure and the change in chemical state (Huat, 2004).

Peat is usually found as an extremely loose, wet, and unconsolidated surface deposit which forms as an integral part of a wetland system, therefore access to the peat deposit is usually very difficult as the water table exists at, near, or above the ground surface. (Edil and Dhowian, 1979) ; (Edil and Dhowian, 1981) have found that the behavior of amorphous peat is similar to clay soil, thus evaluation of its compressibility characteristics can be made based on Terzaghi one-dimensional theory of consolidation.

Fibrous peat is the one that consists of fiber content more than 20 % (ASTM D4427). The behavior of fibrous peat is very different from clay due to the existence of the fiber in the soil. The fibrous peat has many void spaces existing between the solid grains. Due to the irregular shape of individual particles, fibrous peat deposits are porous and the soil is considered as a permeable material. Therefore the rate of consolidation of fibrous peat is high but the rate decreases significantly due to consolidation.

2.1.2 Structural Arrangement

The structural arrangement or texture of peat highly influences its engineering properties. The different textures are woody, fibrous, and granular amorphous. They are dependent on the forming plant, the conditions on which the peat is accumulated and deposited, and the degree of decomposition.

According to Berry and Poskitt (1972), the mechanical properties of peat vary considerably with the difference of their structure. The presence of fiber alters the consolidation process of peat from that of clay and amorphous granular peat. The texture of fibrous peat is coarser when compared to clay. This condition give an implication on the geotechnical properties of peat related to the particle size and compressibility behavior of peat.

The fibrous peat has essentially an open structure with interstices filled with a secondary structural arrangement of non-woody, fine fibrous material (Dhowian and Edil, 1980), thus physical properties of fibrous peat differ markedly from those of mineral soils. The fibrous peat has many void spaces existing between the solid grains. Due to the irregular shape of individual particles, fibrous peat deposits are porous and the soil is considered as a permeable material. Kogure et al. (1993) presented the idea of multi-phase system of fibrous peat, which consists of organic bodies and organic space.

The organic body consists of organic matter and water in inner voids, while the organic space consists of water in outer voids and the soil particles. The solid organic matter can be drained under consolidation pressure. The cross section of deposition and diagram of the multi-phase system of fibrous peat are schematically shown in Figure 2.1(a) and (b).

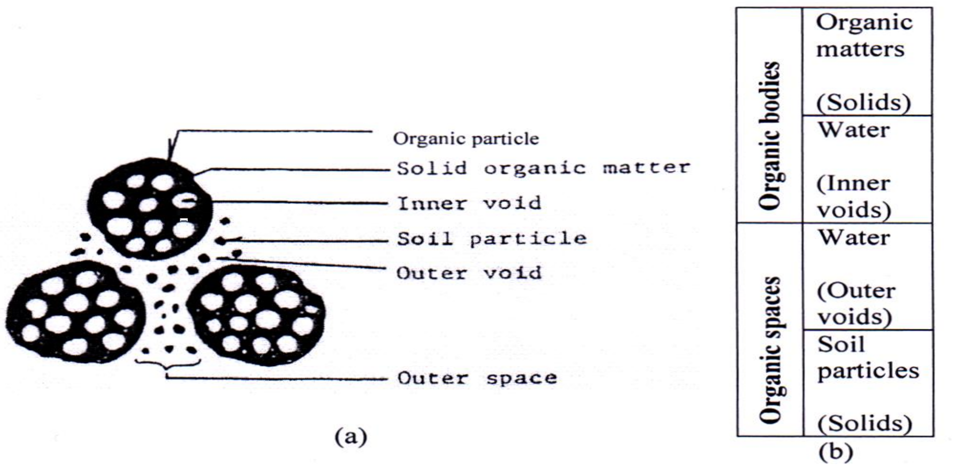


Figure 2.1: Schematic diagram of (a) deposition and (b) multi-phase system of fibrous peat (Kogure et al., 1993)

Dhowian and Edil (1980) showed that fiber arrangement appears to be a major compositional factor in determining the way in which peat soils behave. However, the difference in the fiber content plays an equal important role in the behavior of fibrous peat. The differences in fiber content can be observed in the micrographs through the Scanning Electron Micrograph (SEM). The higher the fiber content, the more the peat will differ from an inorganic soil in its behavior. Figure 2.2 shows a Scanning Electron Micrograph of Middleton fibrous peat specimen under 400 kPa vertical consolidation pressures (Fox and Edil, 1996). The photograph was taken in vertical and horizontal planes.

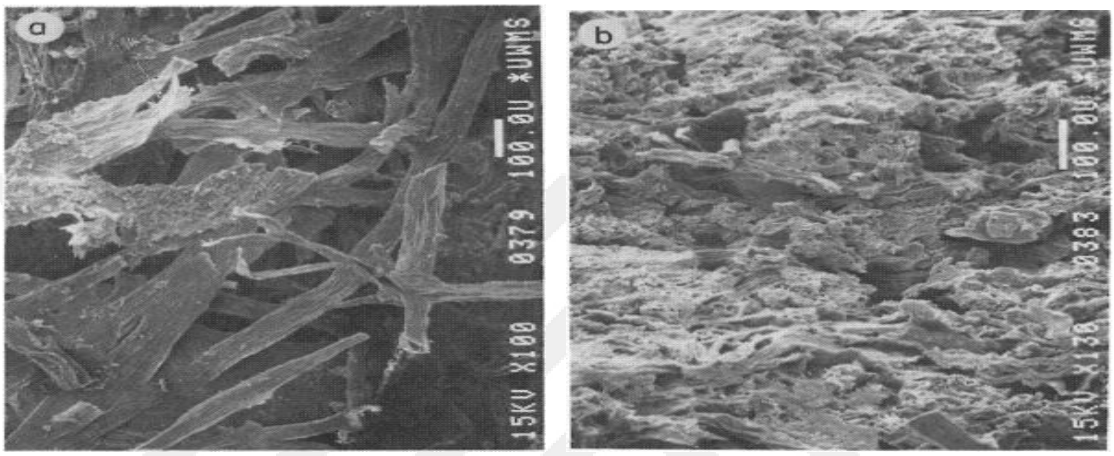


Figure 2.2: Scanning Electron Micrographs of Middleton fibrous peat; (a) horizontal plane, (b) vertical plane (Fox and Edil, 1996)

Comparison of the two micrographs in Figure 2.2 indicates a pronounced structural anisotropy for the fibrous peat with the void spaces in the horizontal direction larger than those in the vertical direction resulting from the fiber orientation within the soil. Individual microstructures remained essentially intact after compression under high-stress conditions. This implies that for the fibrous peat, horizontal rates of permeability and consolidation are larger than their respective vertical rates of permeability and consolidation (Fox and Edil, 1996).

2.1.3 Physical Properties of Peat

Variability of peat is extreme both horizontally and vertically. According to Hobbs (1986) and Edil (1997)as refereed in Huat (2004), The variability results in a wide range of physical properties such as water content ,color ,degree of humification ,specific gravity, density ,acidity and organic contents should be included in full description of peat . The physical properties of peat are influenced by main component of its formation such as mineral content, organic content, moisture and air. When one of these components changes, it will in the changes of the whole physical properties of the peat soil. The results of previous researches on the physical properties of peat around the world are presented in Table 2.1.

Fibrous peat generally has very high natural water content due to its natural water-holding capacity. Soil fabric, characterized by organic coarse particles, holds a considerable amount of water because the coarse particles are generally very loose, and the organic particle itself is hollow and largely full of water. High water content results in high buoyancy and high pore volume leading to low bulk density and low bearing capacity.

Unit weight of peat is typically lower compared to inorganic soils. The average unit weight of fibrous peat is almost equal to or slightly higher than the unit weight of water. Sharp reduction of unit weight was identified with increasing of water content.

Table 2.1: Physical properties of peat based on location (Huat, 2004)

Soil deposits	Natural water content (ω_o, %)	Unit weight γ (kN/m^3)	Specific gravity (G_s)	Organic content (%)
Fibrous peat Quebec	370-450	8.7-10.4	-	-
Fibrous peat, Antoniny Poland	310-450	10.5-11.1	-	65-85
Fibrous peat, Co. Offaly Ireland	865-1400	10.2-11.3	-	98-99
Amorphous peat, Cork, Ireland	450	10.2	-	80
Cranberry bog peat, Massachusetts	759-946	10.1-10.4	-	60-77
Peat Austria	200-800	9.8-13.0	-	-
Peat Japan	334-1320	-	-	20-98
Peat Italy	200-300	10.2-14.3	-	70-80
Peat America	178-600	-	-	-
Peat Canada	223-1040	-	-	17-80
Peat Hokkaido	115-1150	9.5-11.2	-	20-98
Peat West Malaysia	200-700	8.3-11.5	1.38-1.70	65-97
Peat East Malaysia	200-2207	8.0-12.0	-	76-98
Peat Central Kalimantan	467-1224	8.0-14.0	1.50-1.77	41-99

Specific gravity of peat depends greatly on its composition and percentage of the organic content. For an organic content greater than 75 %, the specific gravity of peat ranges between 1.3 and 1.8 with an average of 1.5 (Davis, 1997). The lower specific gravity indicates a lower degree of decomposition and low mineral content. Natural void ratio of peat is generally higher than that of inorganic soils indicating their higher capacity for compression. Natural void ratio of 5-15 is common and a value as high as 25 have been reported for fibrous peat (Hanrahan, 1954).

Peat will shrink extensively when dried. The shrinkage could reach 50 % of the initial volume, but the dried peat will not swell up upon re-saturation because dried peat cannot absorb water as much as initial condition; only 33 % to 55 % of the water can be reabsorbed (Mochtar, 1997).

2.1.4 Chemical Properties of Peat

Chemically, peat consists of carbon, hydrogen, oxygen, and small amount of nitrogen. Previous researches (Soper and Obson, 1922; Chynoweth, 1983; Schelkoph et al., 1983; Cameron et al., 1989) showed that the percentage carbon, hydrogen, oxygen, and small amount of nitrogen are in the ranges of 40-60 %, 20-40 %, 4-6 %, and 0-5 % respectively. The composition is greatly related to the degree of decomposition, the more the peat is decomposed, the less the percentage of the carbon is produced.

According to Ajlouni (2000), the chemical properties of peat are greatly affected by the chemical composition of peat's components, the environment in which they were deposited and the extent of decomposition they have suffered. It is important to know some unique chemical characteristics of peat. Thus, some of the important chemical characteristics are listed below:

a) Peat soils are very acidic with low pH values, often lies between 4 and 7 (Lea, 1956). The acidity tends to decrease with depth, and the decrease may be large near the bottom layer depending on the type of the underlying soil (Muttalib et al., 1991).
b) Cat ion Exchange Capacity (CEC). Peat soils cat ion exchange value is very high that it could reach 100. The common exchangeable cat ions in peats are Ca2+, Mg3+, Fe3+, Al3+, K+, Na+, (NH4)+ . Coastal peat land in Selangor has the value of cat ion exchange nearly to 30, whilst highland peat such as in Simpang Renggam has the cat ion exchange value around 8. Higher value of cat ion exchange is good for the plants to accumulate the nutrients it needed.
c) Peat has no critical nutrient such as phosphate.

d) Electrical conductivity. Peat soils have low value of electrical conductivity around 1dS/m, which will vary according to the area condition. For coastal area, the values can reach up to 5dS/m.
e) Organic carbon. Organic carbon of peat soil increases with the depth increment of the soil. The value can range from 30 % to 40 %.

The submerged organic component of peat is not entirely inert but undergoes very slow decomposition, accompanied by the production of methane and less amount of nitrogen and carbon dioxide and hydrogen sulfide. Gas content affects all physical properties measured and field performance that relates to compression and water flow. The gas content is difficult to determine and no widely recognized 18 method is yet available. A gas content of 5 to 10 % of the total volume of the soil is reported for peat and organic soils (Muskeg Engineering Handbook, 1969). The results of previous researches on the physical and chemical properties of peat around the world are presented in Table 2.2.

Table 2.2: Important physical and chemical properties for some peat deposits (Ajlouni, 2000)

Peat type	Natural water content (%)	Bulk density (Mg/m^3)	Specific Gravity (G_s)	Acidity (pH)	Ash content (%)	Reference
Fibrous-woody	484-909	-	-	-	17	Colley (1950)
Fibrous	850	0.95-1.03	1.1-1.8	-	-	Hanrahan (1954)
Fibrous	605-1290	0.87-1.04	1.41-1.7	-	4.6-15.8	Samson and LaRochell, 1972
Coarse Fibrous	613-886	1.04	1.5	4.1	9.4	Berry and Vickers, 1975
Fibrous sedge	350	-	-	4.3	4.8	Levesqe et al. 1980
Fibrous Sphagnum	778	-	-	3.3	1	
Coarse Fibrous	202-1159	1.05	1.5	4.17	14.3	Berry ,1983
Fine Fibrous	660	1.05	1.58	6.9	23.9	NG and Eischen 1983
Peat Portage	600	0.96	1.72	7.3	19.5	Edil and Mochtar 1984
Peat Waupaca	460	0.96	1.68	6.2	15	
Fibrous Peat Middleton	510	0.91	1.41	7	12	
Fibrous Peat Noblesville	173-757	0.84	1.56	6.4	6.9-8.4	
Fibrous	660-1590	-	1.53-1.68	-	0.1-32.0	Lefebvre et al. 1984
Fibrous (Middleton)	510-850	0.99-1.1	1.47-1.64	4.2	5-7	Ajlouni, 2000
Fibrous (James Bay)	1000-1340	0.85-1.02	1.37-1.55	5.3	4.1	

2.1.5 Classification of Peat

There are many types of classification exist to classify the peat soil. Physical, chemical, and physicochemical properties of peat such as texture, organic content, pH, color, moisture content, and degree of decomposition could serve as a basis for peat classification (Ajlouni, 2000). A literature review was conducted on the peat classification published by Farnham (1968) and also by the International Peat Society. According to Farnham (1968) the existing classification systems of peat are based on:

a. Chemical properties of the peat. Several classification systems are based on chemical properties. The distinction into eutrophic (nutrient rich), mesotrophic (moderately nutrient rich) and oligotrophic (nutrient poor) organic soils applies to the peat material. Eutrophic peat environments are characterized by flooding with nutrient rich water, whereas oligotrophic peat is fed by nutrient-poor water. Peat can be classified on their inherent chemical properties as well as their general chemical environment. Peat are classified in this way using the amounts of water-soluble substances, the ether and alcohol soluble substances, and the cellulose and hemi cellulose content.
b. Physical characteristics of the peat. The first person to classify peat on physical properties was von Post. Von Post has developed a field method to indicate stages of decomposition. The von Post scale (Table 2.3) recognizes 10 steps.
c. Topography and geomorphology. Topographical classification systems deal primarily with aspects of landscape. The aspects of landscape meant here are the hydrological conditions, the origin of the peat swamp, and the nature of the accumulated material. The topographical classifications are useful for indicating possible limitation on reclamation and necessary management procedures.
d. Surface vegetation. Peat lands or swamps can be classified according to the present vegetation cover, as is done in Canada and northern Europe. Such systems will be concern much only when if there is a relation with management requirements and, particularly, reclamation problems.
e. Botanical origin of the peat. Peat can be divided into major vegetation types such as moss peat, sedge peat, heath, saw-grass peat. One of the problems of

this type of classification is that peat deposits are often characterized by vertical sequences or layers of peat of different vegetative origin, each layer indicating a specific stage in the development of the deposit.

f. Genetic processes within the peat swamp. Classifications using assumed genetic processes are based mainly on the climate under which peat is formed and changes in the peat, including those as a result of a soil forming process, after reclamation. In the Russian system genetic origin is used at a high categorical level.

Table 2.3: Classification peat soil from von Post (Huat, 2004)

Condition of peat before squeezing				Condition of peat on squeezing		
Degree of Humification	Soil color	Degree of decompo-sition	Plant structure	Squeezed solution	Material extruded (passing between fingers)	Nature of Residue
H_1	White or yellow	None	Easily identified	Clear, color-less water	Nothing	Not pasty
H_2	Very pale brown	Insignifi-cant	Easily identified	Yellowish water/pale brown-yellow	Nothing	Not pasty
H_3	Pale brown	Very slight	Still identified	Dark brown, muddy water not peat	Nothing	Not pasty
H_4	Pale brown	Slight	Not easily identified	Very dark brown muddy water	Some peat	Somewhat pasty
H_5	Brown	Moderate	Recognizable but vague	Very dark brown muddy water	Some peat	Strongly pasty
H_6	Brown	Moderately strong	Indistinct (more distinct after squeezing)	Very dark brown muddy water	About one-third of peat squeezed out	Very strongly pasty
H_7	Dark brown	Strong	Faintly recognizable	Very dark brown muddy water	About one-half of peat squeezed out	Very strongly pasty
H_8	Dark brown	Very strong	Very indistinct	Very dark brown pasty water	About two-third squeezed out	Very strongly pasty
H_9	Very dark brown	Nearly complete	Almost recognizable	Very dark brown muddy water	Nearly all the peat squeezed out as fairly uniform paste	Very stronglypasty
H_{10}	Black	Complete	Not discernible	Very dark brown muddy paste	All the peat passes between the fingers; no free water visible	N/A

Generally, the most common classification system used in geotechnics for classification of peat soil is Organic content (Table 2.4). Ash content is defined as the percentage of ash to the weight of dried peat. In addition, the peat is classified according to fiber content because the consolidation process of fibrous peat from that of organic soil or amorphous peat can be changed by presence of fiber. If fiber content is less than 20 % in any organic soil, this soil is called as amorphous peat (ASTM D4427). It includes mostly particles of colloidal size (less than 2 microns), and it absorb the pore water around the particle surface. There are some similarities between the behavior of amorphous granular peat and clay soil. If fiber content is more than 20 % this soil is called as fibrous peat (ASTM D4427). It possesses two types of pore i.e: macro-pores (pores between the fibers) micro-pores (pores inside the fiber itself). Table 2.4 shows the classification of peat based on organic and fiber content.

Table 2.4: Classification of peat based on organic and fiber content

Classification of peat soil based on ASTM standards	
Fiber Content (ASTM D1997)	Fibric : Peat with greater than 67 % fibers
	Hemic : Peat with between 33 % and 67 % fibers
	Sapric : Peat with less than 33 % fibers
Ash Content (ASTM D2974)	Low Ash : Peat with less than 5 % ash
	Medium Ash : Peat with between 5% and 15 % ash
	High Ash : Peat with more than 15 % ash
Acidity (ASTM D2976)	Highly Acidic : Peat with a pH less than 4.5
	Moderately Acidic : Peat with a pH between 4.5and 5.5
	Slightly Acidic : Peat with a pH greater than 5.5 and less than 7
	Basic : Peat with a pH equal or greater than 7

2.2 Soil Compressibility

2.2.1 Introduction

In general, the compressibility of a soil consists of three stages, namely initial compression, primary consolidation, and secondary compression. While initial compression occurs instantaneously after the application of load, the primary and secondary compressions are time dependent. The initial compression is due partly to the compression of small pockets of gas within the pore spaces and the elastic compression of soil grains. Primary consolidation is due to dissipation of excess pore water pressure caused by an increase in effective stress whereas secondary compression takes place under constant effective stress after the completion of dissipation of excess pore water pressure.

The time required for the water to dissipate from the soil depends on the permeability of the soil itself. In granular soil, the process is rapid and hardly noticeable due to its high permeability. On the other hand, the consolidation process may take years in clay soil. For peat, the primary consolidation occurs rapidly due to high initial permeability and secondary compression takes a significant part of compression.

The compressibility characteristics of a soil are usually determined from consolidation tests. General laboratory tests for measurement of compression and consolidation characteristics of a soil are: Oedometer test, Constant Rate of Strain (CRS) test, and Row cell test. The procedures for these tests are fully described in BS 1377-6 and Head (1982, 1986).

Although more sophisticated consolidation tests are now available, Oedometer test is still recognized as the standard test for determining the consolidation characteristics of soil. Oedometer cell can accommodate 50 mm diameter and 20 mm thick samples. The schematic diagram of consolidation test on Oedometer cell is shown in Figure 2.3.

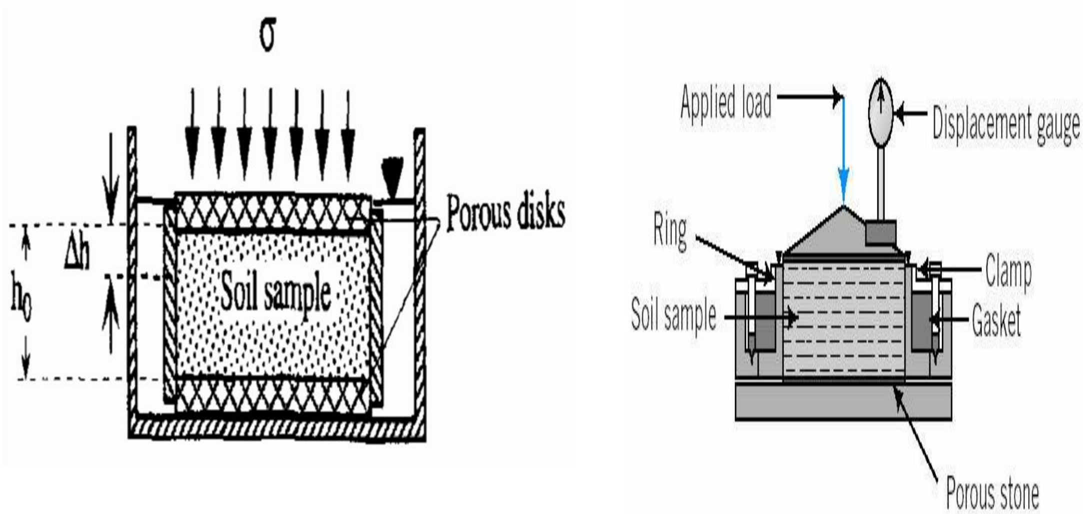


Figure 2.3: Schematic diagram of Oedometer cell (Bardet, 1997)

For standard test, the samples were subjected to consolidation pressures with load increment ratio of 1. The load is applied through a mechanical lever arm system, thus measurement can be easily affected by sudden shock. Excessive disturbance affects the e-log p' plot, gives low value of pre-consolidation pressure and high coefficient of volume compressibility at low stresses. The other limitation of the standard Oedometer test is that there is no means of measuring excess pore water pressures, the dissipation of which control the consolidation process. Therefore the estimation of compressibility is based solely on the change of height of the specimen.

The analysis of compression of such soils present certain difficulties when the conventional methods are applied because the curves obtained from the conventional Oedometer tests and the behavior exhibits by them differ from that of clay. Furthermore, such soils are more prone to decomposition during Oedometer testing. Gas content and additional gas generation also may complicate the interpretation of Oedometer tests (Edil, 2003). Some researchers (Berry and Poskitt, 1972; Ajlouni, 2000; Colleselli et al., 2000; Robinson, 2003) had presented the behavior of fibrous peat and the recent advances in formulating their behavior.

Advantages and disadvantages of Oedometer test are outlined by Head (1986). Among the advantages is the relatively small size of specimen. The small specimen size gives a reasonable consolidation time and the test can be extended to observe the secondary compression. The test provides a reasonable estimate of the amount of settlement of structure on inorganic clay deposits. On the other hand, the rate of settlement is often underestimated, that the total settlement is reached in a shorter time than that predicted from the test data. This is largely due to the size of sample, which does not represent soil fabric and its profound effect on drainage conditions.

2.2.2 Primary Consolidation

One-dimensional theory of consolidation developed by Terzaghi in 1925 carries an assumption that primary consolidation is due to dissipation of excess pore water pressure caused by an increase in effective stress whereas secondary compression takes place under constant effective stress after the completion of the dissipation of excess pore water pressure. Other important assumptions attached to the Terzaghi consolidation theory are that the flow is one-dimensional and the rate of consolidation or permeability is constant throughout the consolidation process.

Consolidation characteristics of soil can be represented by consolidation parameters such as coefficient of axial compressibility a_v, coefficient of volume compressibility m_v, compression index c_c, and recompression index c_r. Another important characteristic of soil compressibility is the pre-consolidation pressure (σ_c'). The soil that has been loaded and unloaded will be less compressible when it is reloaded again, thus settlement will not usually be great when the applied load remains below the pre-consolidation pressure. These parameters can be estimated from a curve relating void ratio (e) at the end of each increment period against the corresponding load increment in linear scale (Figure 2.4) or logarithmic scale (Figure 2.5).

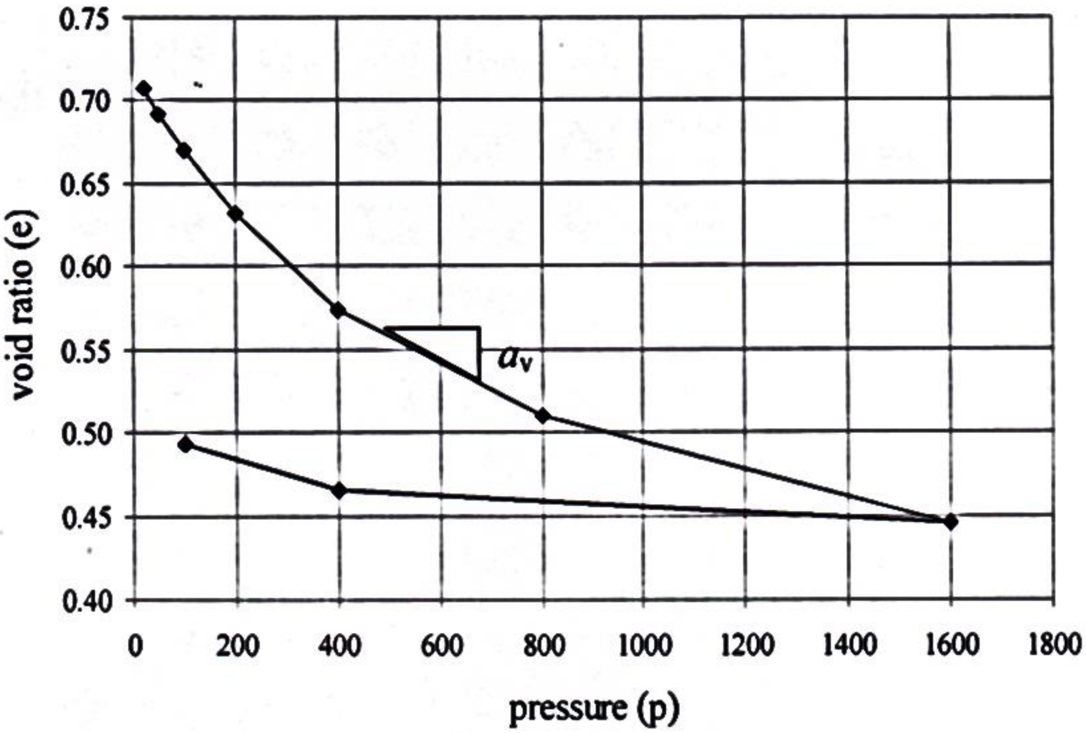


Figure 2.4: Plot of void ratio versus pressure in linear scale (Nurly Gofar and Khairul Anuar Kassim, 2005)

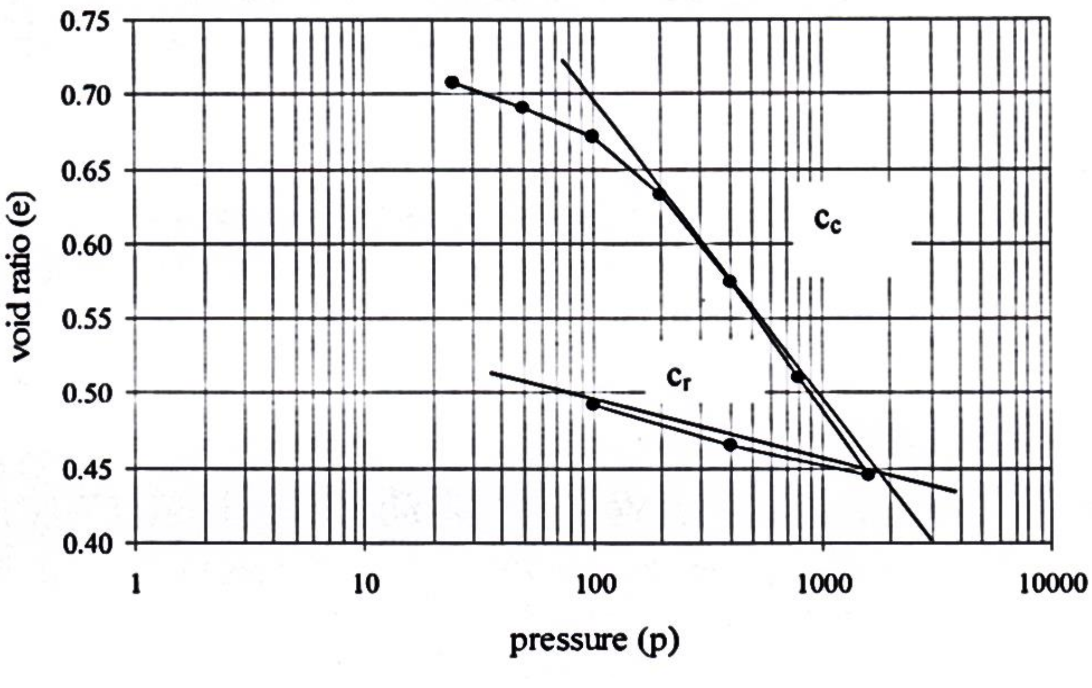


Figure 2.5: Plot of void ratio versus pressure in logarithmic scale (Nurly Gofar and Khairul Anuar Kassim, 2005)

As shown in Figure 2.4, the coefficient of axial compressibility a_v is the slope of the e-p' curve for a certain range of stress while the coefficient of volume compressibility m_v can be computed as:

$$m_v = \frac{a_v}{1+e_0} \tag{2.1}$$

Where $m_v =$ Coefficient of volume compressibility,

$a_v =$ Coefficient of axial compressibility, and

$e_0 =$ Initial void ratio

The compression index c_c and recompression index c_r are the slope of the e-log p' curve (Figure 2.5) for loading and unloading stages.

Consolidation settlement is calculated based on the value of either the coefficient of volume compressibility (m_v) or the compression indices (c_c and c_r). Due to construction, the total vertical stress on a soil element at depth z is increased by $\Delta\acute{\sigma}$. This increase of stress will results in the decrease of void ratio corresponds to $\Delta e = e_0 - e_1$.

By knowing the ratio of the change in void ratio to the change in the effective stress in e-p' curve (Figure 2.4), then

$$s_c = \Delta H = \left[\frac{e_0 - e_1}{\acute{\sigma}_1 - \acute{\sigma}_0}\right] \left[\frac{\acute{\sigma}_1 - \acute{\sigma}_0}{1+e_0}\right] H \tag{2.2}$$

$$s_c = a_v \left[\frac{1}{1+e_0}\right] \left(\acute{\sigma}_1 - \acute{\sigma}_0\right) H \quad = \quad m_v \, \Delta\acute{\sigma} H \tag{2.3}$$

$$s_c = \left[\frac{\Delta e}{1+e_0}\right] H \tag{2.4}$$

By using the e-log p' curve, the change in void ratio can be written as:

$$\Delta e = c_c \log \frac{\sigma_1}{\sigma_0} \tag{2.5}$$

And the settlement of normally consolidated clay due to change of stress $\Delta\acute{\sigma}$ is given as:

$$s_c = c_c \frac{H}{1+e_0} \log \frac{\acute{\sigma}_0+\Delta\sigma}{\acute{\sigma}_0} \qquad (2.6)$$

Where $s_c = \Delta H$ = consolidation settlement,

H = thickness of consolidation soil layer,

$\Delta\acute{\sigma} = \acute{\sigma}_1 - \acute{\sigma}_0$= the change in the effective in e-p' curve,

$\Delta e = e_0 - e_1$ = the change in void ratio, and

c_c= compression index.

The soil that has been loaded and unloaded will be less compressible when it is reloaded again. Thus, it is also necessary to estimate the pre-consolidation pressure i.e.: the stress carried by soil in the past (σ_c') because consolidation settlement will not usually be great when the applied load remains below the pre-consolidation pressure. The pre-consolidation pressure can be obtained from the consolidation curve by procedure suggested by cassagrande.

If the pre-consolidation pressure obtained from laboratory test ($\acute{\sigma}_c$) is greater than the existing overburden pressure ($\acute{\sigma}_o$) and the added stress increases the existing pressure below the pre-consolidation pressure, then the compression index (c_c) should be replaced with the recompression index (c_r) in Equation 2.6, which results in Equation 2.7 If the additional stress increases the existing pressure beyond the pre-consolidation pressure, then Equation 2.6 is modified as Equation 2.8.

$$s_c = c_r \frac{H}{1+e_0} \log \frac{\acute{\sigma}_0+\Delta\acute{\sigma}}{\acute{\sigma}_o} \qquad (2.7)$$

$$s_c = c_r \frac{H}{1+e_0} \log \frac{\acute{\sigma}_c}{\acute{\sigma}_0} + c_c \frac{H}{1+e_0} \log \frac{\acute{\sigma}_0+\Delta\sigma}{\acute{\sigma}_c} \qquad (2.8)$$

$\acute{\sigma}_c$ =Pre-consolidation pressure, and

c_r = recompression index.

2.2.3 Secondary Compression

For some soils, especially those containing organic material, the compression does not cease when the excess pore water pressure has completely dissipated but continues at a gradually decreasing rate under constant effective stress. Thus, it is common to differentiate the two processes as primary consolidation and secondary compression. Secondary compression, also referred as creep, is thought to be due to the gradual readjustment of the clay particles into a more stable configuration following the structural disturbance caused by the decrease in void ratio.

Previous researchers (Leonards and Girault, 1961; Berry and Vickers, 1975; Lefebvre et al., 1984; Hobbs, 1986; Kogure et al., 1986) have shown that both primary consolidation and secondary compressions can take place simultaneously. However, it is assumed that the secondary compression is negligible during primary consolidation, and is identified after primary consolidation is completed. Secondary compression of soil is conveniently assumed to occur at a slower rate after the end of primary consolidation. The rate of secondary compression in the standard consolidation test can be defined by the slope (c_α) of the final part of the void ratio versus logarithmic of time curve (Figure 2.6).

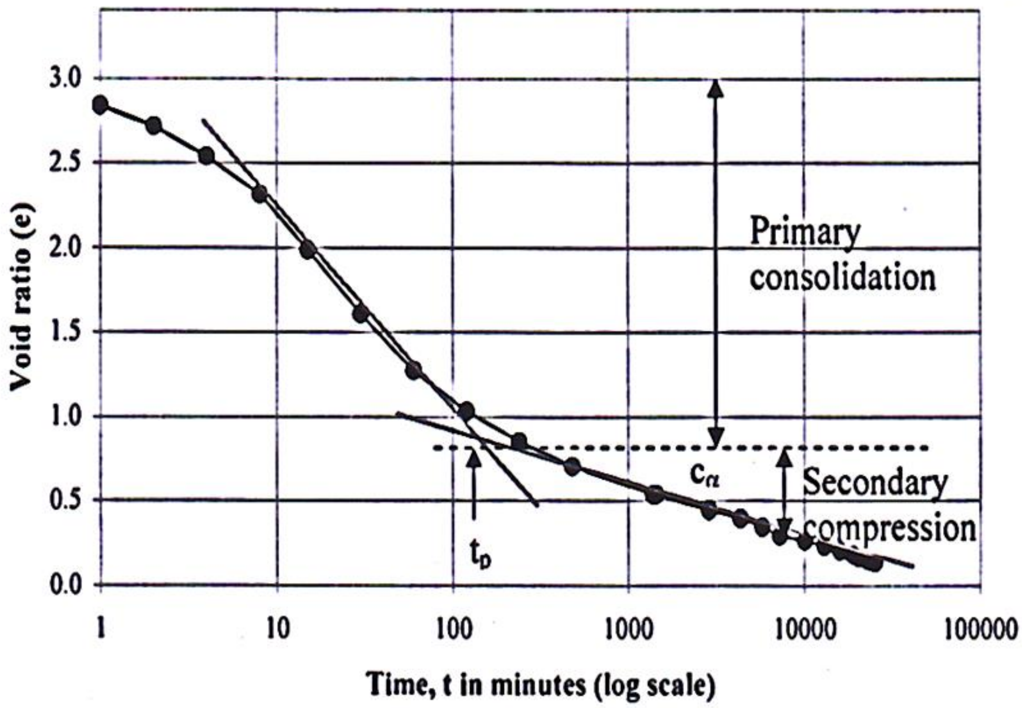


Figure 2.6: Determination of the coefficient of rate of secondary compression from consolidation curve (Cassagrande's method) (Nurly Gofar and Khairul Anuar Kassim, 2005)

The axial rate of consolidation can be obtained from Figure 2.6 as the ratio of change on the void ratio to the change on the logarithmic of time.

$$c_\alpha = \frac{\Delta e}{\Delta \log t} = \frac{\Delta e}{\log \frac{t_f}{t_p}} \tag{2.9}$$

Where c_α = coefficient of secondary compression,

t_p = time of the completion of primary consolidation,

t_f = time for which the secondary compression settlement is required (Design life of a structure), and

Δe = the change of void ratio from t_p to t_f

Research showed that the ratio of c_α / c_c is almost constant and varies from 0.025 to 0.06 for inorganic soil, while a slightly high range was obtained for organic soils and peat (Holtz and Kovacs, 1981). A higher ratio was obtained for highly compressible clay and organic soils, thus the amount of secondary compression settlement may be quite significant. The settlement due to the secondary compression (s_s) is therefore:

$$s_s = \frac{c_\alpha}{1+e_0} H \log \frac{t_f}{t_p} \tag{2.10}$$

Where s_s = settlement due to secondary compression, and

H = initial thickness.

2.2.4 Compressibility of Fibrous Peat

The compression behavior of fibrous peat is different from that of clay soil. The compressibility of fibrous peat consists of two stages: primary consolidation and secondary compression. The primary consolidation of the fibrous peat is very rapid, and large secondary compression, even tertiary compression is observed. Secondary compression is generally found as the more significant part of compression because the time rate is much slower than the primary consolidation. Subsequently the formula used to estimate the amount of compression is different from that of clay soil.

Generally fibrous peat undergoes large settlements in comparison to clays when subjected to loading. The compression behavior of fibrous peat varies from the compression behavior of other types of soils in two ways. First, the compression of peat is much larger than of other soils. Second, the creep portion of settlement plays a more significant role in determining the total settlement of peat than of other soil types.

Researches (Mesri and Rokhsar, 1974; Mesri and Choi, 1985b; Mesri and Lo, 1991; Lan, 1992) showed that Terzaghi's theory of consolidation is not applicable for the prediction of the compression of fibrous peat. Subsequently, many theories of consolidation have been developed mainly as modifications to Terzaghi's theory. Such modifications, mostly intended for soft clays and silts, include decrease in permeability with the progress of consolidation, the changes in compressibility during consolidation, time related compressibility during and after primary consolidation phase, the finite value of strains, and effect of self-weight. Of all methods, few theories were developed solely to model compressibility of fibrous peat (Gibson and Lo, 1961; Barden, 1968; Berry and Poskitt, 1972; DenHaan, 1996).

The rate of primary consolidation of fibrous peat is very high; however it decreases with the application of consolidation pressure. Lea and Browner (1963) indicated a significant decrease of coefficient of rate of consolidation (c_v) during application of pressure from 10 to 100 kPa. Compression of fibrous peat continues at a gradually decreasing rate under constant effective stress, and this is termed as the secondary compression. The secondary compression of peat is thought to be due to further decomposition of fiber which is conveniently assumed to occur at a slower rate after the end of primary consolidation (Mesri et al., 1997).

Mesri and Rokhsar (1974) developed a theory of consolidation based on assumptions for soil properties that were more realistic than those in the original Terzaghi theory of one-dimensional consolidation. The assumptions were that:

1. The soil undergoes a finite strain.
2. The compressibility and the permeability of the soil are variable during consolidation.
3. The soil may display recompression and compression behavior.
4. A unique relationship between compressibility and effective stress and time.

Mesri and Choi (1985b) modified the theory of consolidation introduced by Mesri and Rokhsar (1974) to include a nonlinear relationship between void ratio and the logarithmic of effective vertical stress. Lan (1992) claimed that the c_α/c_c concept is not applicable to peat compression. Therefore, based on the uniqueness of σ'_v-e-e' concept and the relationship between e and σ'_v, he proposed a constitutive equation for modeling the primary consolidation and secondary compression of peat in the normally consolidated range.

Fox (2003) stated that the standard procedure for consolidation test specified the load increment ratio (LIR) of one and each load is maintained for 24 hour. For some soils, especially peat, the end of primary consolidation can be reached at time much less than 24 hour. Thus, the estimation of the compression index (c_c) based on consolidation test conducted on fibrous peat in which the primary consolidation occurs rapidly may not be accurate.

2.3 Shear Strength

2.3.1 Introduction

Shear strength is one of the most important engineering properties of a soil, because it is required whenever a structure is dependent on the soil's shearing resistance. The shear strength is needed for engineering situations such as determining the stability of slopes or cuts, finding the bearing capacity for foundations, and calculating the pressure exerted by a soil on a retaining wall, (Reddy, 1998).Soil will eventually reach failure and deform excessively when it is subjected to gradually increasing load. This failure is related to the shear strength some failure criteria are needed to define the shear strength of the soil. The failure criteria are developed based on stress-strain relationship of the soil. The concepts of elasticity theory apply to soil in a very approximate way. It assumed that the material is homogeneous, isotropic, and have a linear stress strain relationship. On the other hand, the soils in general are non-homogeneous, exhibit anisotropy, and have non-linear stress-strain relationships. The amount of strain developed in soil depends not only on the applied load, but also on the composition, void ratio, past stress history, and the manner in which the stress is applied.

The stress-strain relationship of the soil can be idealized in several forms: (a) elastic-plastic, (b) elastic-perfectly plastic, (c) rigid-perfectly plastic, and (d) elastic strain-hardening plastic. All of these relationships assume elasticity at lower strain level, but soil will eventually reach plastic condition after yielding condition is achieved. Thus the most realistic stress-strain relationship is the elasto plastic behavior.

Coulomb (1776) conducted numerous tests to measure the shear strength of a soil and concluded that the shear strength of a soil composed of two components: (1) that depends on the normal stress internal friction angle (ϕ) and (2) the cohesion (c) which is independent on the normal stress. This theory is combined with the Mohr failure envelope and resulted in the Mohr-Coulomb failure criterion which relates the shear strength of soil to the applied normal stress:

$$\tau_f = c + \sigma_n \tan \phi \tag{2.11}$$

Where c = apparent cohesion (assumed to be constant),

σ_n= normal stress on slip surface, and

ϕ = angle of friction (or angle of shearing resistance).

The relationship for the limiting shear strength is plotted as a straight line to obtain the shear strength parameters ϕ and c (Figure 2.7).

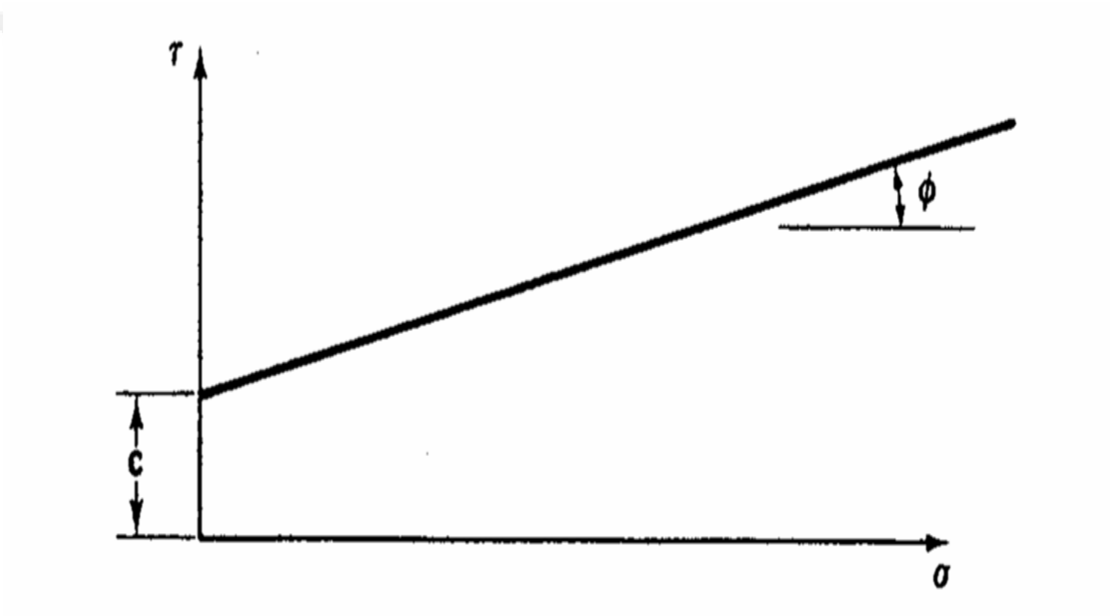


Figure 2.7: The coulomb strength equation presented graphically (Holtz and Kovacs, 1981)

The shear strength of soil is usually evaluated for total and effective stress conditions. The total stress condition happened in undrained condition with short time critical period, while the effective stress condition usually occurred in drained condition with long term critical period and zero pore water pressure.

The simplest type of shear test, in principle, is direct shear. Direct shear test is usually conducted in accordance to BS 1377: Part 7 and ASTM D3080. Direct shear test is the most popular test done to determine shear strength of soil with friction. In a direct shear test, the soil is placed in a split shear box and stressed to failure by moving one part of the container relative to the other. Figure 2.8 show the schematic diagram of direct shear test.

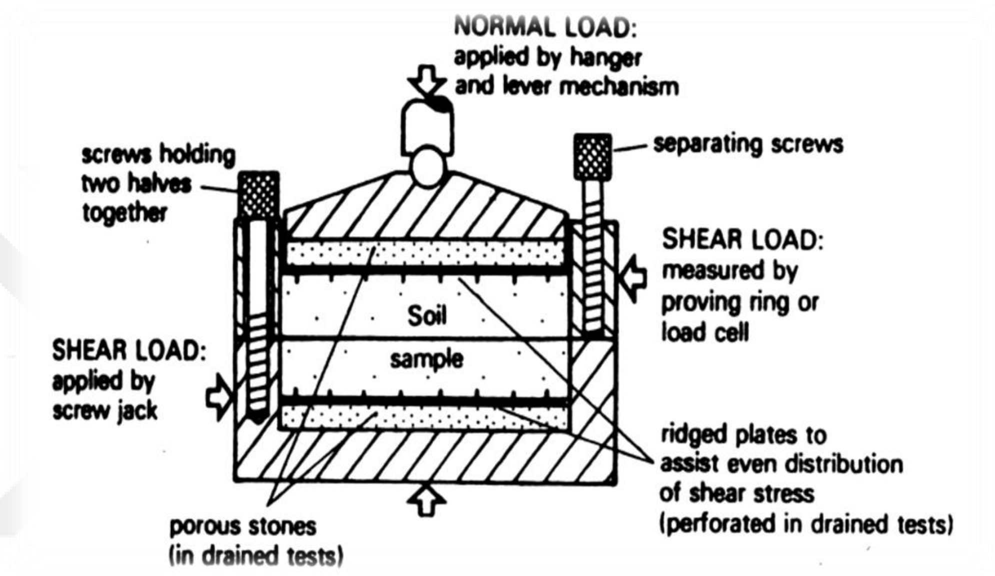


Figure 2.8: Direct shear apparatus (Whitlow, 2001)

A vertical force (N) is applied to the specimen through a loading plate and shear stress is gradually applied on horizontal plane by causing the two halve of the box to move relative to each other. The shear force (T) being measured together with the corresponding shear displacement (Δl). Normally the change in thickness (Δh) of the specimen is also measured. A number of specimens of the soil are tested under different normal forces, and the value of shear stress at failure is plotted against the normal stress for each test. The shear strength parameters are often obtained from the best line fitting the plotted points.

The direct shear test offers the easiest way to measure the friction angle of sand or other dry soil. It is not useful for testing soils containing water unless they are free draining and have a very high permeability, because it is difficult to control the drainage and thus volume changes during testing. For this reason, the direct shear tests should be used with caution in determining the undrained shear strength of cohesive soils. Figure 2.9 shows the typical results from a set of direct shear tests (Head, 1980).

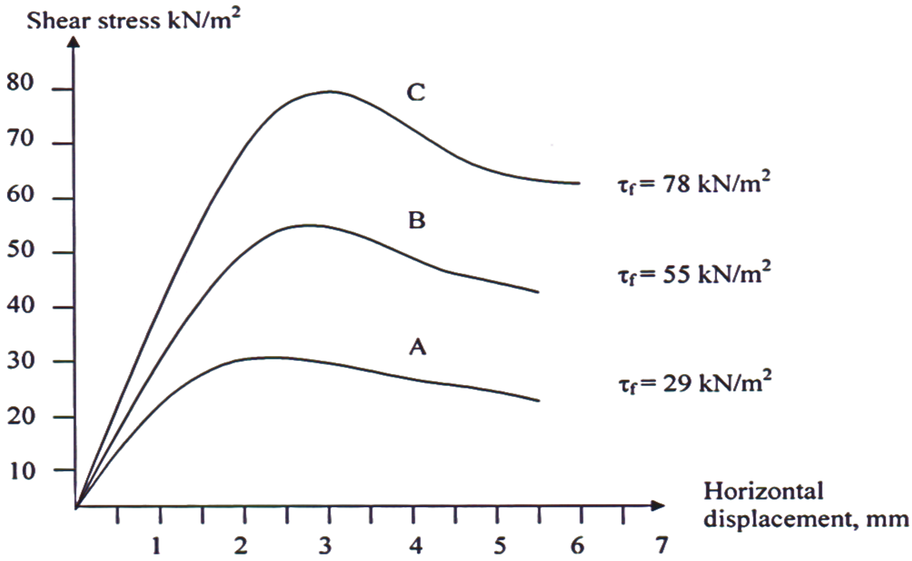


Figure 2.9: Shear stress against displacement curve (Head, 1980)

By carrying out tests on a set of three similar specimens of the same soil under different normal pressures, the relationship between the shear stress at failure and normal applied stress is obtained.

The direct shear test apparatus has certain advantages for the determination of the shear strength through testing. According to Gan et al (1988), direct shear testing of soil is desirable since less time is required to fail the soil specimen than when using the triaxial test. The time to failure in the direct shear test is greatly reduced because the specimen is relatively thin. These advantages could be summarized as below (Head, 1980):

a. The test is relatively quick and simple to carry out.

b. The basic principle is easily understood.

c. Preparation of the test specimens is not difficult.

d. The principle can be extended to gravelly soils and other materials containing large particles, which would be more expensive to test by other type of test.

e. The angle of friction between soils can be easily measured.

The disadvantages of the direct shear box are summarized below (Head, 1980):

a. Pour water pressure cannot be measured.

b. The area of contact between the soil in the two halves of the shear box decreases as the test proceeds.

c. The soil specimen is constrained to fail along a predetermined plane of shear.

2.3.2 Shear Strength of Fibrous Peat

Peat is created under the conditions of low temperature and high humidity and its known that peat is fibrous and highly compressible compared with most mineral soils, (Kogure et al, 1993). Peat soils are classified as problematic soils mainly because of their high compressibility and low shear strength. Shear strength plays an important role not only during the construction for supporting construction equipment but as well as the end of construction in supporting the structure. The undrained shear strength of peat is a critical parameter as for other soil. The undrained shear strength of peat deposits increases rapidly during and after construction, (Ajlouni, 2000). Determination of peat soil's shear strength is always associated with the problems due to several variables such as origin of soil, water content, organic content and degree of humification (Huat, 2004). Another problem in obtaining the shear strength of peat is the difficulty encountered during the specimen trimming (Ajlouni, 2000). The shear strength parameters are generally lower with increasing degrees of humification which means less fiber content in it. The angle of friction is generally higher for the more fibrous peat.

In situations where peat is loaded by the sheer force, friction can develop amongst adjacent fibers and between fibers and fill material. The force will be taken by the fiber. If the load direction is in the same direction as the fibers, it has the effect of reinforcement. As stated in Huat (2004), the effect of organic matter and stiffness of soils depends largely on whether the organic matter is decomposed or consists of fibers which can act as reinforcement. In general fibrous peat has higher shear strength than other group of peat such as hemic peat and sapric peat.

As noted by Edil (1997), the presence of fiber affects the strength behavior of peat. The fiber of peat contributes to the shear strength as the fiber can be considered as reinforcement. The degree of reinforcement depends on the loading direction in relation to main fiber direction. As a result of the sedimentation process and compaction, the main direction is usually horizontal however it is possible that a section of peat has a vertical orientation. The shear strength behavior of peat is highly anisotropy, Hanzawa et al (1994). The shear strength of a soil is not only a function of the material itself, but also of the stress applied, and the manner in which the stress is applied. However, the friction is mostly due to the fiber and the fiber is not always solid because it is usually filled with water and gas. Thus, the high friction angle does not actually reflect the high shear strength of the soil Edil and Dhowian (1981).

Some researchers have studied shear strength properties of several types of peat through laboratory tests and the results show that their behavior is essentially frictional, with high friction angles and relatively low cohesion intercepts (Adam, 1965; Edil and Dhowian, 1981). The angle of friction is generally higher for the more fibrous peat. As noted by Edil (1997 Edil and Dhowian (1981) reported an angle of friction of 50 for amorphous peat and 53 ° - 57° for fibrous peat while Landva (1983) indicated a range of friction angle of 27 ° - 32 ° under a normal pressure of 3 to 50 kPa .

CHAPPTER III

EXPERIMENTAL PROGRAM

3.1 Introduction

The study is an experimental research, which focuses on laboratory tests. Literature review was carried out to identify the problem area and to enhance the understanding of the behavior of fibrous peat especially soil physical properties, shear strength of peat soil and to gather sufficient information on consolidation behavior of fibrous peat. Preliminary test was done in this research; including the determination of moisture content, unit weight, specific gravity, acidity and liquid limit.

Classification tests were also conducted in order to classify the peat based on the degree of decomposition, organic content, and fiber content. The research used remolded sample of fibrous peat soil sampled from Sakarya region, Turkey. The focuses of the research were to determine shear strength parameter of the fibrous peat with sand column based on direct shear test results and to determine the compressibility characteristics of fibrous peat with sand column analyzed based on data obtained from the results of consolidation test using Oedometer.

All the laboratory test procedures are based on the manual of soil laboratory testing (Head, 1981, 1982, 1986) in accordance with the British (BS) and U.S. (ASTM) Standards. Figure 3.1 shows summery of experimental program of the research.

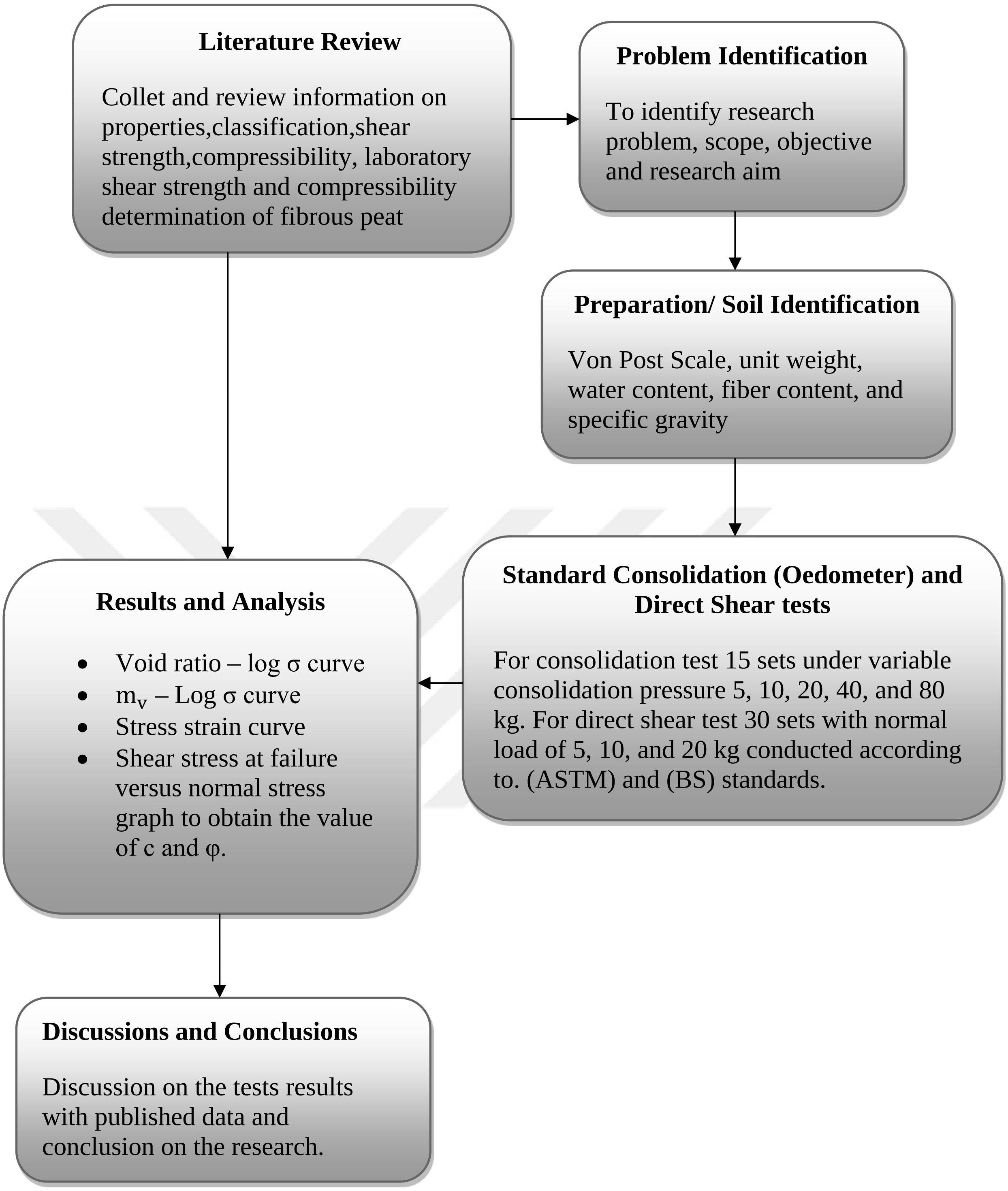


Figure 3.1: Experimental program flow chart of research

3.2 Materials

3.2.1 Sand

The sand used for the test is poorly graded passing from 2 mm sieve size and retaining on 0.075 mm sieve size (Appendix A). The main reason of selecting this sieve range is to minimize the effect of size of the sand particle on test results, and also to only observe changing the diameter of sand column in the organic soil by mass. Because of this reason poorly graded river sand was used in all tests. River sand particles used in the testing program are shown in Figure 3.2 and 3.3.

Figure 3.2: River sand particles used in the testing program (Electronic Microscope)

In this study cylindrical thin tubes having four different diameters 1.7 cm, 2.5 cm, 3.5cm, and 4.7 cm, were used to make sand column in organic soil. Figure3.3 shows the different sand column used in this study.

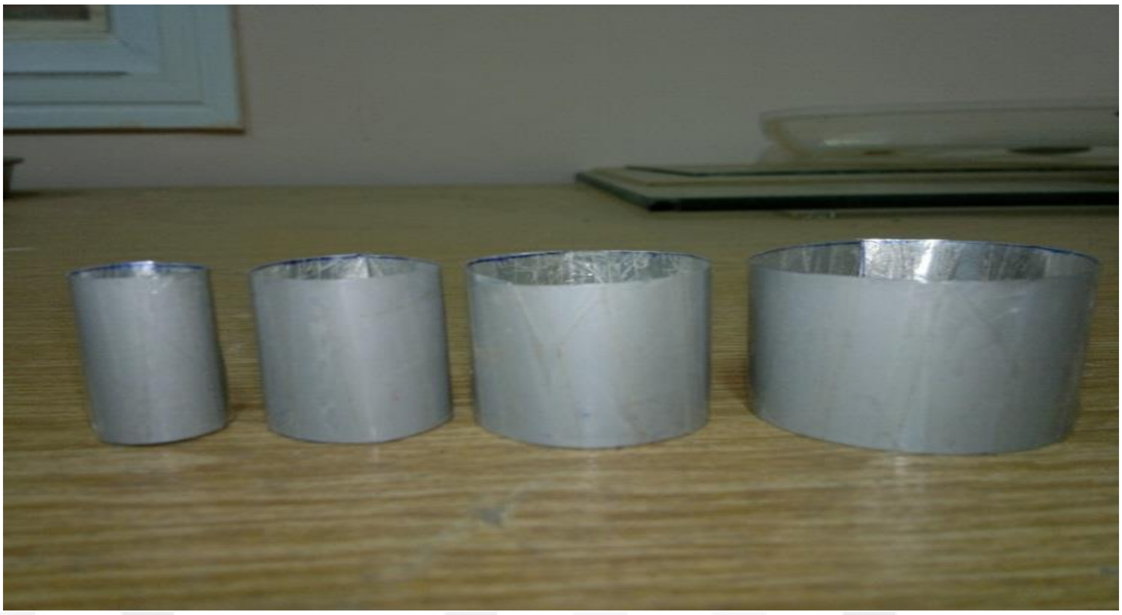


Figure 3.3: All sand column used in the study

3.2.2 Peat Soil

The peat soil used in this study was obtained from Sakarya region, Turkey. The fibrous peat used for all test is passing from 2mm sieve size and retaining on #100 (0.15 mm) sieve size, the sample is put in water for two days then used in the test. This organic soil is classified as peat by Unified Soil Classification System (USCS). And peat by classification system suggested by Wüst et al., (2003) in Figure 3.4.

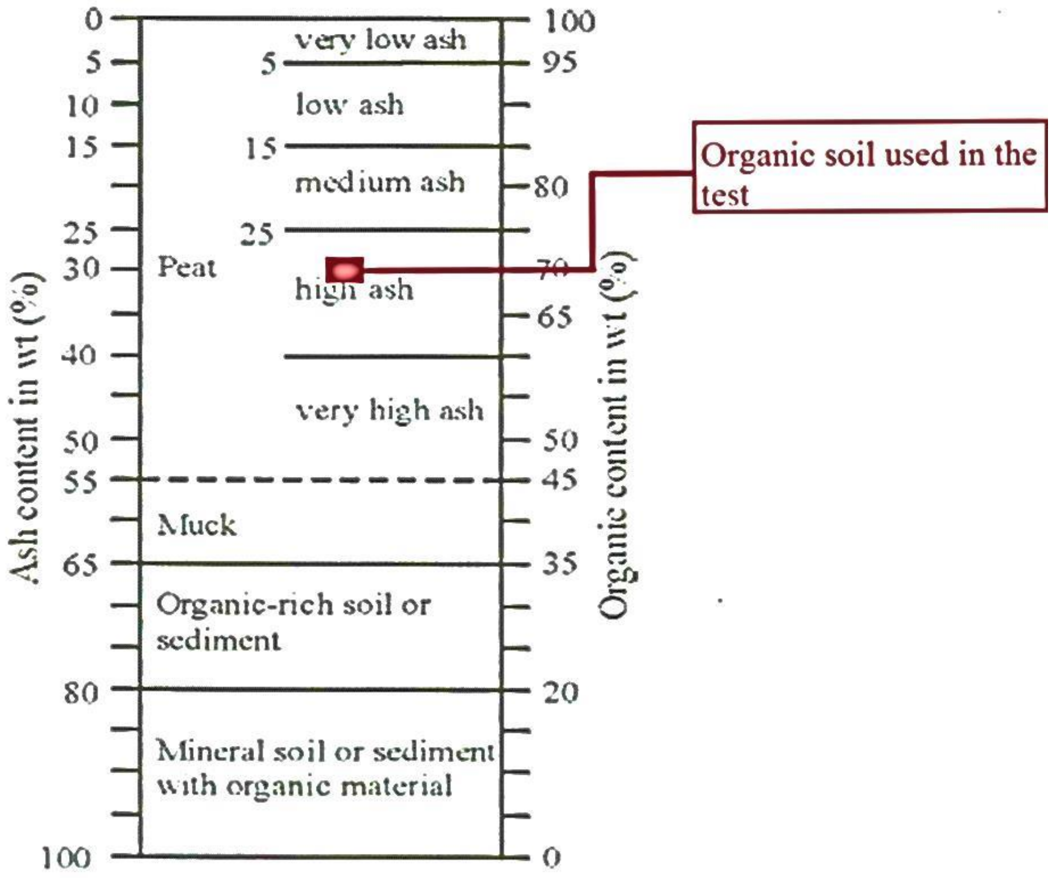


Figure 3.4: Classification system for peat deposits (Wüst et al., 2003)

Organic content was estimated by firing process at 440 ℃ in an oven for 4 hours according to ASTM D 2974. According to this process ash content of the soil was defined as 30 % and 70 % organic materials (Appendix A). Wet sieve analysis was carried out on ash and it was found that soil contains 10 % silt and clay, 20 % sand. Liquid limit of the organic soil was estimated by fall cone test according to ASTM D 4318 and found to be 119 % (Appendix A). The organic soil can be classified as fibric (ASTM D 1997), high ash (ASTM D 2974), moderately acidic (ASTM D 2976) and H_1- H_4on degree of humification (Von Post, 1922). According to ASTM Standards Soil Classification System was given in Table 2.4. Close up view of the organic soil used in this study was given in Figure 3.5 and 3.6.

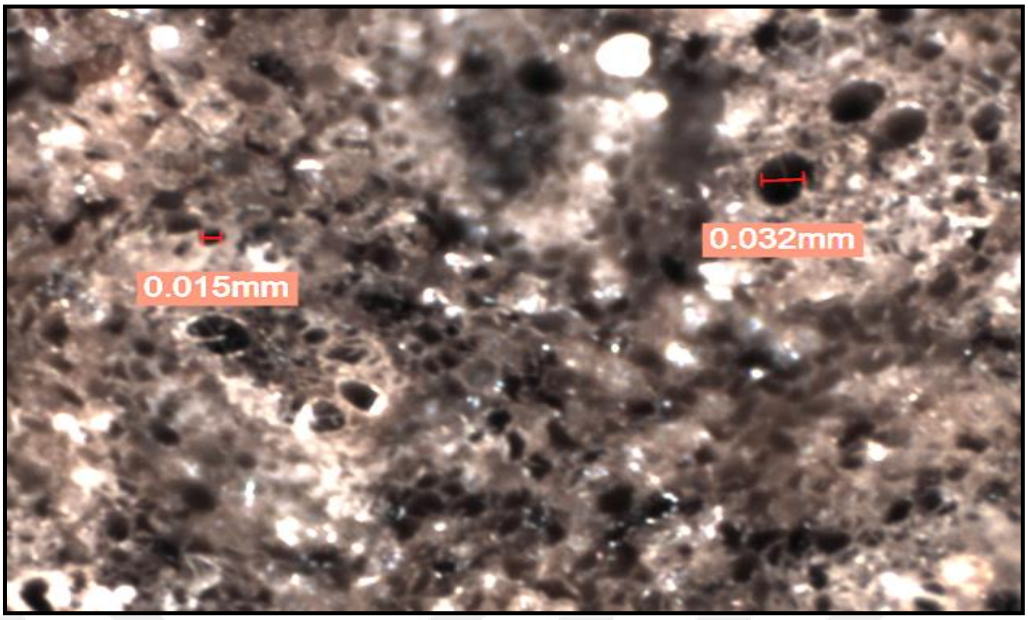


Figure 3.5: Close up view of organic soil (By LEICA Z16 APO electronic Microscope.)

Figure 3.6: Organic soil and sand used in this study

The peat is further classified with respect to fiber content because the content of fiber shows some alternatives in the consolidation process of fibrous peat from that of organic soil or amorphous peat. If fiber content of peat is less than 20 % these types of soils are called as amorphous peat (ASTM D 4427). It has particles of colloidal size less than 2 microns, and the pore water can be kept around the particle Surface. The behavior of amorphous granular peat has some similarities with clay soil. If any peat has fiber content more than 20 % these soils are called as fibrous peat according to ASTM D 4427.

3.3 Laboratory Tests

3.3.1 Consolidation Test

The standard consolidation test on Oedometer cell was conducted as preliminary tests to estimate the consolidation behavior of the fibrous peat samples. In all tests ELE marked consolidation test machine was used. The tests are carried out based on the standard procedure outlined in BS 1377-5. The Oedometer cell is 50 mm in diameter and 20 mm in height (Figure 3.7) since the sample was taken from shallow depth (1 to 2 m), and subsequently the in-situ stress is very low, then the consolidation test started at a very low pressure. The test is conducted with load increment ratio (LIR) of half, and applied loads were 25 kPa, 50 kPa, 100 kPa, 200 kPa, and 400 kPa. Each load was maintained for one day or 1440 minutes for loading stages during the first tests, but was modified to 9 days upon determination of the end of primary consolidation.

The equipment used in the test are consolidation device (including ring, porous stones, water reservoir, and load plate), dial gauge (0.0001 inch = 1.0 on dial), clock, moisture can, filter paper, sensitive balance accuracy 0.01 g Figure 3.7.

Figure 3.7: Standard consolidation assembly of all components of Oedometer test

The standard consolidation test was conducted on 15 samples. The sand column diameters used in this test were (1.7cm, 2.5cm, 3.5cm,). Table 3.1 shows test programs performed in this test.

Table 3.1: Sand column diameter and area orientations for consolidation test

Test No.	**Sand column diameter (cm)**	**S/O ratio (%)**
1	0	0
2	1.7	11.56
3	2.5	25
4	3.5	49
5	5	100

The brief procedures of the consolidation test conducted to obtain the compressibility characteristic of peat soil in Sakarya region are as follows:

1) The initial mass of peat soil and sand prepared for the test are weighed. The weights of sand and peat soils in the mold were summarized in Table 3.2

2) The consolidometer assembled, the consolidation cell placed in the load frame then the bottom porous stone and the ring were placed.
3) Test samples were prepared in the following order. First, the predetermined thin tube located at the centre of the cell. Second the fibrous peat was placed around the tube to fill the cell, and then the sand was loosely filled in the predetermined thin tube. Finally the thin tube holding sand was pulled out, the top porous stone and loading cap placed after that figure 3.8.
4) The consolidation cell filled with water.
5) The loading block centrally positioned on the top porous stone. Mounted the assembly on the loading frame and the dial gauge placed in position.
6) The first load applied (25 kPa) simultaneously, the valve opened (by quickly lifting the toggle switch to the up (open) position) the timing clock started and the dial gauge readings recorded at 0, 0.25, 1, 2.25, 4.0, 6.25, 9.0, 12.25, 16.00, 20.25, 25.00, 36, 49, 64, 81, 100, 121, 144, 169, 196, 225, 256, 289, 324, 361, 400, 500, 600, and 1440 minutes.
7) The above steps repeated for 50 kPa, 100 kPa, 200 kPa and 400 kPa loading pressures and for unloading pressures of 200 kPa, 100 kPa, 50 kPa and 25 kPa.
8) The specimen removed from the consolidation ring, and placed in the previously weighed moisture can. The moisture can placed containing the specimen in the oven and dried for 12 to 18 hours.

Table 3.2: Weights of sand and organic soil in the mold for all sand column diameters (Oedometer test)

Diameter of sand column (cm)	**Weight of sand in the mold (g)**	**Weight of organic in the mold(g)**
0		49.78
1.7	7.22	43.73
2.5	15.62	36.65
3.5	30.61	25.22
5	62.5	

Figure 3.8: Steps for sample preparation for consolidation test

3.3.2 Direct Shear Test

The direct shear tests were done by using the fibrous peat and sand column the UU and CU method were used. In all tests ELE marked direct shear test machine was used with preparing different sand column Figure 3.9. The sample specimens consisted of fibrous peat and sand column were prepared according to ASTM D 3080-03. The direct shear test was conducted on 30 samples, the sand column diameters used in this test were (2.5cm, 3.5cm, and 4.7cm), Table 3.3 shows test programs performed in this test.

Table 3.3: Sand column diameters and area orientations for direct shear test

Test No.	Sand column diameter (cm)	S/O ratio (%)
1	0	0
2	2.5	13.62
3	3.5	26.71
4	4.7	48.16
5		100

The test procedures of conducting direct shear test is done to the BS 1377: Part 7:1990 clause 4.5. The brief procedures of the direct shear test conducted to obtain the shear strength parameters of peat soil in Sakarya region are as follows:

1) The initial mass of peat soil prepared for the test is weighed. The weights of sand and peat soils in the mold are summarized in table 3.4.
2) The width and height of the shear box are measured.
3) The shear box is carefully assembled and placed in the direct shear device. Then retaining plate, porous stone and perforated plate are placed.
4) Test samples were prepared in the following order. First, the fibrous peat was placed around the predetermined thin tube located at the center of the shear box apparatus to fill it. Then, the sand was loosely filled in the tube. Finally the thin tube holding sand was pulled out and the perforated plate and porous

plate are placed after that. Figure 3.9 shows different sand column diameters incorporated in fibrous peat in the direct shear box.

5) Finally, the loading pad is placed on top.
6) The large alignment screws are removed from the shear box.
7) The assembly of the direct shear devices is completed and the three gauges consist of horizontal displacement gauge, vertical displacement gauge and shear load gauge are set, and then filled with water.
8) The vertical load is set to a predetermined value. In this experiment the value of the vertical load are 5 kg, 10 kg and 20 kg respectively. After the vertical load setting, the load is applied to the soil by raising the toggle switch.
9) The test is started with the selected speed so that the rate of shearing is at selected constant rate (1 mm/min. speed rate was used in the test). The values of the horizontal displacement gauge, vertical displacement gauge and shear load gauge readings are obtained through the electronic data logger connected to the direct shear device. Figure (3.10).
10) The readings are taken after 1hour (UU) and 24 hour (CU) for each sample until the horizontal shear load reached peak.
11) The moisture content of tested peat soils is obtained.

Table 3.4: Weights of sand and peat soil in the mold for all sand column diameters (Direct Shear test)

Diameters of sand column (cm)	**Weight of sand in the mold (g)**	**Weight of organic in the mold(g)**
0		127.23
2.5	19.97	109.45
3.5	39.14	92.87
4.7	70.6	65.7
	159.55	

Figure 3.9: Different sand column diameters incorporated in fibrous peat in the direct shear box.

Figure 3.10: All procedures of direct shear test

CHAPTER IV

RESULTS AND DISCUSSIONS

This chapter reports the results of standard laboratory tests carried out on peat obtained from Sakarya region in Turkey. The tests were done to identify the general characteristics of the soil including water content, specific gravity, and initial void ratio. Organic content and fiber content are used to determine the classification of the peat. Fibrous peats have macro pores and micro pores inside of them according to this information the organic soil used for the thesis was divided into two parts as fibrous peat and amorphous granular peat. In all tests fibrous peats were used. The other properties discussed in this chapter are the shear strength, and compressibility obtained from the standard consolidation test on Oedometer cell

4.1 Physical Properties

The preliminary identification of the soil was made based on the index properties and classification tests conducted on six samples. Index properties include the determination of water content, specific gravity, bulk unit weight, and the initial void ratio. The summary of index properties is presented in Table 4.1 while the results of each index test are presented in Appendix A.

The average natural water content obtained from laboratory tests is 236 % which is considered high compared with mineral soil. This value is within the range (200-700 %) (Huat, 2004) .

Table 4.1: The summary of index properties of peat soil in Sakarya region in Turkey

	Parameters	**Results of this study**
Index properties	Natural moisture content (%)	236
	Specific Gravity(Gs)	1.97
	Bulk unit Weight (kN/m^3)	11.2
	Dry unit Weight(kN/m^3)	3.33
	Initial void ratio (e_0)	4.6
	Acidity (pH)	4.5

The average specific gravity obtained using kerosene on pycnometer test is 1.97 and it is within the range for fibrous peat. As shown in Figure 4.1, for water content of 236 %, specific gravity of about 1.97.

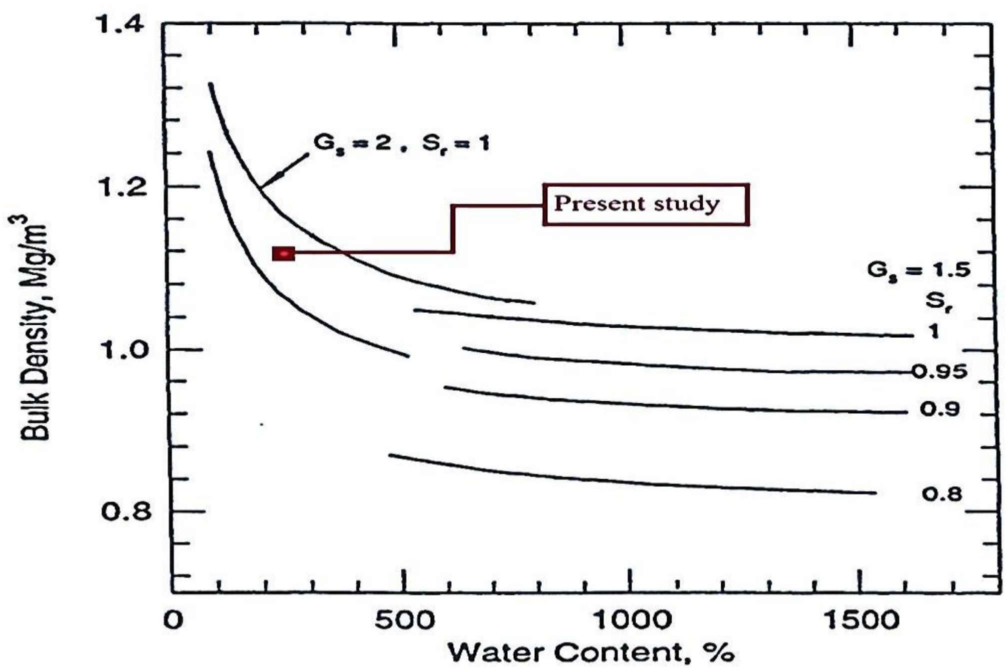


Figure 4.1: Correlation of bulk density, water content, specific gravity, and degree of saturation of fibrous peat (Hobbs, 1986)

The average bulk unit weight of the peat is 11.2 kN/m^3 which give a bulk density of 1.12 Mg/m^3 (Figure 4.1). This value is within the range (8.30-11.50 kN/m^3) (Huat, 2004). The dry unit weight of the peat is 3.33 kN/m^3

The average void ratio for the fibrous peat obtained in this study is 4.6 and this is within the range (3-15) predicted by Huat (2004). The void ratio also includes the volume of gas generated during decomposition process. The test results showed that the average pH value of the fibrous peat used in this study is 4.5 which is in the range (3.0-4.5) predicted by Muttalib et al. (1991).

4.2 Classification

The peat in this study was classified based on the degree of humification (von Post scale) and the organic and the fiber content. The von post scale is based on the appearance of soil water that is extruded when a sample of the soil is squeezed in the hand. When brown water comes out from the soil and the soil left on the hand has a large amount of fiber, then the peat is classified as fibrous peat with range between H_1 - H_4 degrees of decomposition according to von post scale.

The organic content of the peat is found as 70 % which is quite high but still correlate well with its specific gravity and water content (Figure 4.2 and Figure 4.3). The loss of ignition or ash content is 30 %. The fiber content of 84 % is considered very high as compared to published data around the world (Table 4.2). The summary of the classification tests results are presented in Table 4.2.

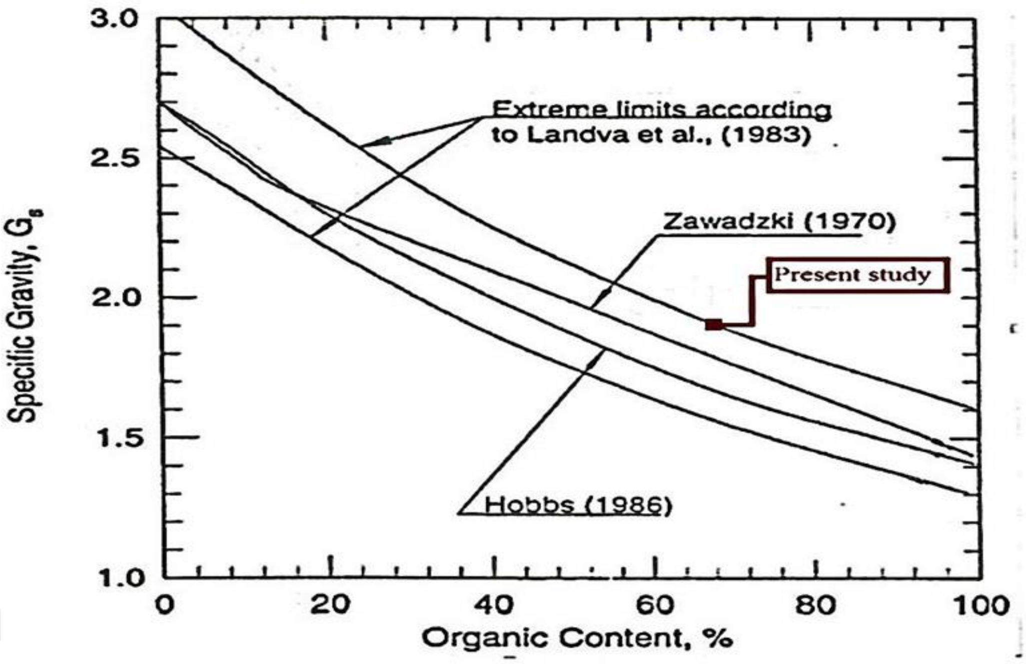


Figure 4.2: The range of organic content of fibrous peat based on specific gravity (Lechowicz et al., 1996)

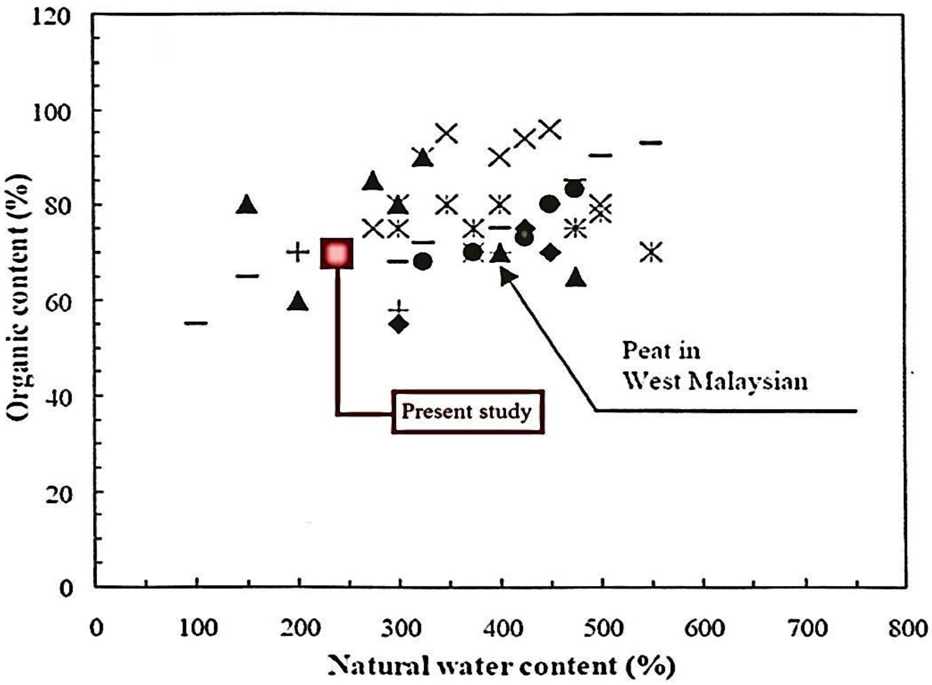


Figure 4.3: The range of organic content of fibrous peat based on water content (Al-Raziqi et al., 2003)

Table 4.2: The summary classification test results in Sakarya region in Turkey

Classification of peat soil based on ASTM standards		The peat soil used in this study
Fiber Content (ASTM D1997)	Fibric : Peat with greater than 67 % fibers	84.20 % (Fibric)
	Hemic : Peat with between 33 % and 67 % fibers	
	Sapric : Peat with less than 33 % fibers	
Ash Content (ASTM D2974)	Low Ash : Peat with less than 5 % ash	30 % (High Ash)
	Medium Ash : Peat with between 5% and 15 % ash	
	High Ash : Peat with more than 15 % ash	
Acidity (ASTM D2976)	Highly Acidic : Peat with a pH less than 4.5	4.5- 6.5 (Moderately Acidic)
	Moderately Acidic : Peat with a pH between 4.5and 5.5	
	Slightly Acidic : Peat with a pH greater than 5.5 and less than 7	
	Basic : Peat with a pH equal or greater than 7	
Degree of Decomposition (Von post,1922)	Between H_1 and H_{10}	H_1 - H_4 Fiber Ratio > 60 % (Hartlen and Wolski,1996)

4.3 Consolidation Test Results

Consolidation tests were undertaken using the conventional Oedometer. The study particularly focused on three consolidation parameters: coefficient of volume compressibility (m_v), primary compression index (c_c) and recompression index (c_r). The tests focused on effect of diameter of granular column in peat soil. Water content of the peat soil was kept constant for all tests. It was 119 % that is liquid limit value.

Three different types of sand column diameters were used; 1.7cm, 2.5cm and 3.5cm in fibrous peat.

The pressures applied to the soil sample are 25 kPa, 50 kPa, 100 kPa, 200 kPa and 400kPa. Each pressure is maintained for 24 hours or 1440 minutes. During this time, deformation of specimen was observed in specified time (e.g. ¼, ½, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, 1440 minutes). The results were presented in term of e - log σ' curve, the time-compression curve, primary compression index (c_c) - sand column curve, recompression index (c_r) - sand column curve, coefficient of volume compressibility (m_v) - σv curve, compression ratio ($c_c/(1+e_0)$) - sand column curve and recompression ratio ($c_r/(1+e_0)$) - sand column curve. The e-log σ curves of samples have mentioned above are shown in Figure 4.4.

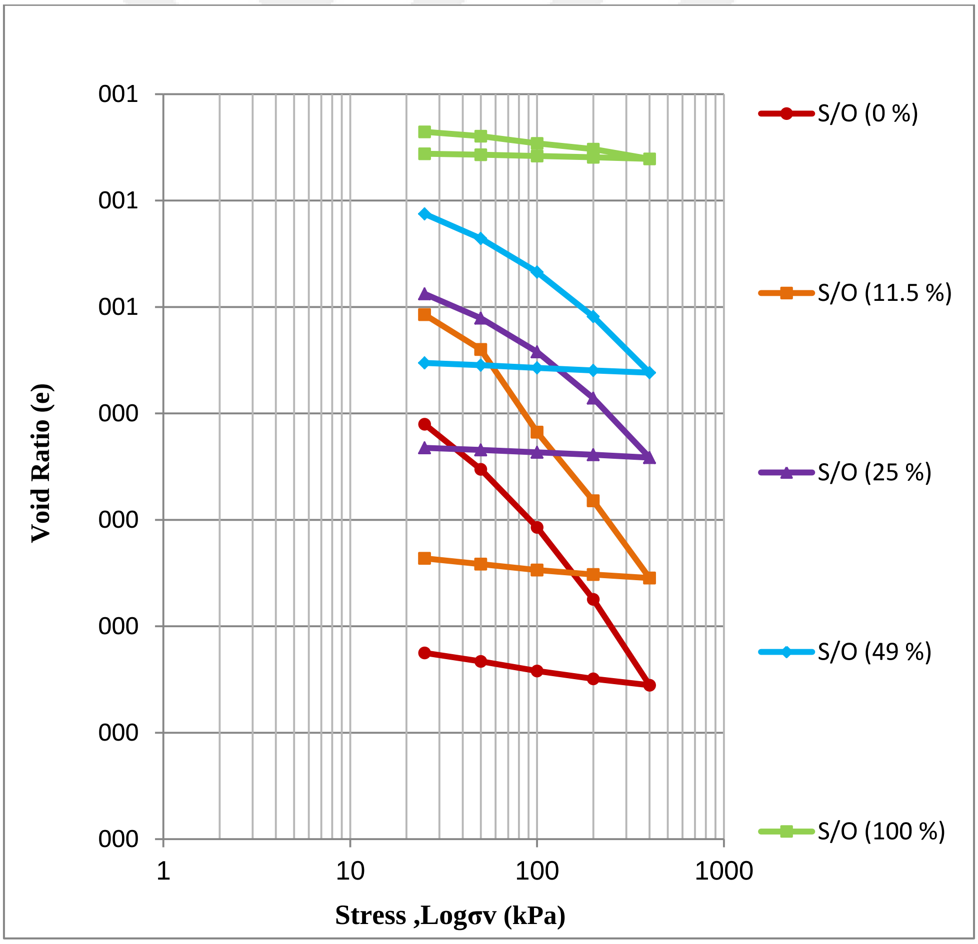


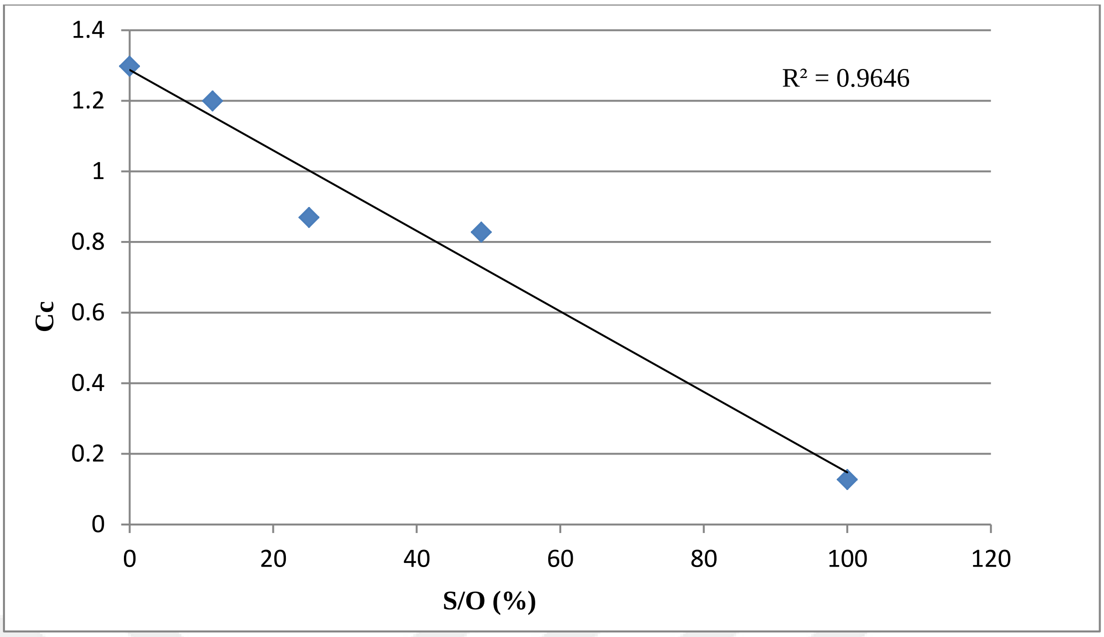
Figure 4.4: e versus log σ curves of all sand column in fibrous peat

Table 4.3: Consolidation characteristics of all sand column in fibrous peat under normal stresses ranging between 25 and 400 kPa

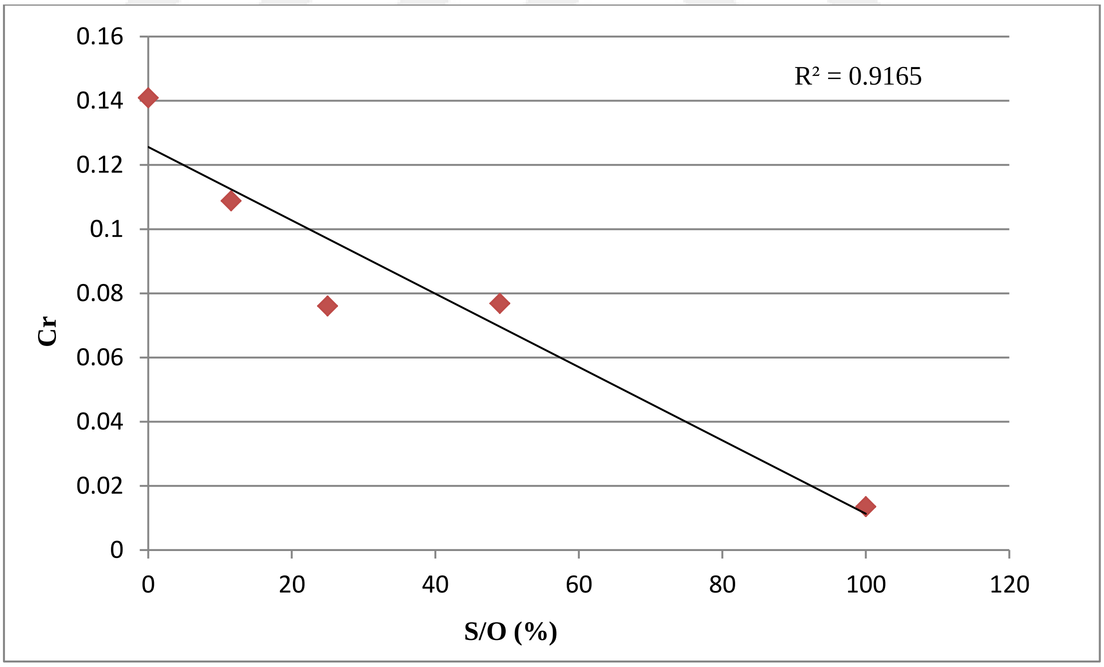
Sand column diameters(cm)	$\mathbf{c_c}$	$\dfrac{\mathbf{c_c}}{\mathbf{1+e_0}}$	$\mathbf{c_r}$	$\dfrac{\mathbf{c_r}}{\mathbf{1+e_0}}$	**m_v (m^2/kN))**
0	1.2978	1.0037	0.1409	0.1013	0.0042-0.00033
1.7	1.1993	0.8676	0.1088	0.0729	0.0023-0.0003
2.5	0.8693	0.5963	0.076	0.0502	0.0015-0.0002
3.5	0.8278	0.54	0.0768	0.0484	0.0011-0.0002
5	0.1271	0.0769	0.0135	0.0081	0.00058-0.000027

The values of c_c in Table 4.3 were estimated from the linear part of the e-log σ curves of each specimen tested .The ranges are generally close to the lower bounds of those given in the literature (Duraisamy, 2007; Huat, 2004).

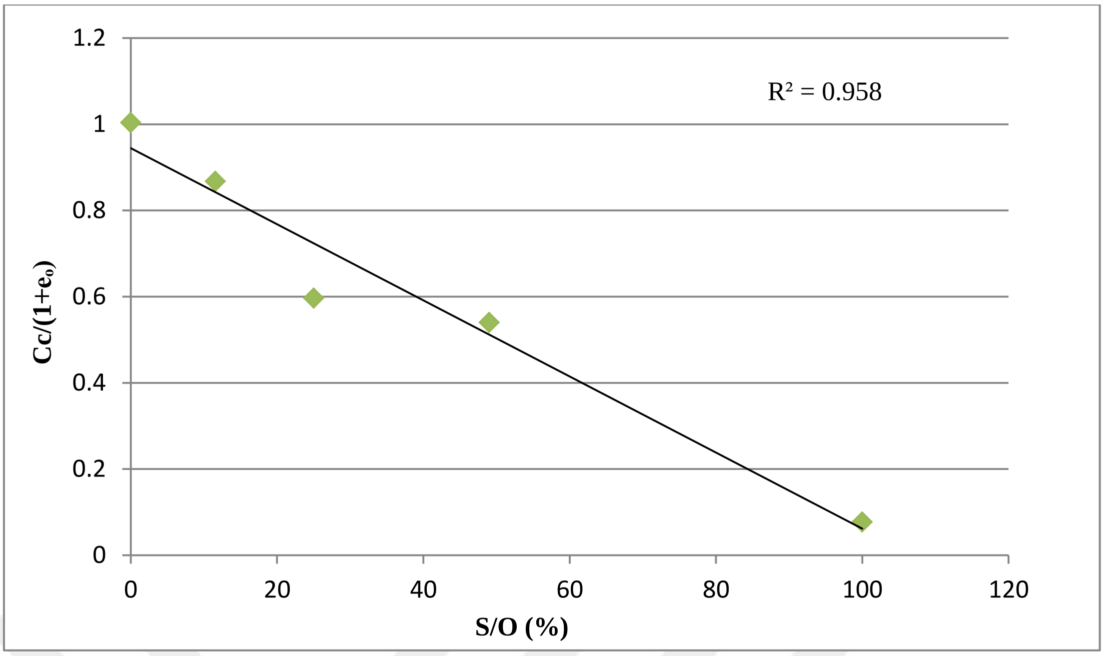
Parameter c_c/ (1+ e_0) is called compression ratio. According to O'Loughlin and Lehane (2003), compression ratio for peat in the range of 0 to 0.05 is classified as very slightly compressible followed by slightly compressible for anything in between 0.05 to 0.10. Moderately compressible peat lies in the range of 0.10 to 0.20 and very compressible peat has ratio in between 0.20 to 0.35. Based on the compression ratios given in Table 4.3 this value is decrease with increasing sand column diameters Figure 4.10 show the effect of sand column on the primary compression index (c_c), recompression index(c_r),compression ratio c_c/(1+e_0) and recompression ratio c_r/ (1+e_0).


1.4
1.2
1
0.8
0.6
0.4
0.2
0
Cc
0
20
40
60
80
100
120
S/O (%)
R² = 0.9646

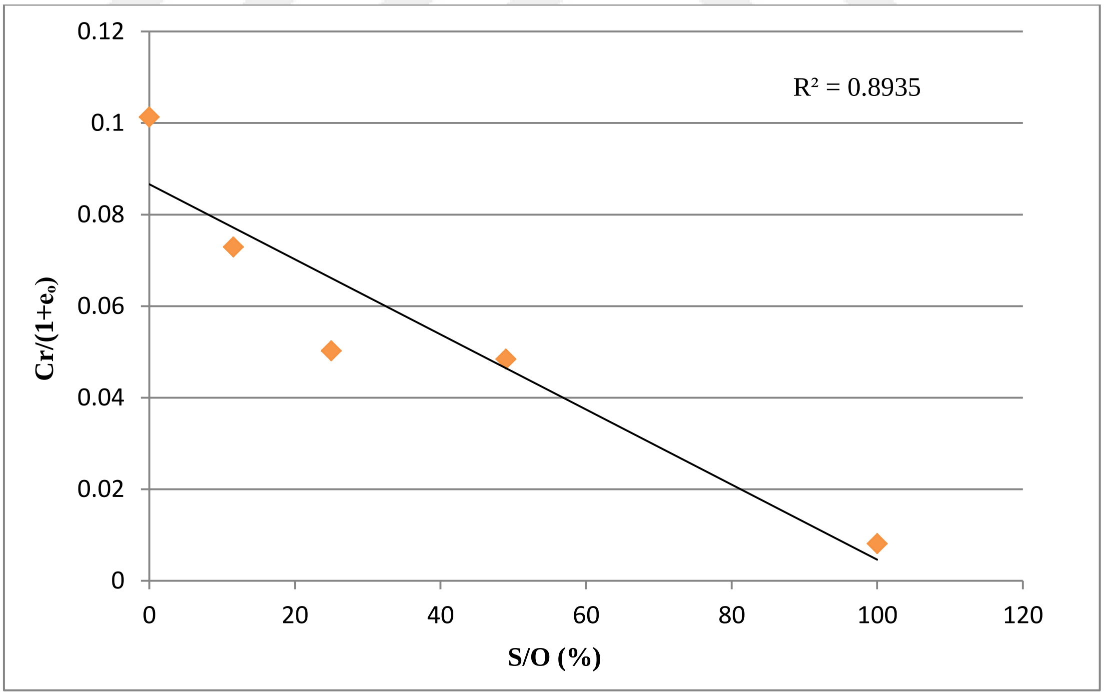
(a)


0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
Cr
0
20
40
60
80
100
120
S/O (%)
R² = 0.9165

(b)



(c)



(d)

Figure 4.5: Effect of sand column on the a-) primary compression index (c_c), b-) recompression index (c_r), c-) compression ratio and d-) recompression ratio

It can be seen in Figure 4.5 when sand column diameter increases the primary compression index, recompression index, compression ratio and recompression ratio decrease. The primary compression index (c_c) of organic soil is 1.29 .When S/O % are 11, 25 and 49 (c_c) decreased to 1.19, 0.86 and 0.82 respectively. And when S/O % is 100 (c_c) decreased to 0.12. The recompression index (c_r) of organic soil is 0.14 .When S/O % are 11, 25 and 49 (c_r) decreased to 0.1, 0.076 and 0.07 respectively and when S/O % is 100 (c_r) decreased to 0.01. Figure 4.6 shows the variation of m_v with consolidation pressure and indicates that mv exhibits an exponential decrease with increase in stress. Also the variation of (m_v) decrease with increases in sand column diameters.

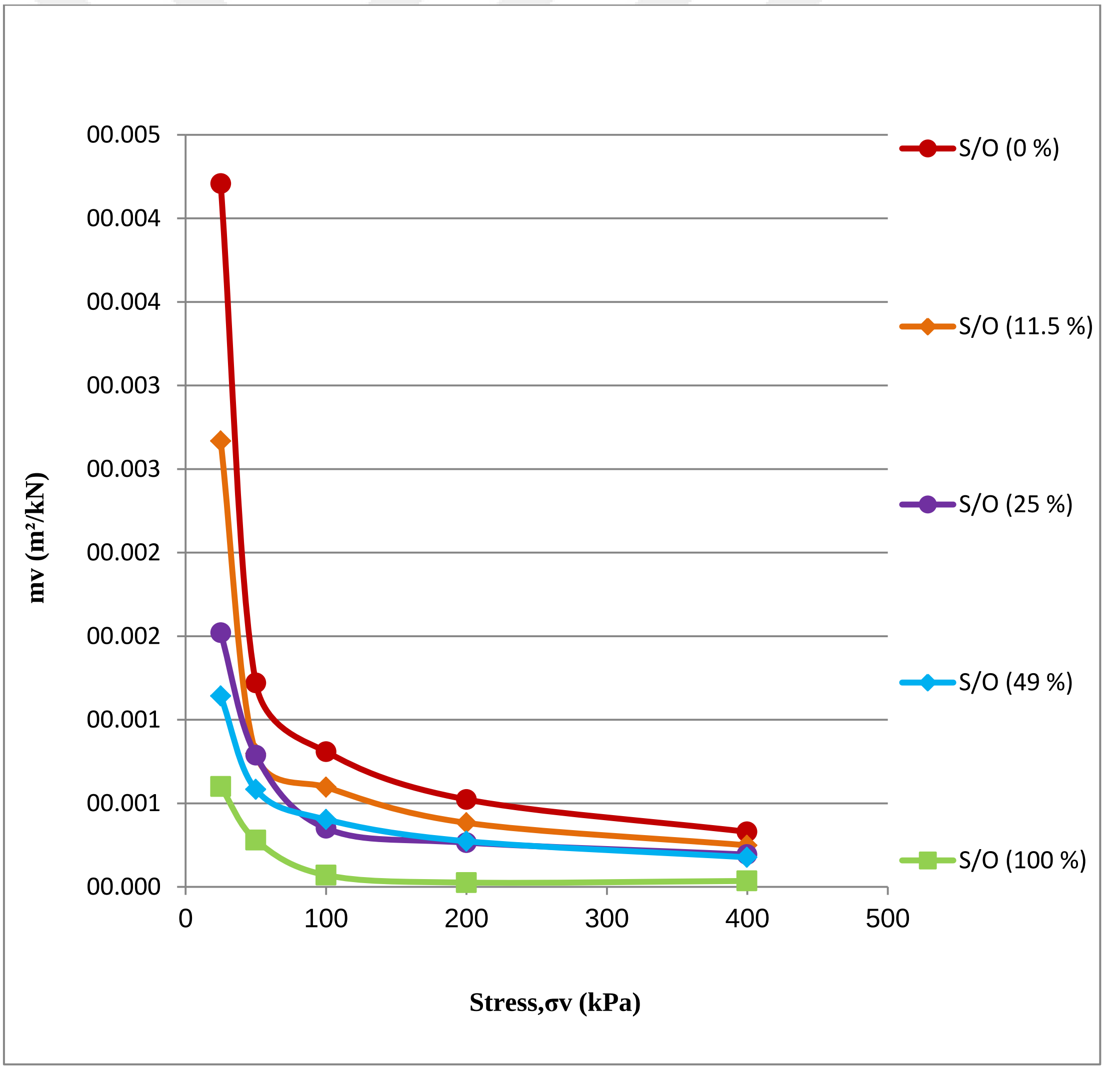


Figure 4.6: Typical m_v -σv graphs of different sand column in fibrous peat

Comparison between the two methods in previous studies on peats (Sing et al. 2008a, b) showed that the compression curves for peats best fit the casegrande's method theoretical curve, hence this method was used in the present study. The results of Oedometer test attached in Appendix B.

4.4 Direct Shear Test Results

Ten sets of test containing three soil samples were tested using the direct shear apparatus. Each soil samples has the length of 60mm, width of 60mm, and thickness of 20mm. The area of the sample is 3600 mm^2.

Results obtained from the direct shear test were used to analyze the shear strength parameters of different diameters of sand column in fibrous peat from Sakarya regin in Turkey. In order to determine the cohesion value (c) and angle of internal friction (φ).The diameters of sand column used in direct shear test were (2.5cm, 3.5cm and 4.7cm). The normal stresses used for direct shear test were (26.38 kPa, 40.27 kPa and 68 kPa). The results of direct shear test data analysis is attached in Appendix C.

Direct shear test data of samples were analyzed to obtain the shear strength parameters of the soil. Firstly, in order to draw a stress – strain curve, the direct shear data were analyzed to obtain the shear stress (τ). The shear stress was calculated by dividing the value of shear force from the direct shear test to the cross sectional area of the test specimen. Shear stresses for each set of test samples were obtained by the same calculation. Then, graphs contained shear stress versus horizontal displacement of each set is plotted. All the three curves set of the test with each consist of different loading is plotted on the same axes.

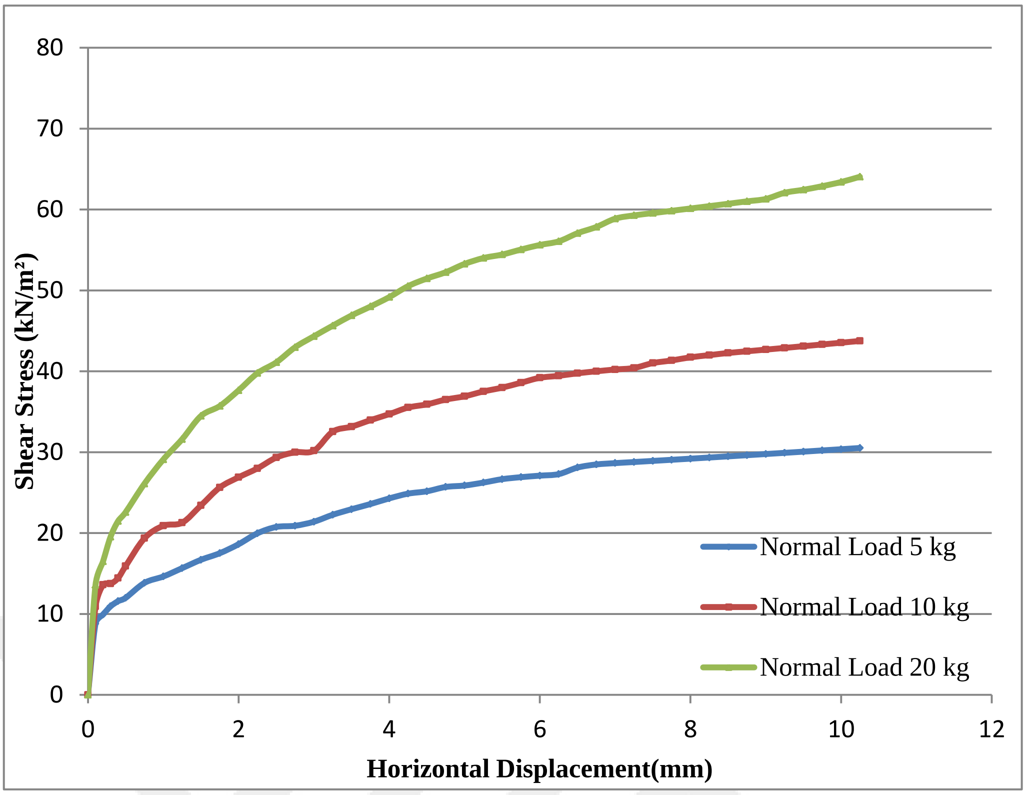
Typical curve for shear stress versus horizontal displacement for a set of tests is shown in Figure 4.7. Table 4.4 and 5.5 shows the maximum shear strength obtained for each test and the data plotted in Figure 4.7.

Table 4.4: Maximum shear stress values from test (UU)

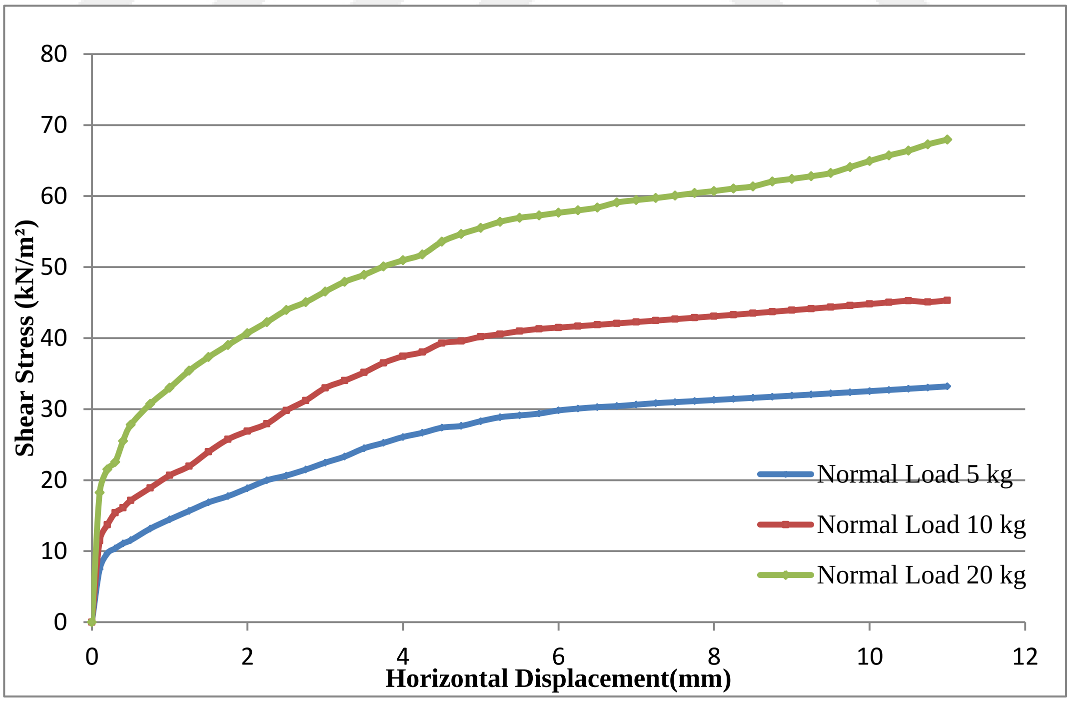
UU							
(S/O) %	**Sand Column Diameters (cm)**	**Normal Stress (kPa)**			**Maximum Shear Stress (kPa)**		
		5 kg	**10 kg**	**20 kg**	**5 kg**	**10 kg**	**20 kg**
0	0	26.38	40.27	68.05	30.9	42.5	60.6
13.62	2.5				30.5	43.8	64.1
26.71	3.5				32.7	44.4	68
48.16	4.7				27.2	44.9	63.9
100	totally sand				30.8	42.4	61.4

Table 4.5: Maximum shear stress values from test (CU)

CU							
(S/O) %	**Sand Column Diameters (cm)**	**Normal Stress (kPa)**			**Maximum Shear Stress (kPa)**		
		5 kg	**10 kg**	**20 kg**	**5 kg**	**10 kg**	**20 kg**
0	0	26.38	40.27	68.05	31.9	41.5	65
13.62	2.5				32.9	44.3	68.5
26.71	3.5				33.7	43.3	69.8
48.16	4.7				28.6	41.2	65.7
100	totally sand				30.8	42.4	61.4



(UU)



(CU)

Figure 4.7: Shear stress versus horizontal displacement for 2.5 sand colum for UU and CU tests

An important feature encountered in Figure 4.7 was that a failure as defined by peak shear stress was not observed in the tested samples. As can be seen, shear resistance increases with displacement during the test, and the slope of the curves increases with the applied normal pressure which is likely to be due to the fibers effect. A similar conclusion was also made by (K. BADV AND T. SAYADIAN 2012). A graph consisted of shear stress at failure (τ_f), was plotted against the corresponding normal stress (σ_n), A line that best fit through the corresponding points of the graph is drawn the effect of the sand column in the organic soil in different diameter on the shear strength parameters is seen clearly in Figure 4.8 and Figure 4.9.

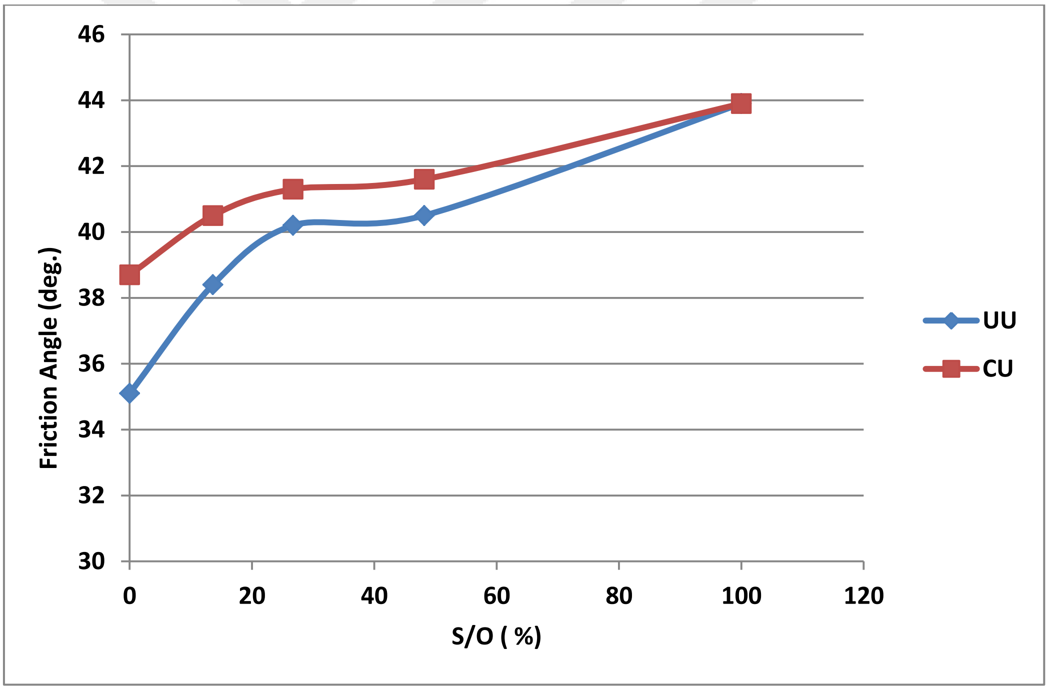


Figure 4.8: Sand column – Internal friction angle relation

As it is shown in Figure 4.8 while the diameters of sand column in the organic soil increase, Internal Friction Angle (IFA) increases. It can be seen IFA of (CU) test more than (UU) test. Initially there is a big difference in IFA between the two methods then it decreases until the two curves closed to each other. At the beginning the deference is 10.2 % then it decreases to 5.4 %, 2.72 % and 2.7 % respectively .IFA of organic soil is 35.1° for (UU) test and 38.7 ° for (CU) test. When S/O is 13.6 %, 26.7 % and 48.1 % increased IFA of the mixture to 38.4°, 40.2°, 40.5 and 43.9° for (UU) test and 40.5°, 41.3° and 41.6° for (CU) test gradually. And when S/O is 100 % IFA is 43.9°.

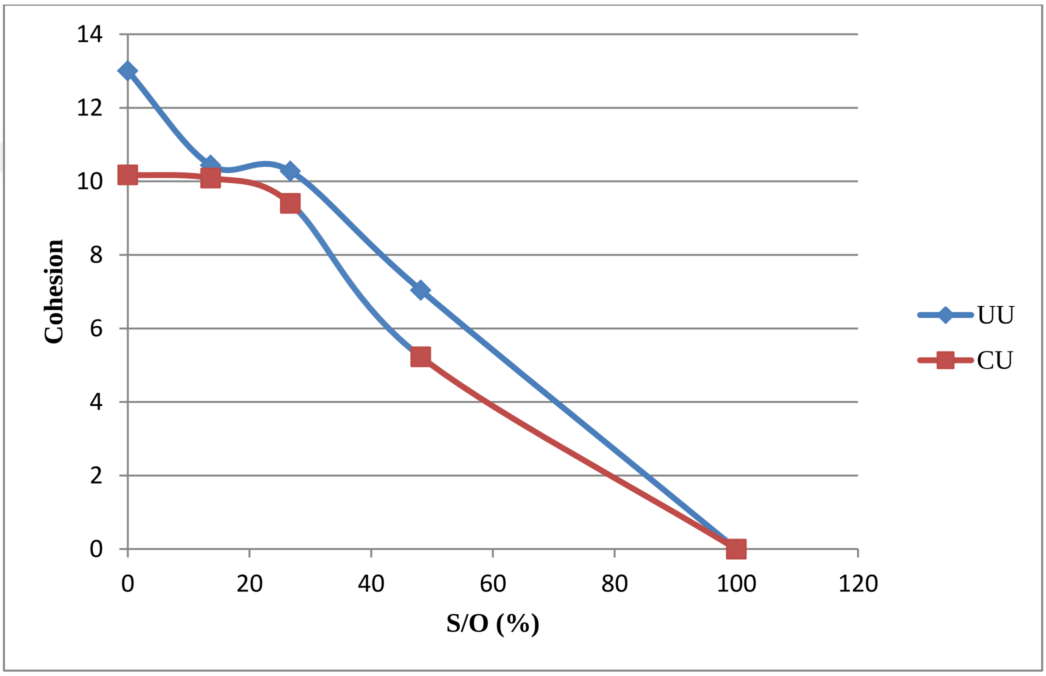


Figure 4.9: Sand column – Cohesion relation

As it is shown in Figure 4.9 while the diameters of sand column in the organic soil increase cohesion decreases. It can be seen cohesion obtained from (CU) t test less than the result obtained from (UU) test in all S/O ratios. There are no proportional ratio in change in cohesion depend on loading type. The cohesion of organic soil for both (UU) and (CU) tests is around 10-13 kPa the change is not too much for both type of loading of test .Table 4.6 and 4.7 contain shear strength parameters of direct shear test for all sand column diameters. Figure 4.12 shows all direct shear samples after testing.

Table 4.6: Shear strength parameters (c and φ) for UU test

(S/O) %	Sand Column Diameters	Parameters of Shear Strength (UU)	
		Cohesion, C (kPa)	**Friction Angle, φ°**
0	0	13.004	35.1
13.62	2.5	10.44	38.4
26.71	3.5	10.278	40.2
48.16	4.7	7.0414	40.5
100	totally sand	0	43.9

Table 4.7: Shear strength parameters (c and φ) for CU test

(S/O) %	Sand Column Diameters	Parameters of Shear Strength (CU)	
		Cohesion, C (kPa)	**Friction Angle, φ°**
0	0	10.177	38.7
13.62	2.5	10.09	40.5
26.71	3.5	9.403	41.3
48.16	4.7	5.228	41.6
100	totally sand	0	43.9

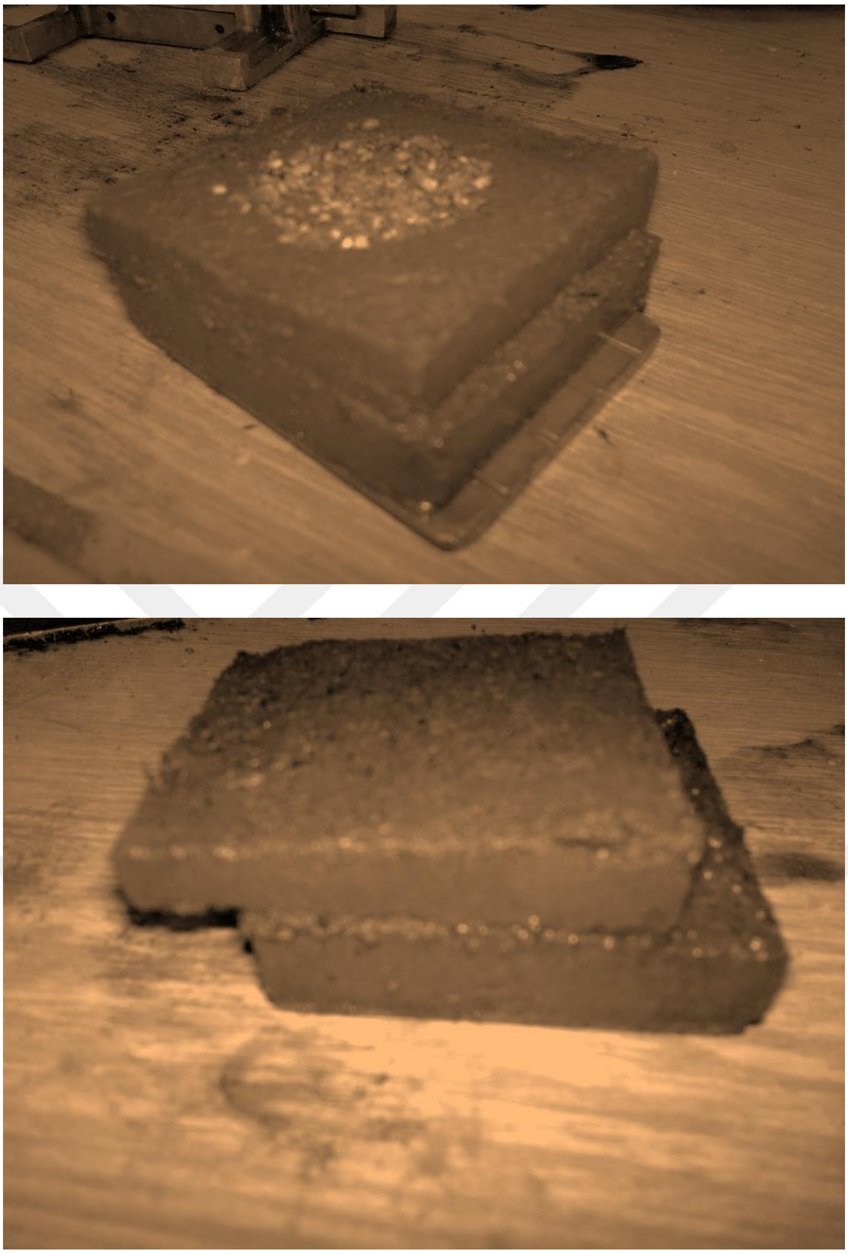


Figure 4.10: Samples of direct shear after testing

CHAPTER V

CONCLUSION

5.1 Conclusion

Conclusions are derived based on the test results obtained from the current research on fibrous peat with sand column, and data from the literature. The conclusions of this study are indicated in the followings:

1) According to Von Post Scale, peat is classified as (H_1- H_4) which is of low to medium degree of decomposition with fibrous structure and easily recognized plant, the natural water content of the peat is 236 % which corresponds to initial void ratio of about 4.6 with high organic and fiber content . Based on ASTM D4427 classification, the peat is classified as fibrous peat.
2) The incoparation of sand column in peat soil has an important effect on consolidation parameters: primary compression index (c_c), recompression index (c_r) and coefficient of volume compressibility (m_v). When the ratio of sand column surface to organic soil surface area (S/O) increases, primary compression index (c_c) and recompression index (c_r) of the mixture decrease.
3) The primary compression index (c_c) of organic soil is 1.29 when S/O % are 11, 25 and 49 (c_c) decreases to 1.19, 0.86 and 0.82 gradually. And when S/O % is 100 (c_c) decreases to 0.12.
4) The recompression index (c_r) of organic soil is 0.14 .When S/O % are 11, 25 and 49 (c_r) decreases to 0.1, 0.076 and 0.07 gradually and when S/O % is 100 (c_r) decreases to 0.01.
5) The variation of (m_v) with consolidation pressure indicates that m_v exhibits an exponential decrease with increase in stress. Also the variation of (m_v) decrease with increase in sand column diameters.

6) The incorporation of sand column in organic soil has an important effect on shear strength parameters of organic soils. When the ratio of sand column surface to organic soil surface area (S/O) increases, internal friction angle of the mixture also increases and cohesion of the mixture decreases.
7) IFA obtained from (CU) test is more than (UU) test. Initially there is a big difference in IFA between the two methods then it decreases. At the beginning the deference is 10.2 % then it decrease to 5.4 %, 2.72 % and 2.7 % gradually with increase in S/O ratio .IFA of organic soil is 35.1° for (UU) test and 38.7 ° for (CU) test. When S/O is 13.6 %, 26.7 % and 48.1 % IFA of the mixture are 38.4°, 40.2 ° and 40.5 ° for (UU) test and 40.5 °, 41.3° and 41.6° for (CU) test gradually. And when S/O % is 100 IFA is 43.9 °.
8) Cohesion obtained from CU test is less than the result obtain from UU test in all S/O ratios, cohesion of organic soil is 13 kPa for (UU) test, and 10.1 kPa for (CU) test. When S/O is 13.6 %, 26.7 % and 48.1 % cohesion of the mixture are 10.4 kPa, 10.2 kPa and 7 kPa for (UU) test, and 10 kPa, 9.4 kPa and 5.2 kPa for (CU) test gradually. And when S/O is 100 % cohesion is zero.
9) Peat soil has unique characteristics because of their high compressibility and low shear strength and there is a tendency in construction to avoid this type of problematic soils. So proper soil stabilization method which is economical and needs less time can overcome this type of problem. Stabilization of this soil by sand column leads to increase of it is shear strength and decrease of it is deformation. Therefore the improvement method can be used to improve bearing capacity and control exceeds settlement quantities constructed on organic soil.

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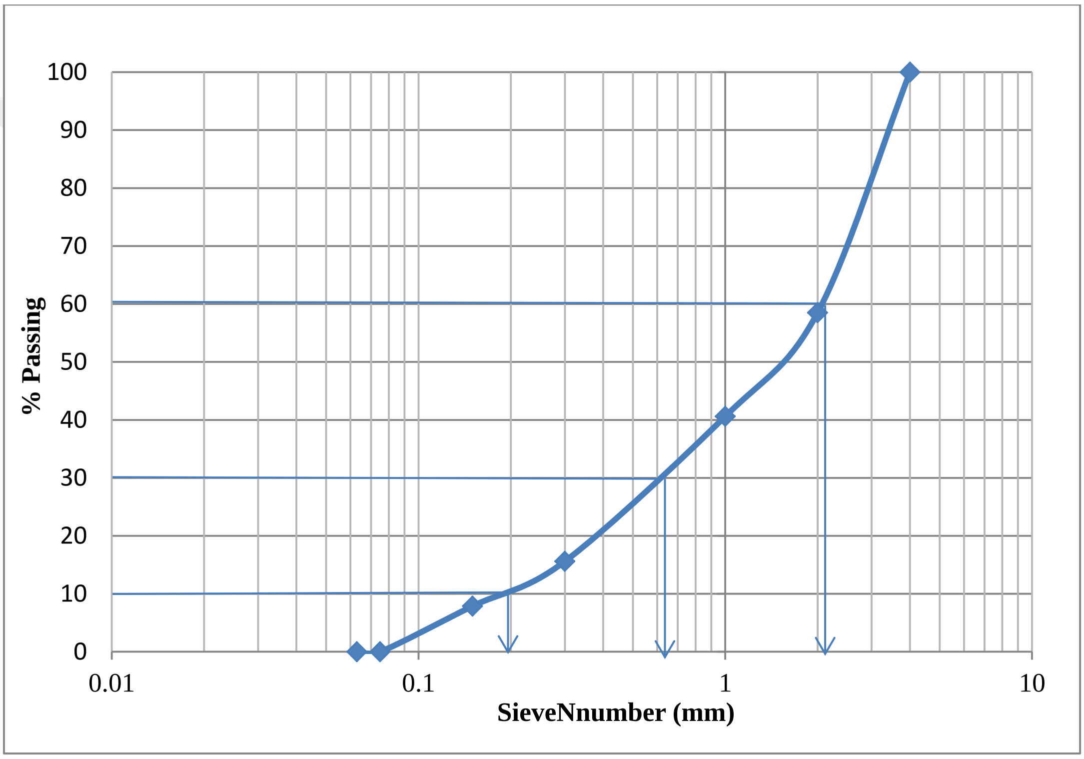
APPENDIX A

INDEX TEST DATA

Sieve Analysis Test



From the graph	
D10 (maximum size of the smallest 10% of the sample) =	0.2mm
D30 (maximum size of the smallest 30% of the sample) =	0.64mm
D60 (maximum size of the smallest 60% of the sample) =	2.1mm
(Cu) = D60/D10	10.5
(Cc) = (D30)2 / (D10 x D60)	0.97
(Cu) =10.5 > 6 and (Cc) =0.97 < 1	
According to Unified Soil Classification System the graded of sand is (PW)	

Organic content and Ash content

Specimen number	**1**	**2**
Mass of empty, clean porcelain dish (g) (M1)	212.6	212.5
Mass of the dish and dry soil (g) (M2)	301	261
Mass of the dish and ash (Burned soil) (g) (M3)	239	227.4
Organic content (OC) =((M2-M3)/(M2-M1)) *100	70.3	69.3
Average OC	70%	
Ash content (AC)= 100 % -OC %	AC=100-70= 30%	

A1: Oven for burning organic soil

Initial Void Ratio

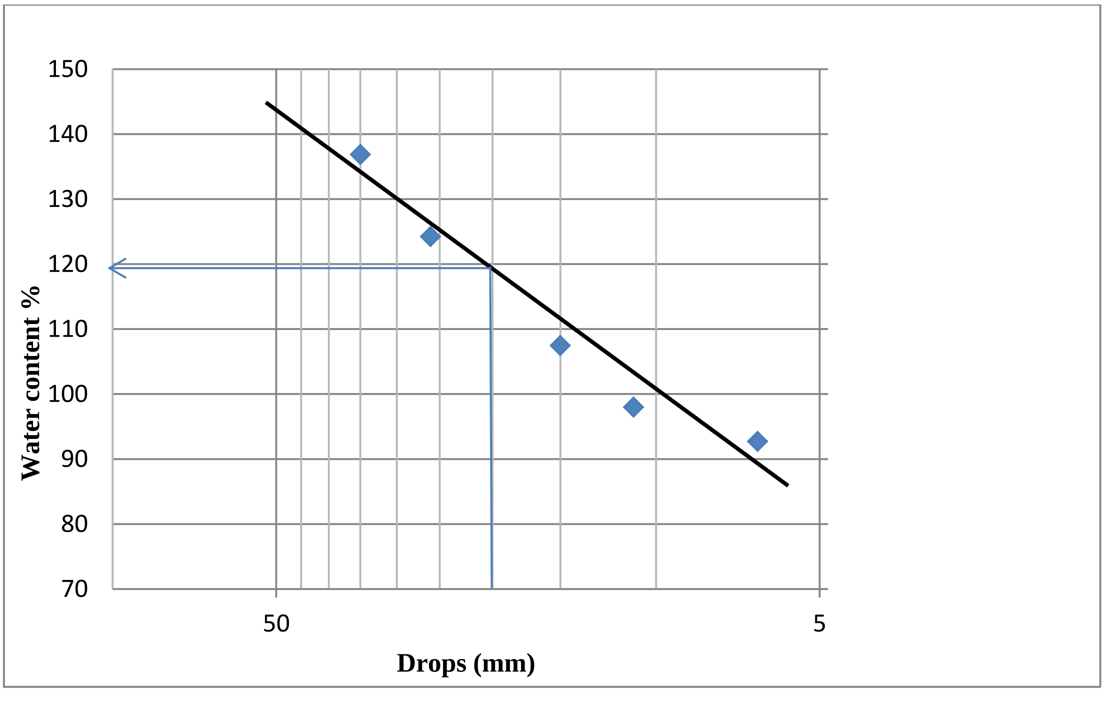
Based on average natural moisture content & average specific gravity:

$e_0 = (G_s * W)/\gamma_w$

$e_0 = (1.97*2.36)/10 = 4.64$

Liquid limit for organic soil by fall con test

Spacemen number	1	2	3	4	5
Drop (mm)	6.5	11	15	26	35
Mass of can (g) M1	9.33	6.5	6.63	8.98	8.6
Mass of can +soil (g) M2	15.42	16.32	26.07	20.46	32.57
Mass of can +dry soil (g) M3	12.49	11.46	16	14.1	18.72
w.c% =(M2-M3)/(M3-M1)	92.72	97.98	107.47	124.22	136.86



A6: Drops verses Water content for L.L determination

From the graph liquid limit (L.L) of organic soil =119%

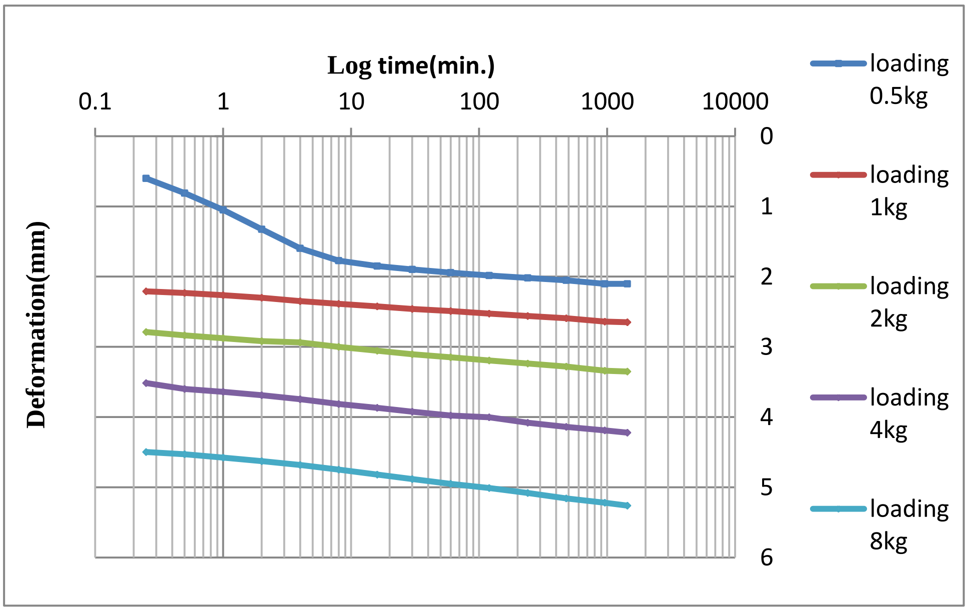
A2: Fall cone test

APPENDIX B

CONSOLIDATION TEST RESULTS

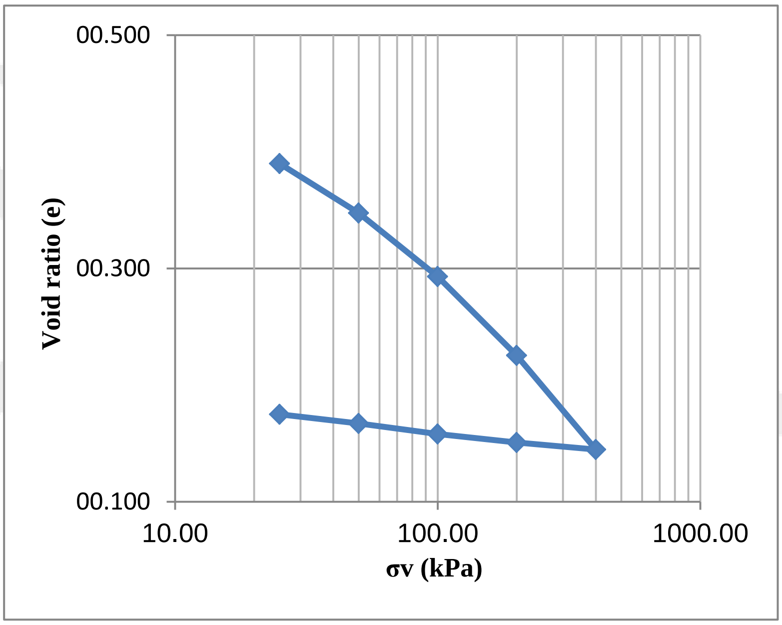
Sample description: Organic soil

Time (min.)	Deformation (mm)								
	Loading					Unloading			
	0.5kg	1kg	2kg	4kg	8kg	4kg	2kg	1kg	0.5kg
0	0	2.103	2.649	3.351	4.2215	5.261	5.183	5.0885	4.973
0.25	0.6	2.21	2.788	3.516	4.498	5.219	5.152	5.068	4.9605
0.5	0.81	2.232	2.837	3.601	4.531	5.218	5.151	5.067	4.96
1	1.05	2.265	2.877	3.641	4.577	5.213	5.149	5.062	4.9595
2	1.325	2.302	2.918	3.689	4.628	5.211	5.148	5.06	4.958
4	1.597	2.348	2.936	3.746	4.683	5.209	5.141	5.058	4.953
8	1.772	2.388	3.001	3.814	4.7495	5.204	5.139	5.052	4.9505
16	1.849	2.425	3.055	3.87	4.8195	5.2	5.132	5.0475	4.942
30	1.898	2.46	3.106	3.923	4.883	5.1995	5.13	5.0415	4.9395
60	1.943	2.491	3.149	3.979	4.952	5.196	5.128	5.035	4.938
120	1.985	2.526	3.195	4.003	5.01	5.191	5.1185	5.029	4.9295
240	2.019	2.56	3.238	4.081	5.081	5.189	5.112	5.0115	4.919
480	2.052	2.593	3.2805	4.14	5.1585	5.186	5.101	4.9895	4.901
960	2.102	2.64	3.34	4.19	5.22	5.184	5.09	4.98	4.88
1440	2.103	2.649	3.351	4.2215	5.261	5.183	5.0885	4.973	4.871

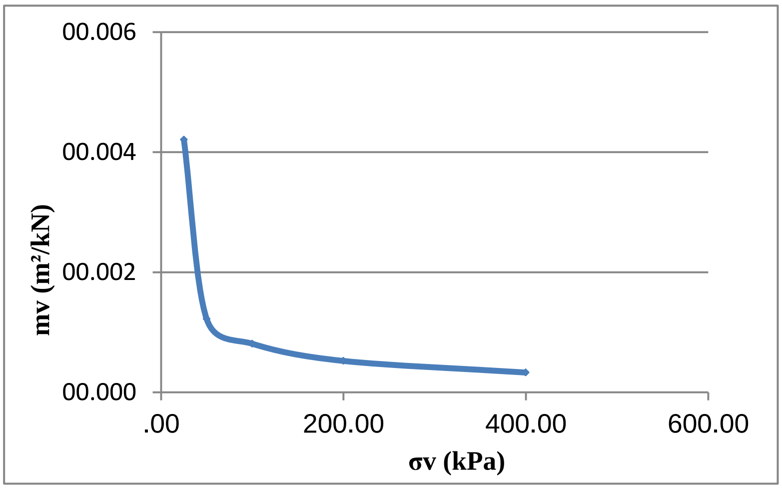


B1: Log time versus deformation

Gmix	Load (kg)	σv (kPa)	lnσv	Final dial reading (mm) (Settlement)	Hs (mm)	Change in specimen height (mm)	Final specimen height (mm)	Height of Voids (mm)	Final void ratio (e)	$C_c/(1+e_o)$	Cc	Cr	mv (m^2/kN)	$C_r/(1+e_o)$
1.97	0	0.0		0	12.876		20	7.124	0.5533	1.0037			0.0000	0.1013
1.97	5	25.0	3.2	2.103		2.103	17.897	5.021	0.3900				0.0042	
1.97	10	50.0	3.9	2.65		0.546	17.351	4.475	0.3476				0.0012	
1.97	20	99.9	4.6	3.351		0.702	16.649	3.773	0.2930				0.0008	
1.97	40	199.9	5.3	4.2215		0.8705	15.779	2.903	0.2254		1.2978		0.0005	
1.97	80	399.8	6.0	5.26		1.0395	14.739	1.863	0.1447				0.0003	
1.97	40	199.9	5.3	5.183		-0.078	14.817	1.941	0.1508					
1.97	20	99.9	4.6	5.0885		-0.094	14.912	2.036	0.1581			0.1409		
1.97	10	50.0	3.9	4.973		-0.116	15.027	2.151	0.1671					
1.97	5	25.0	3.2	4.87		-0.102	15.129	2.253	0.1750					



B2: e versus log σ curve of organic soil

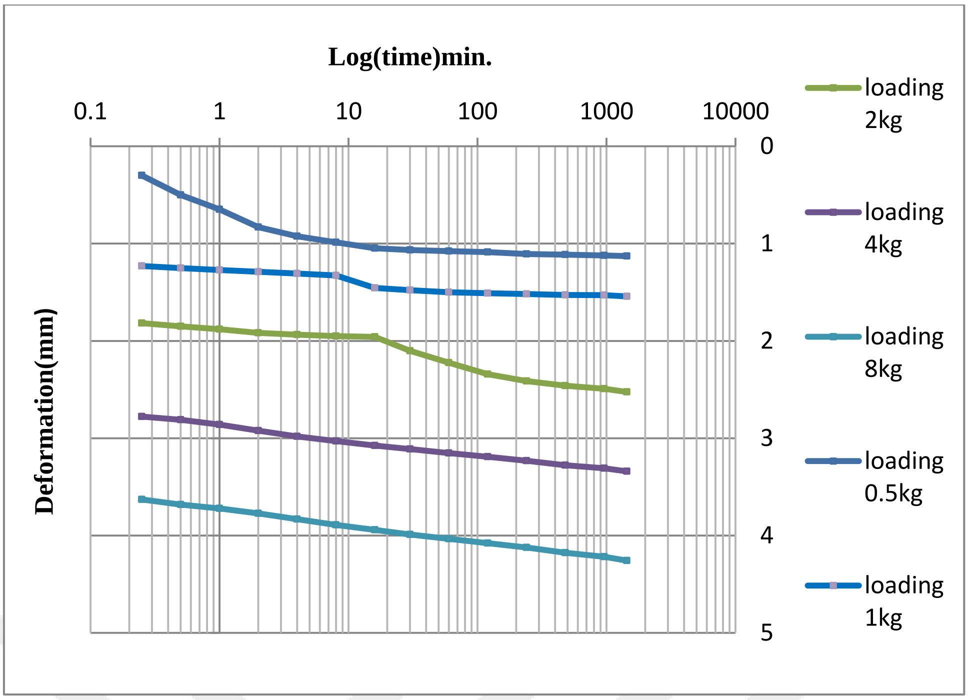


B3: σ versus m_v curve of organic soil

B4: Sample of organic soil

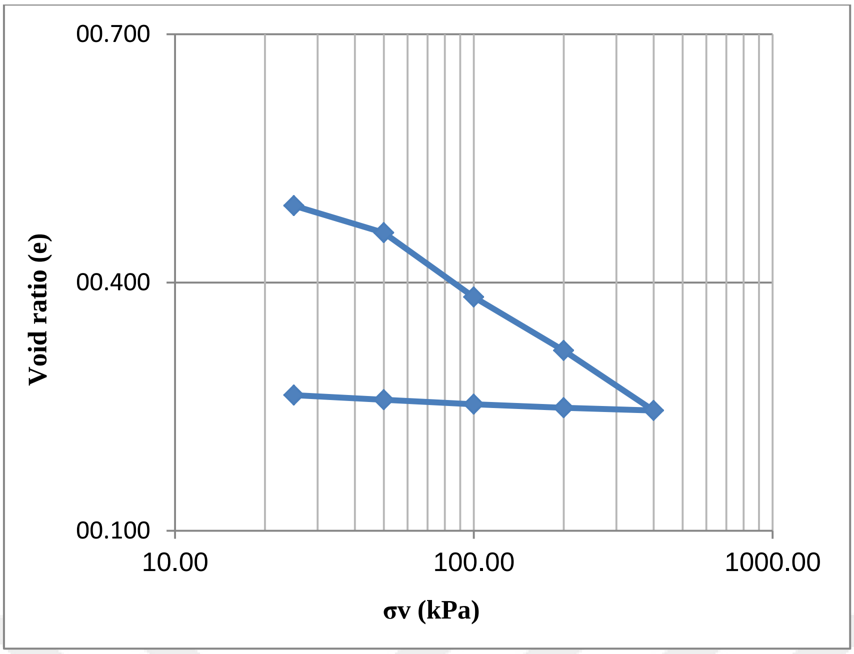
Sample description: Sand colum 1.7 cm

Time (min.)	Deformation (mm)								
	Loading					Unloading			
	0.5kg	1kg	2kg	4kg	8kg	4kg	2kg	1kg	0.5kg
0	0	1.128	1.542	2.524	3.339	4.257	4.214	4.161	4.092
0.25	0.3	1.231	1.818	2.778	3.629	4.222	4.189	4.14	4.078
0.5	0.5	1.252	1.85	2.81	3.682	4.221	4.188	4.138	4.075
1	0.648	1.272	1.879	2.86	3.721	4.2205	4.184	4.137	4.073
2	0.83	1.29	1.918	2.922	3.772	4.22	4.182	4.132	4.0715
4	0.925	1.308	1.936	2.982	3.831	4.2195	4.181	4.13	4.069
8	0.987	1.327	1.951	3.029	3.89	4.219	4.179	4.128	4.068
16	1.048	1.455	1.958	3.075	3.941	4.2185	4.178	4.125	4.063
30	1.066	1.478	2.101	3.112	3.99	4.218	4.177	4.1215	4.06
60	1.078	1.5	2.223	3.152	4.034	4.2175	4.173	4.121	4.058
120	1.088	1.51	2.342	3.19	4.078	4.217	4.171	4.111	4.052
240	1.107	1.518	2.413	3.231	4.123	4.216	4.169	4.109	4.047
480	1.114	1.528	2.46	3.277	4.177	4.2155	4.1675	4.1035	4.039
960	1.122	1.53	2.492	3.308	4.217	4.215	4.166	4.1	4.03
1440	1.128	1.542	2.524	3.339	4.257	4.214	4.161	4.092	4.022

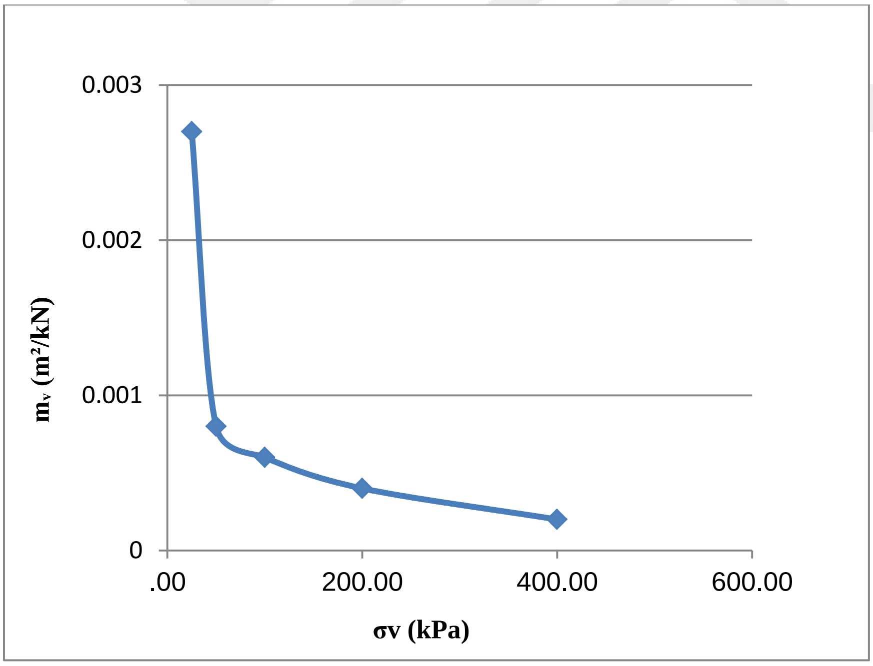


B5: Log time versus deformation curve of sand colum 1.7 cm

Gmix	Load (kg)	σv (kPa)	lnσv	Final dial reading (mm) (Settlement)	Hs (mm)	Change in specimen height (mm)	Final specimen height (mm)	Height of Voids (mm)	Final void ratio (e)	Cc	Cr	mv (m^2/kN)	Cc/(1+e_o)	Cr/(1+e_o)
2.0532	0	0.0		0	12.642		20	7.358	0.5820			0.0000		0.0729
2.0532	5	25.0	3.2	1.128		1.128	18.872	6.230	0.4928			0.0023		
2.0532	10	50.0	3.9	1.54		0.414	18.458	5.816	0.4601			0.0009		
2.0532	20	99.9	4.6	2.524		0.982	17.476	4.834	0.3824			0.0011		
2.0532	40	199.9	5.3	3.339		0.815	16.661	4.019	0.3179	1.1993		0.0005	0.8676	
2.0532	80	399.8	6.0	4.26		0.918	15.743	3.101	0.2453			0.0003		
2.0532	40	199.9	5.3	4.214		-0.043	15.786	3.144	0.2487					
2.0532	20	99.9	4.6	4.161		-0.053	15.839	3.197	0.2529		0.1088			
2.0532	10	50.0	3.9	4.092		-0.069	15.908	3.266	0.2584					
2.0532	5	25.0	3.2	4.02		-0.07	15.978	3.336	0.2639					



B6: e versus log σ curve of sand colum 1.7 cm

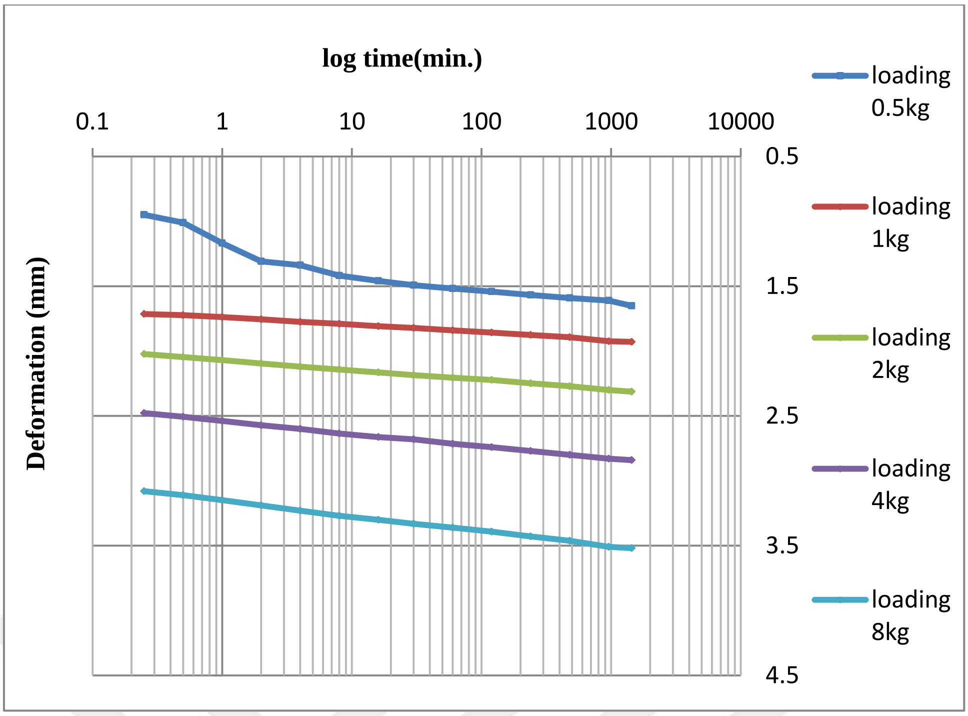


B7: σ versus m_v curve of sand colum 1.7 cm

B8: Sample of sand colum 1.7 cm

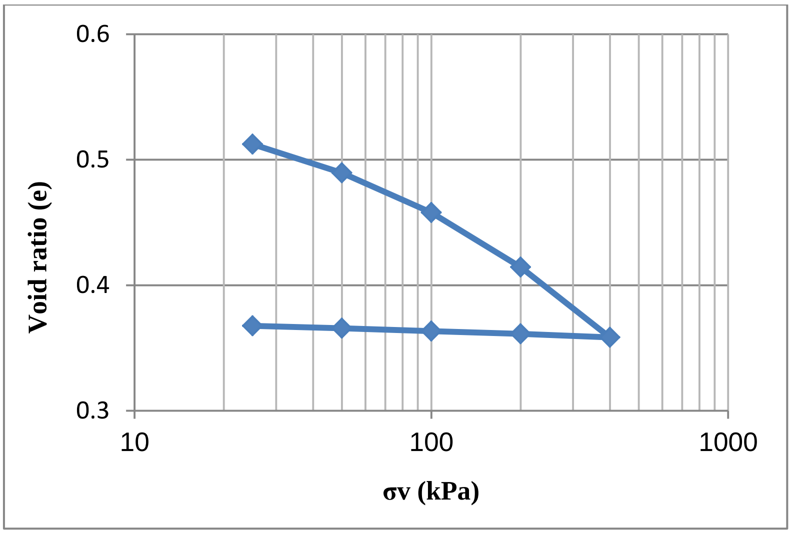
Sample description: Sand colum 2.5 cm

Time(min.)	Deformation (mm)								
	Loading					Unloading			
	0.5kg	1kg	2kg	4kg	8kg	4 kg	2 kg	1 kg	0.5 kg
0	0	1.652	1.9295	2.313	2.8405	3.5185	3.485	3.459	3.4305
0.25	0.95	1.715	2.023	2.478	3.08	3.4905	3.468	3.446	3.4215
0.5	1.01	1.723	2.046	2.506	3.111	3.49	3.467	3.446	3.421
1	1.168	1.738	2.07	2.538	3.149	3.49	3.4655	3.445	3.4205
2	1.309	1.755	2.096	2.572	3.19	3.4895	3.465	3.443	3.42
4	1.338	1.775	2.1205	2.6005	3.2295	3.489	3.464	3.4425	3.4195
8	1.419	1.79	2.142	2.636	3.2695	3.4885	3.4625	3.442	3.419
16	1.459	1.808	2.165	2.663	3.301	3.488	3.462	3.441	3.419
30	1.492	1.822	2.186	2.68	3.331	3.4875	3.4615	3.4405	3.4185
60	1.519	1.84	2.205	2.715	3.361	3.487	3.4605	3.4405	3.418
120	1.542	1.858	2.223	2.741	3.392	3.486	3.4605	3.4395	3.4165
240	1.568	1.876	2.248	2.77	3.429	3.4855	3.46	3.439	3.415
480	1.591	1.893	2.2705	2.7995	3.4625	3.4855	3.4595	3.435	3.4105
960	1.6105	1.924	2.3	2.829	3.509	3.485	3.459	3.43	3.407
1440	1.652	1.9295	2.313	2.8405	3.5185	3.485	3.459	3.4305	3.407

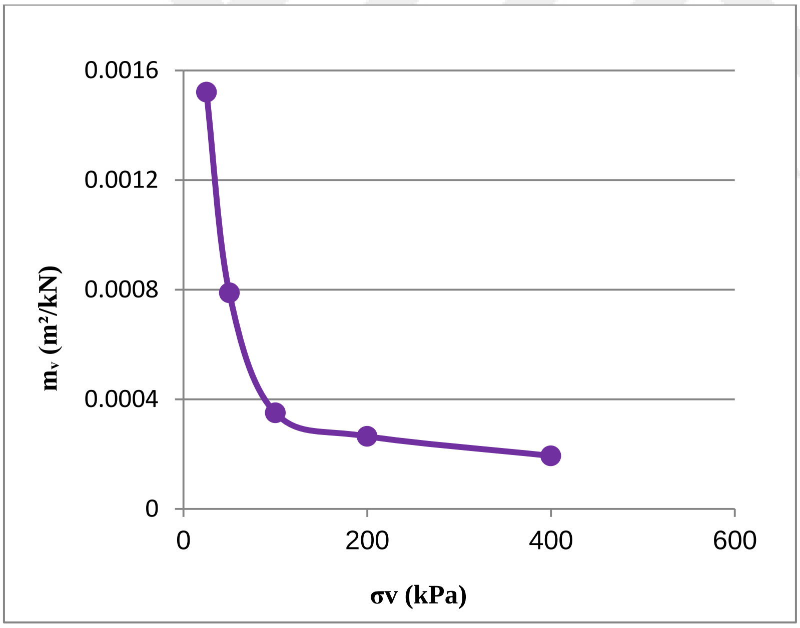


B9: Log time versus deformation curve of sand colum 2.5 cm

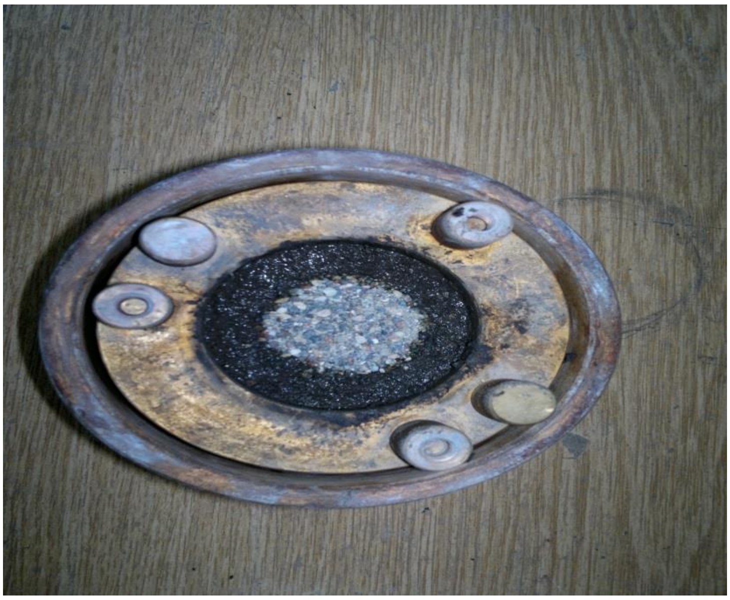
Gmix	Load (kg)	σv (kPa)	lnσv	Final dial reading (mm) (Settlement)	Hs (mm)	Change in specimen height (mm)	Final specimen height (mm)	Height of Voids (mm)	Final void ratio (e)	Cc	Cr	mv (m^2/kN)	Cc/(1+e_o)	Cr/(1+e_o)
2.15	0	0.0		0	12.132		20	7.868	0.6485			0.0000		0.0502
2.15	5	25.0	3.2	1.652		1.652	18.348	6.216	0.5123			0.0015		
2.15	10	50.0	3.9	1.93		0.2775	18.071	5.938	0.4895			0.0008		
2.15	20	99.9	4.6	2.313		0.3835	17.687	5.555	0.4579			0.0006		
2.15	40	199.9	5.3	2.8405		0.5275	17.16	5.027	0.4144	0.8693		0.0004	0.5963	
2.15	80	399.8	6.0	3.52		0.678	16.482	4.349	0.3585			0.0002		
2.15	40	199.9	5.3	3.485		-0.034	16.515	4.383	0.3613					
2.15	20	99.9	4.6	3.459		-0.026	16.541	4.409	0.3634		0.0760			
2.15	10	50.0	3.9	3.4305		-0.028	16.57	4.437	0.3658					
2.15	5	25.0	3.2	3.41		-0.024	16.593	4.461	0.3677					



B10: e versus log σ curve of sand colum 2.5 cm



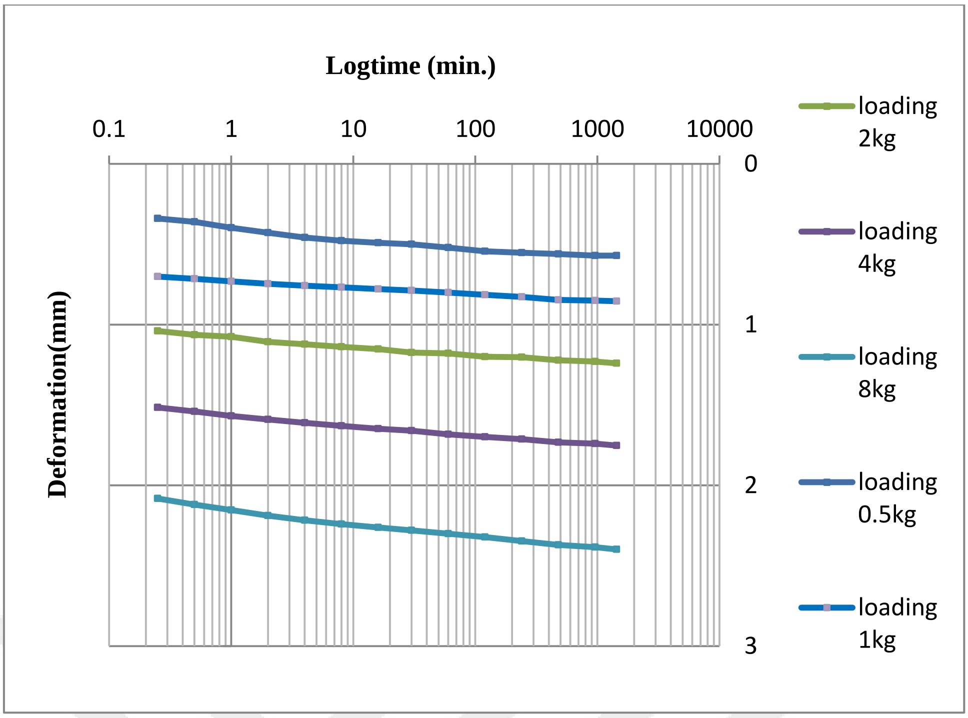
B11: σ versus m_v curve of sand colum 2.5 cm



B12: sample of sand colum 2.5 cm

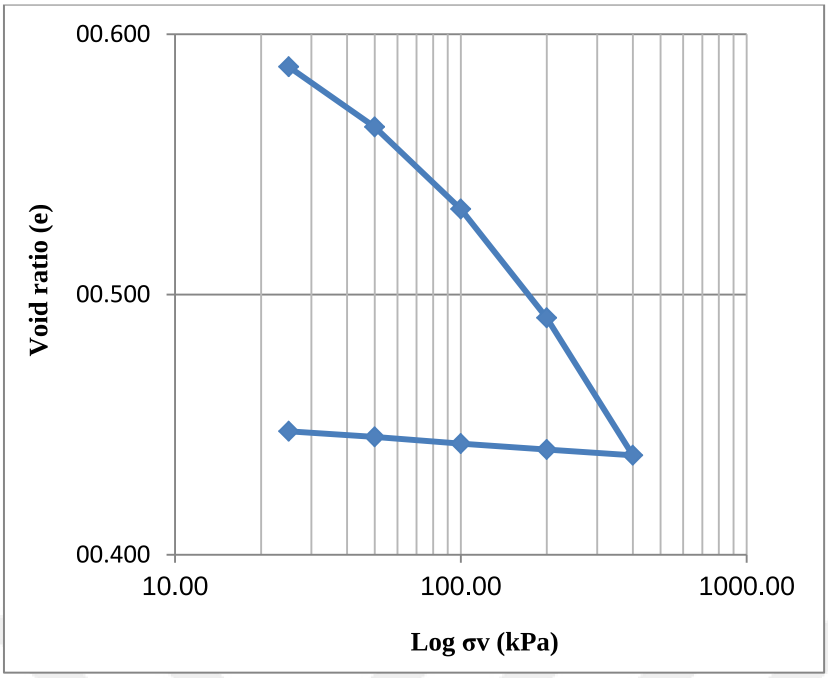
Sample description: Sand colum 3.5 cm

Time (min.)	Deformation (mm)								
	Loading					Unloading			
	0.5kg	1kg	2kg	4kg	8kg	4kg	2kg	1kg	0.5kg
0	0	0.571	0.854	1.24	1.752	2.398	2.371	2.343	2.3115
0.25	0.34	0.701	1.039	1.515	2.082	2.376	2.351	2.32	2.298
0.5	0.36	0.715	1.063	1.54	2.12	2.375	2.35	2.3195	2.2975
1	0.397	0.731	1.075	1.568	2.153	2.375	2.349	2.3195	2.297
2	0.428	0.746	1.107	1.59	2.188	2.375	2.349	2.319	2.2965
4	0.458	0.758	1.122	1.611	2.217	2.3745	2.3485	2.3185	2.296
8	0.478	0.768	1.138	1.63	2.241	2.374	2.348	2.318	2.294
16	0.491	0.779	1.152	1.648	2.262	2.3735	2.348	2.318	2.293
30	0.5	0.788	1.174	1.66	2.28	2.373	2.3475	2.318	2.2905
60	0.521	0.8	1.179	1.682	2.3005	2.373	2.3475	2.318	2.29
120	0.544	0.814	1.2	1.698	2.322	2.373	2.347	2.317	2.29
240	0.553	0.827	1.203	1.7125	2.346	2.3725	2.347	2.316	2.2895
480	0.561	0.846	1.222	1.732	2.37	2.3715	2.3455	2.315	2.2885
960	0.571	0.85	1.23	1.74	2.384	2.371	2.344	2.312	2.2875
1440	0.571	0.854	1.24	1.752	2.398	2.371	2.343	2.3115	2.285

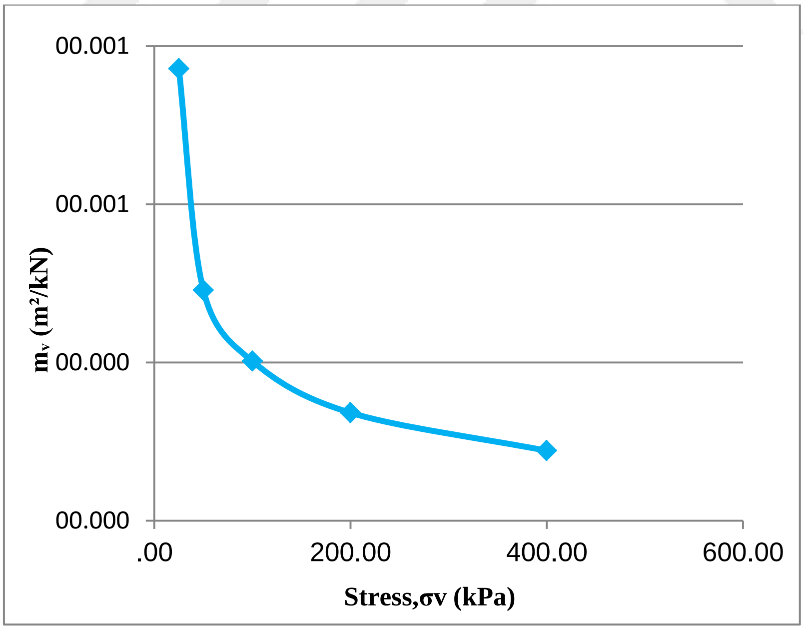


B13: Log time versus deformation curve of sand colum 3.5 cm

Gmix	Load (kg)	σv (kPa)	lnσv	Final dial reading (mm) (Settlement)	Hs (mm)	Change in specimen height (mm)	Final specimen height (mm)	Height of Voids (mm)	Final void ratio (e)	Cc	Cr	mv (m^2/kN)	Cc/(1+e_0)	Cr/(1+e_0)
2.3228	0	0.0		0	12.239		20	7.761	0.6342			0.0000		0.0484
2.3228	5	25.0	3.2	0.571		0.571	19.429	7.190	0.5875			0.0011		
2.3228	10	50.0	3.9	0.85		0.283	19.146	6.907	0.5644			0.0006		
2.3228	20	99.9	4.6	1.24		0.386	18.76	6.521	0.5328			0.0004		
2.3228	40	199.9	5.3	1.752		0.512	18.248	6.009	0.4910	0.8278		0.0003	0.5400	
2.3228	80	399.8	6.0	2.40		0.646	17.602	5.363	0.4382			0.0002		
2.3228	40	199.9	5.3	2.371		-0.027	17.629	5.390	0.4404					
2.3228	20	99.9	4.6	2.343		-0.028	17.657	5.418	0.4427		0.0768			
2.3228	10	50.0	3.9	2.3115		-0.031	17.689	5.450	0.4453					
2.3228	5	25.0	3.2	2.29		-0.027	17.715	5.476	0.4475					



B14: e versus log σ curve of sand colum 3.5 cm

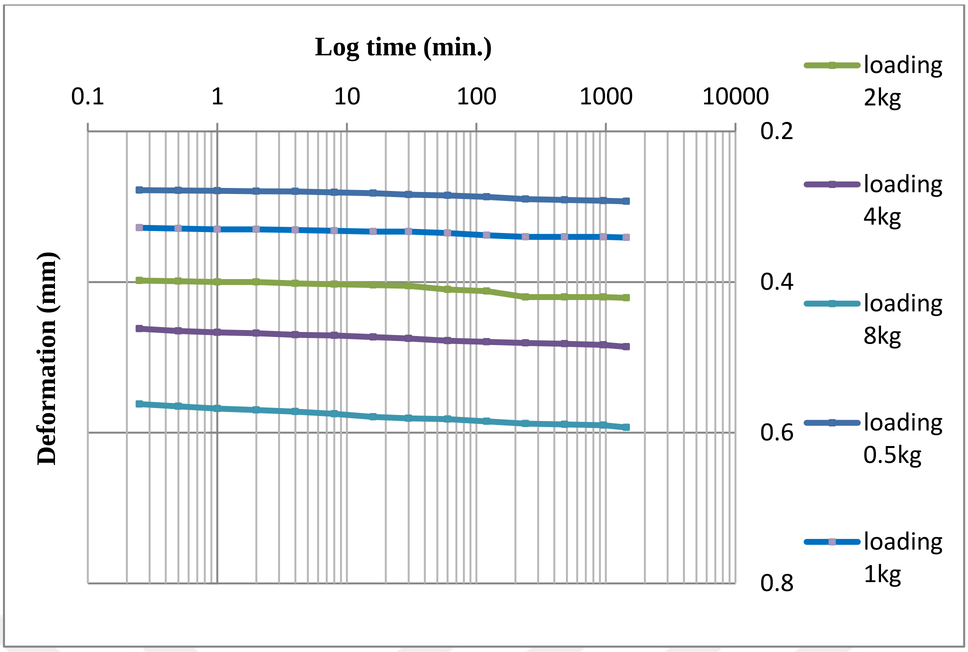


B15: σ versus m_v curve of sand colum 3.5 cm

B16: Sample of sand colum 3.5 cm

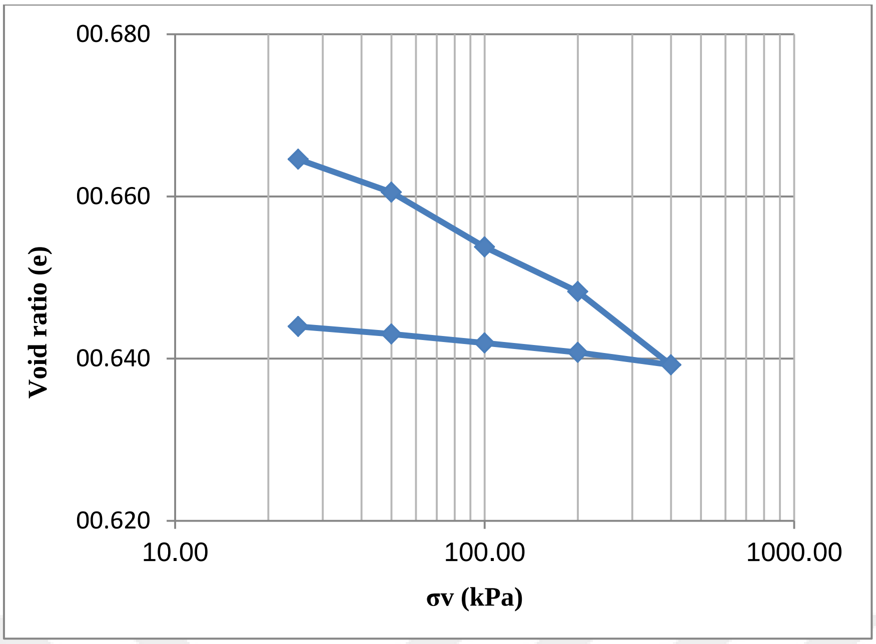
Sample description: Sand soil

Time(min.)	Deformation								
	Loading					Unloading			
	0.5kg	1kg	2kg	4kg	8kg	4kg	2kg	1kg	0.5kg
0	0	0.293	0.341	0.421	0.486	0.593	0.575	0.561	0.548
0.25	0.278	0.328	0.398	0.462	0.562	0.576	0.562	0.549	0.537
0.5	0.2785	0.329	0.399	0.465	0.565	0.576	0.562	0.549	0.537
1	0.279	0.33	0.4	0.467	0.568	0.576	0.562	0.549	0.537
2	0.2795	0.33	0.4	0.468	0.57	0.576	0.562	0.549	0.537
4	0.2798	0.331	0.402	0.47	0.572	0.576	0.562	0.549	0.537
8	0.281	0.332	0.403	0.471	0.575	0.576	0.562	0.549	0.537
16	0.282	0.333	0.404	0.473	0.579	0.576	0.562	0.549	0.537
30	0.284	0.333	0.405	0.475	0.581	0.576	0.562	0.549	0.537
60	0.285	0.335	0.41	0.478	0.582	0.576	0.562	0.549	0.537
120	0.287	0.338	0.412	0.4795	0.585	0.576	0.561	0.549	0.537
240	0.29	0.34	0.42	0.481	0.588	0.575	0.561	0.548	0.537
480	0.291	0.34	0.42	0.482	0.589	0.575	0.561	0.548	0.537
960	0.292	0.34	0.42	0.4835	0.59	0.575	0.561	0.548	0.537
1440	0.293	0.341	0.421	0.486	0.593	0.575	0.561	0.548	0.537

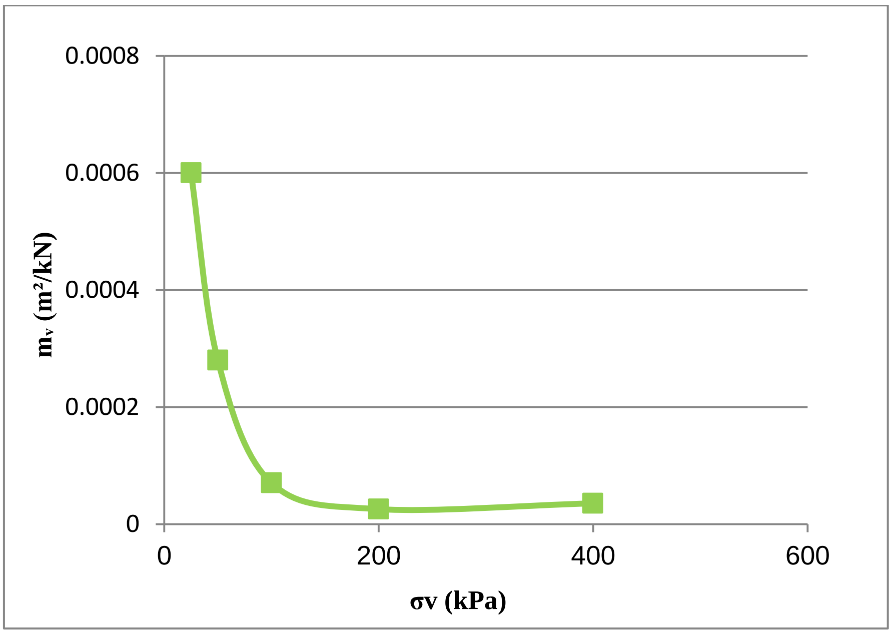


B17: Log time versus deformation curve of sand soil

Gmix	Load (kg)	σv (kPa)	lnσv	Final dial reading (mm) (Settlement)	Hs (mm)	Change in specimen height (mm)	Final specimen height (mm)	Height of Voids (mm)	Final void ratio (e)	Cc	Cr	Mv (m^2/KN)	Cc/(1+e0)	Cr/(1+e0)
2.69	0	0.0		0	11.839		20	8.161	0.6893			0.0000		0.0081
2.69	5	25.0	3.2	0.293		0.293	19.707	7.868	0.6646			0.0005864		
2.69	10	50.0	3.9	0.34		0.048	19.659	7.820	0.6605			0.0000975		
2.69	20	99.9	4.6	0.421		0.08	19.579	7.740	0.6538			0.000081		
2.69	40	199.9	5.3	0.486		0.065	19.514	7.675	0.6483	0.1271		0.000033	0.0769	
2.69	80	399.8	6.0	0.59		0.107	19.407	7.568	0.6392			0.000027		
2.69	40	199.9	5.3	0.575		-0.018	19.425	7.586	0.6408					
2.69	20	99.9	4.6	0.561		-0.014	19.439	7.600	0.6419		0.0135			
2.69	10	50.0	3.9	0.548		-0.013	19.452	7.613	0.6430					
2.69	5	25.0	3.2	0.54		-0.011	19.463	7.624	0.6440					



B18: e versus log σ curve of sand soil



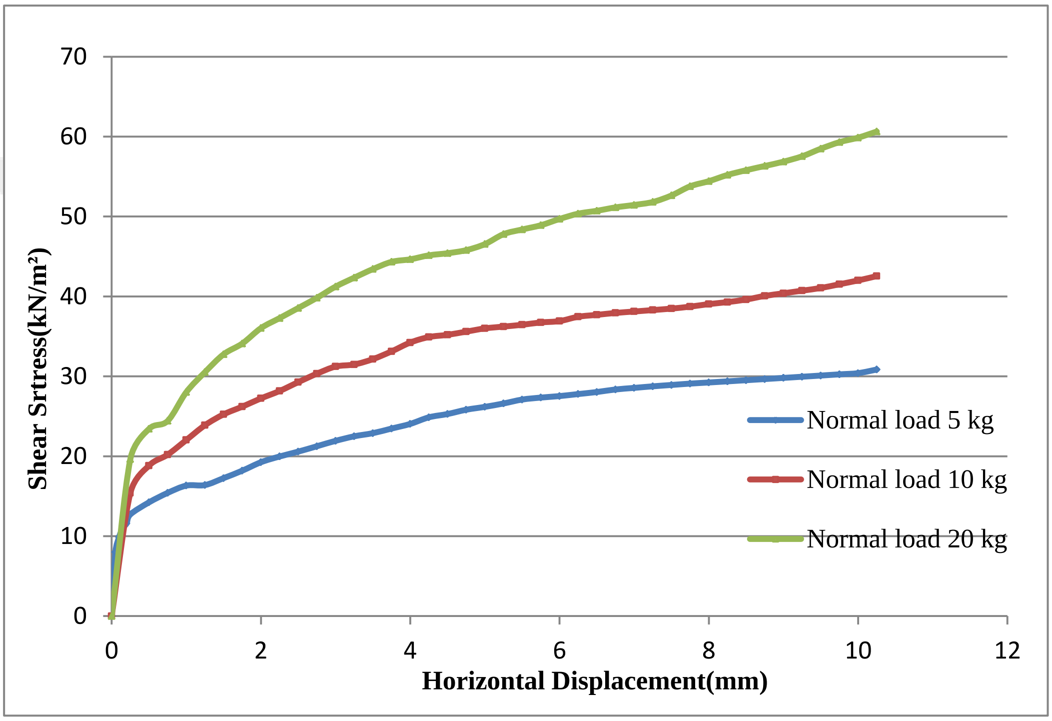
B19: σ versus m_v curve of sand soil

B20: Samples for water content after testing

APPENDIX C

DIRECT SHEAR TEST RESULTS

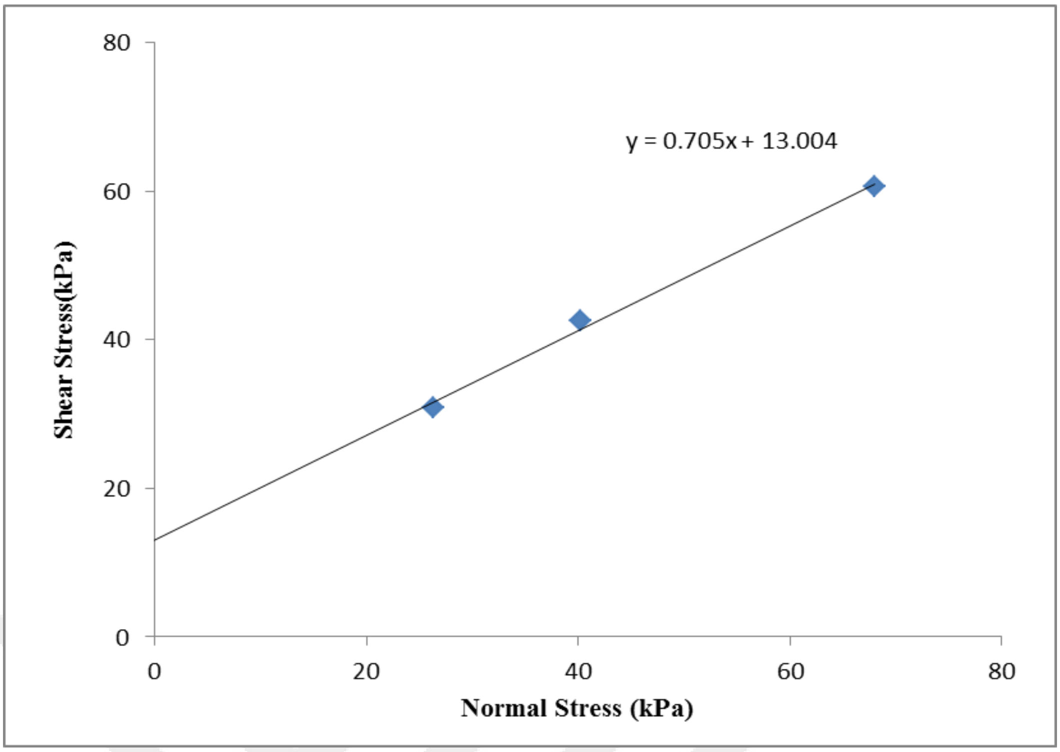
Sample description: Organic soil (UU)



C1: Horizontal displacement versus shear stress for organic soil (UU)

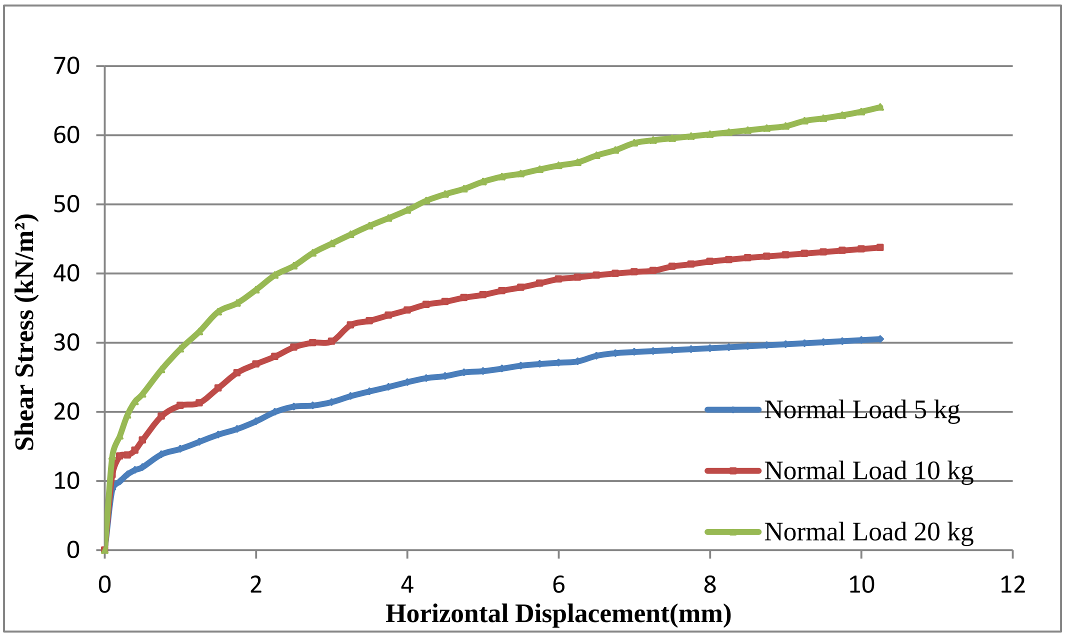
Normal load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	30.9
10 kg	40.27778	42.5
20 kg	68.05556	60.6

C(kPa) = 13.004
φ(degree)= tan invert 0.705 =35.18



C2: Normal stress versus shear stress for organic soil (UU)

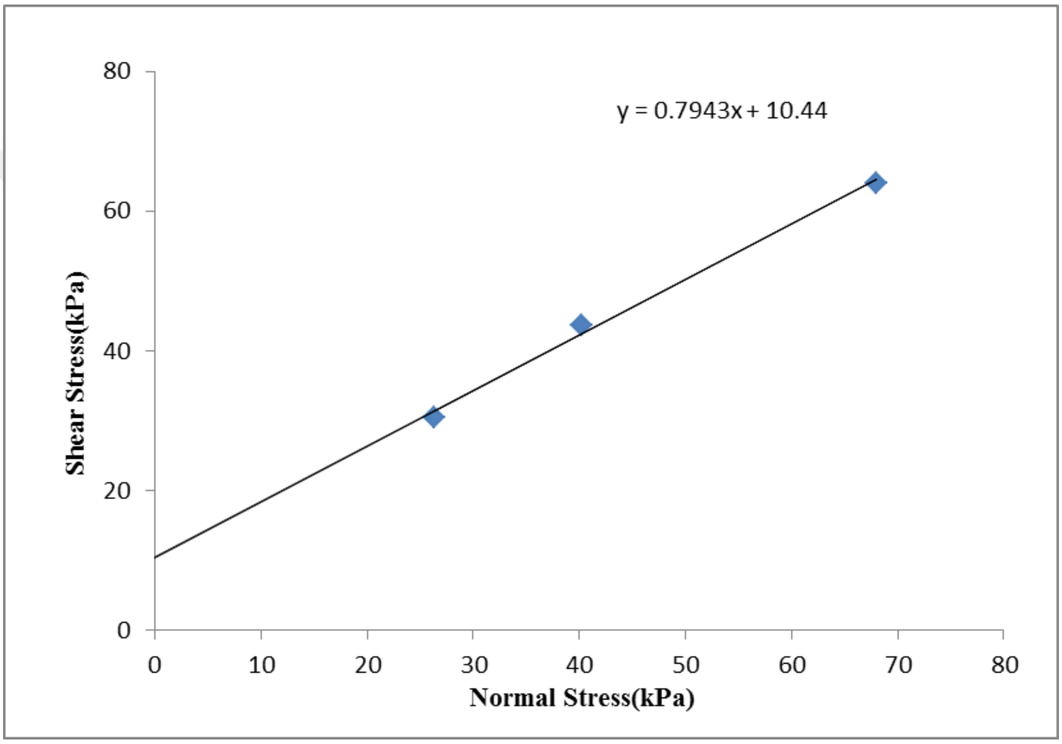
Sample description: Sand colum 2.5 (UU)



C3: Horizontal displacement versus shear stress for sand colum 2.5 (UU)

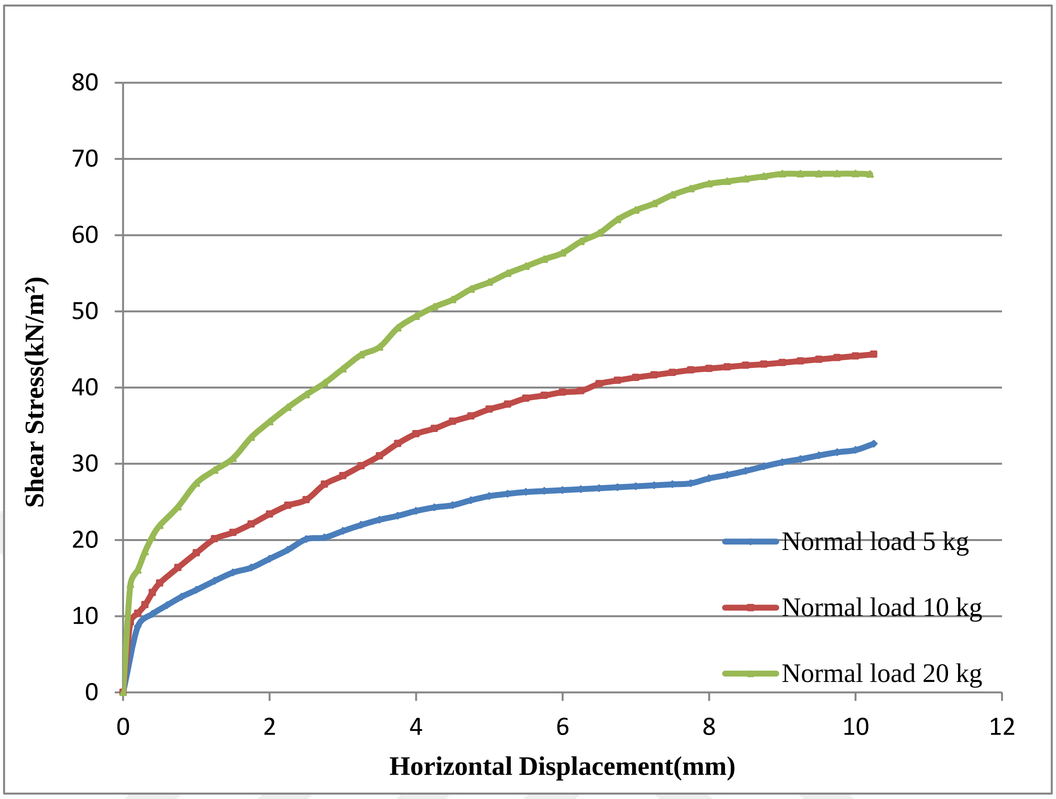
Normal load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	30.5
10 kg	40.27778	43.8
20 kg	68.05556	64.1

C(kPa) = 10.44
φ(degree)= tan invers 0.7943=38.46



C4: Normal stress versus shear stress for sand colum 2.5 (UU)

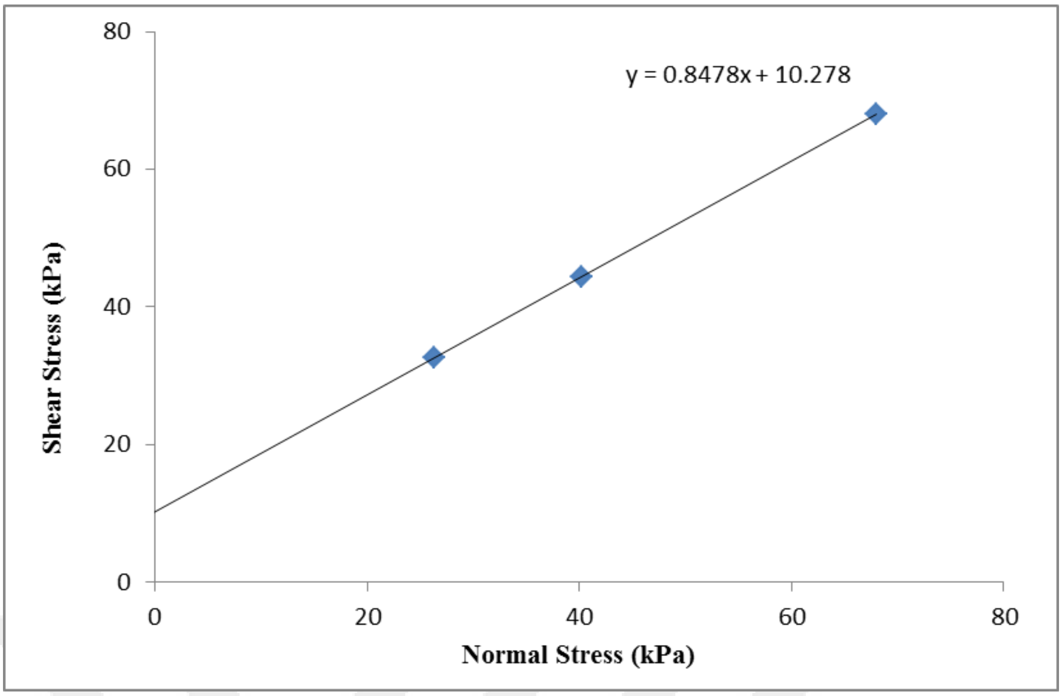
Sample description: Sand colum 3.5 (UU)



C5: Horizontal displacement versus shear stress for sand colum 3.5 (UU)

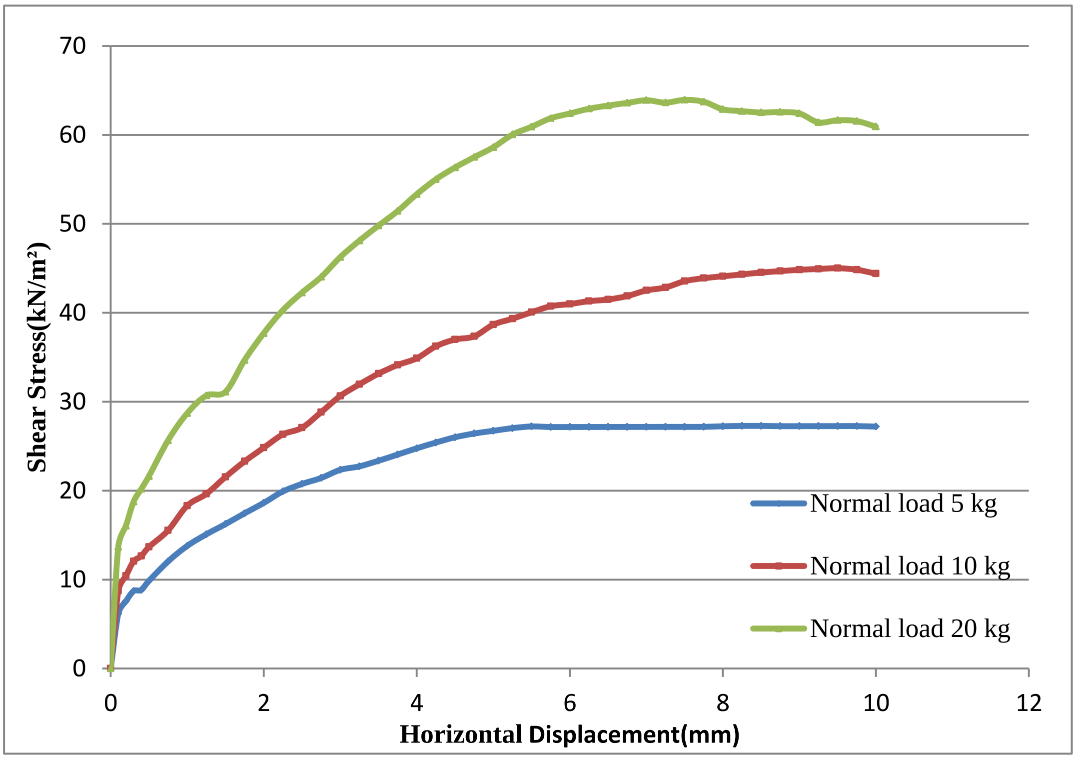
Normal Load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	32.7
10 kg	40.27778	44.4
20 kg	68.05556	68.0

C (kPa) = 10.278
φ (degree)= tan invers 0.8478 = 40.2



C6: Normal stress versus shear stress for sand colum 3.5 (UU)

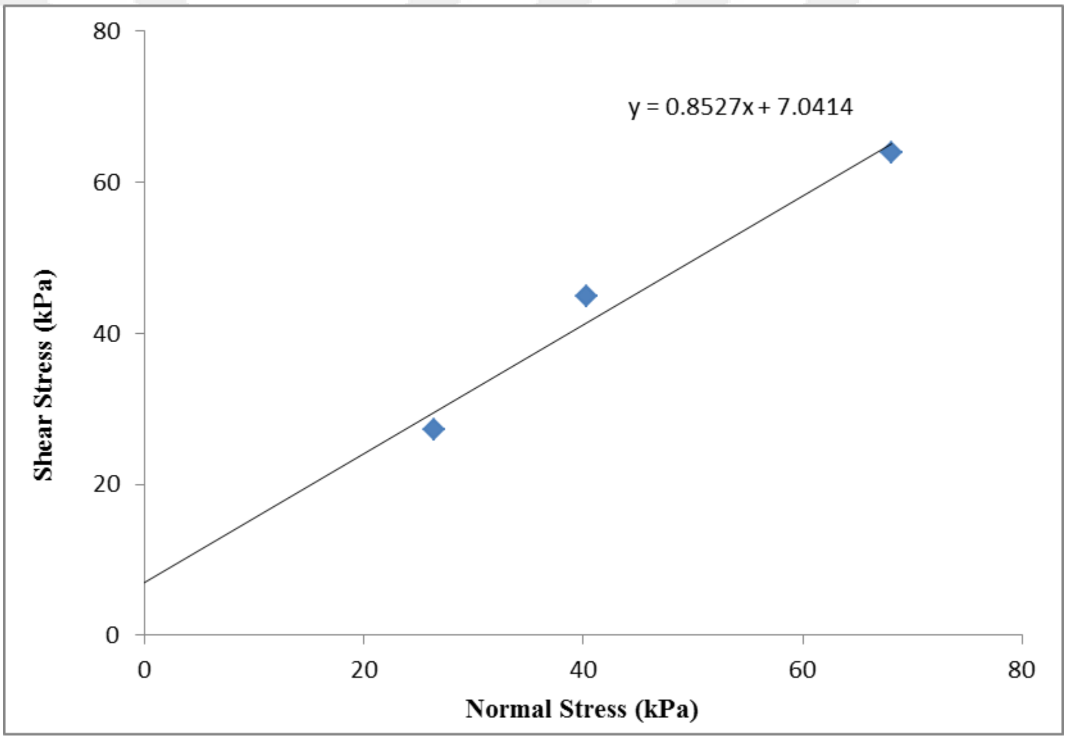
Sample description: Sand colum 4.7 (UU)



C7: Horizontal displacement versus shear stress for sand colum 4.7 (UU)

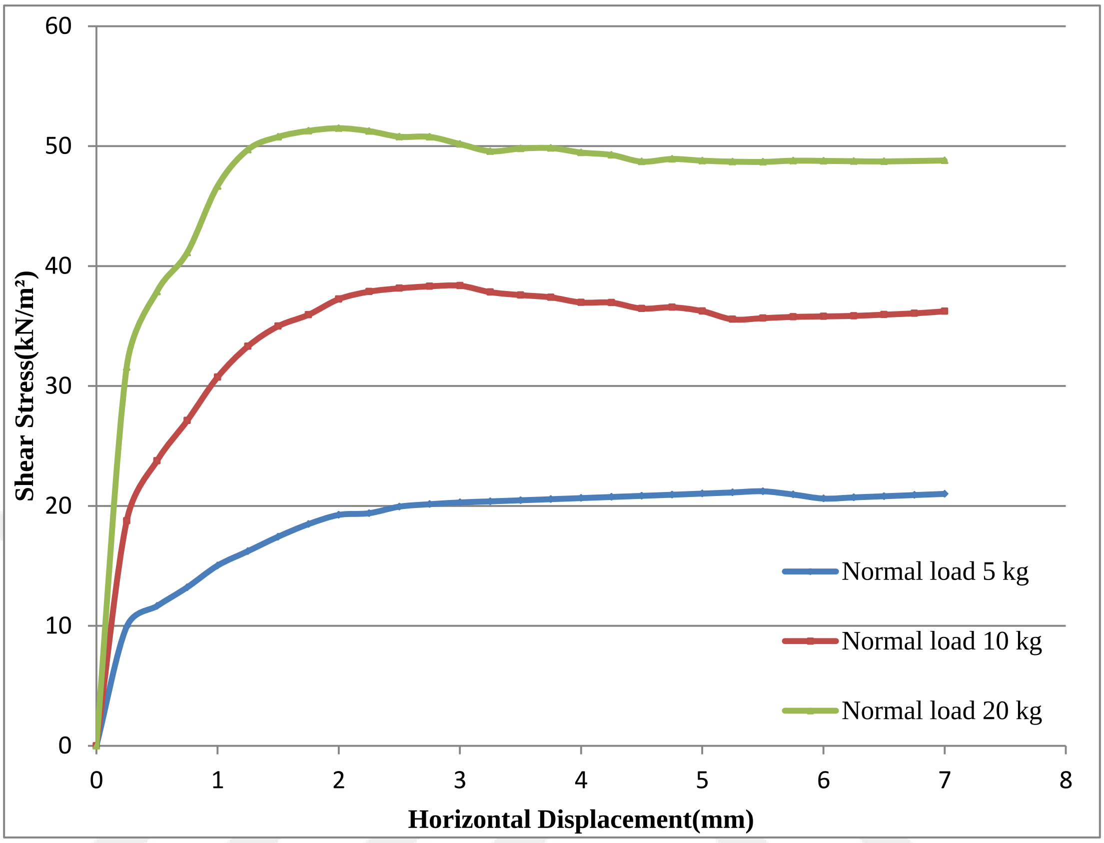
Normal Load	Normal Stress (kpa)	Shear Stress (kpa)
5kg	26.38889	27.2
10kg	40.27778	44.9
20kg	68.05556	63.9

C (kPa) = 7.0414
φ (degree)= tan invers 0.8527 = 40.5



C8: Normal stress versus shear stress for sand colum 4.7 (UU)

Sample description: Loose sand (UU)

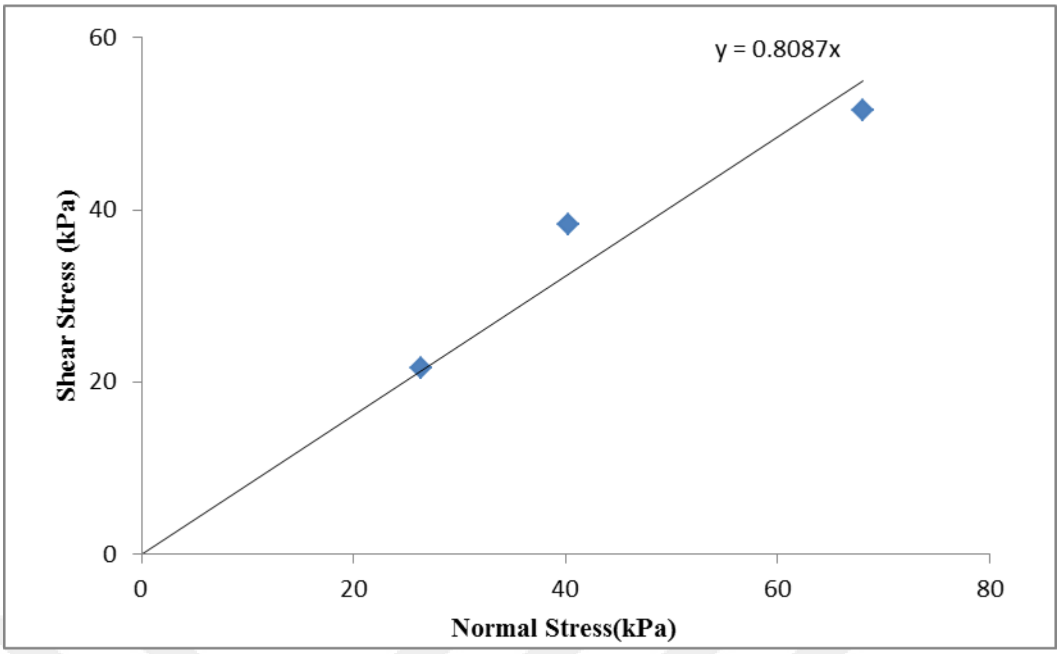


C9: Horizontal displacement versus shear stress for loose sand (UU)

Normal Load	**Normal Stress (kpa)**	**Shear Stress (kpa)**
5 kg	26.38889	21.6
10 kg	40.27778	38.4
20 kg	68.05556	51.5

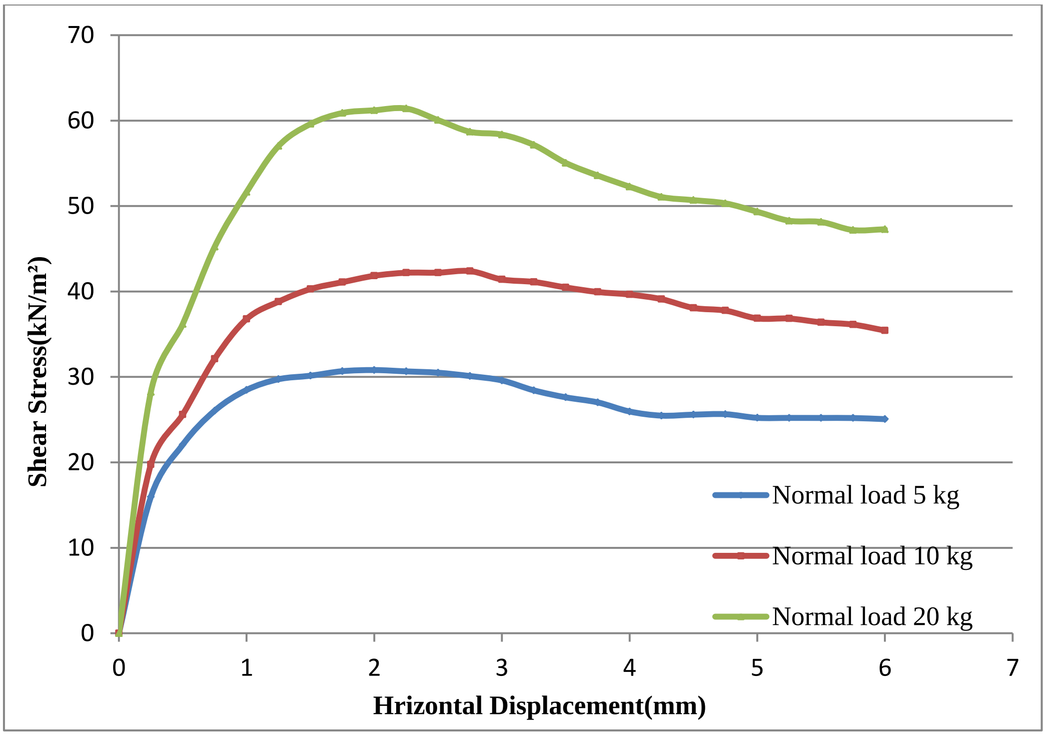
C (kpa) = o

φ (degree)=tan invers 0.8087 = 38.96



C10: Normal stress versus shear stress for loose sand (UU)

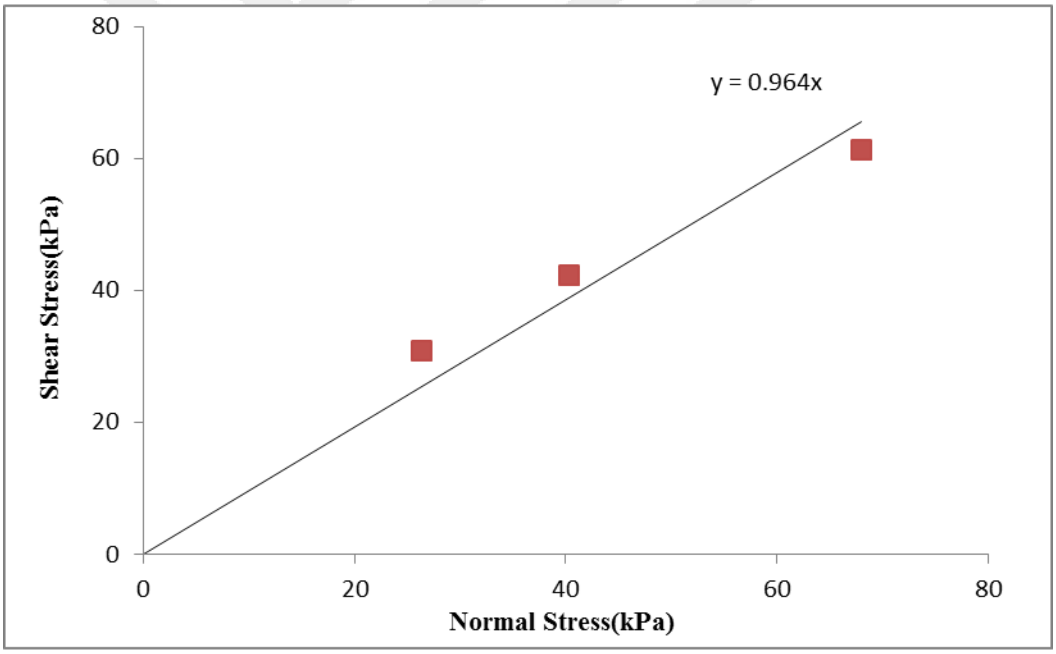
Sample description: Compacted sand (UU)



C11: Horizontal displacement versus shear stress for compacted sand (UU)

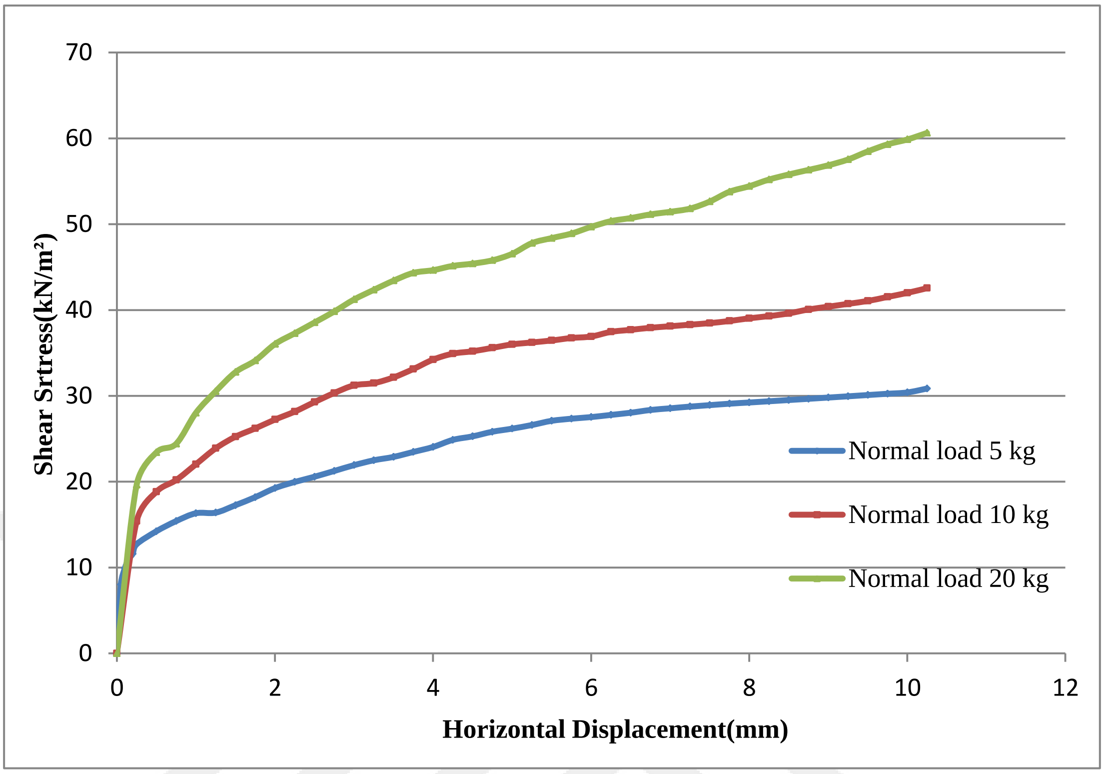
Normal load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	30.8
10 kg	40.27778	42.4
20 kg	68.05556	61.4

C (kPa) = 0
φ (degree)= tan invers 0.964= 43.949



C12: Normal stress versus shear stress for compacted sand (UU)

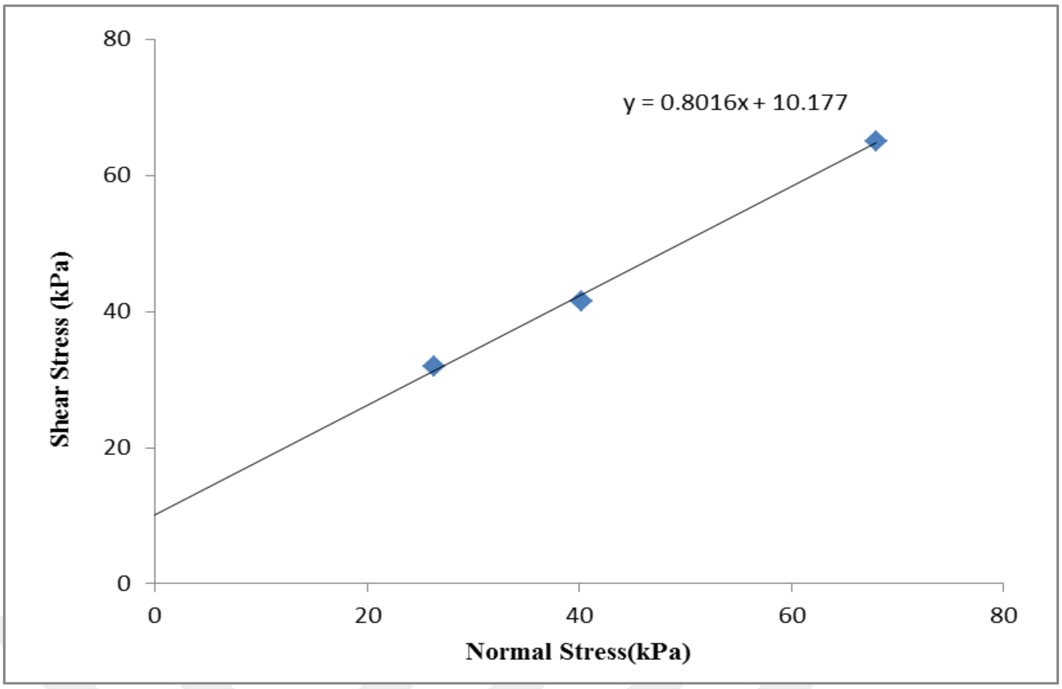
Sample description: Organic soil (CU)



C13: Horizontal displacement versus shear stress for organic soil (CU)

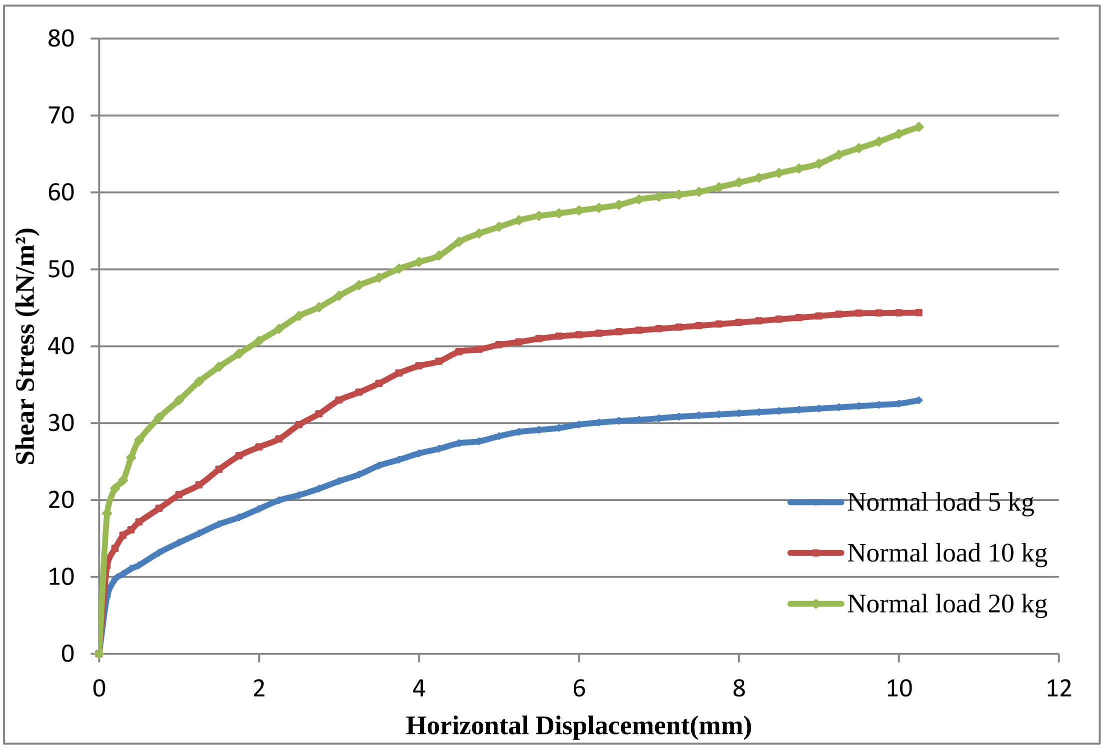
Normal load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	31.9
10kg	40.27778	41.5
20kg	68.05556	65.0

C (kPa) = 10.177
φ (degree)= tan invers 0.8016 = 38.715



C14: Normal stress versus shear stress for organic soil (CU)

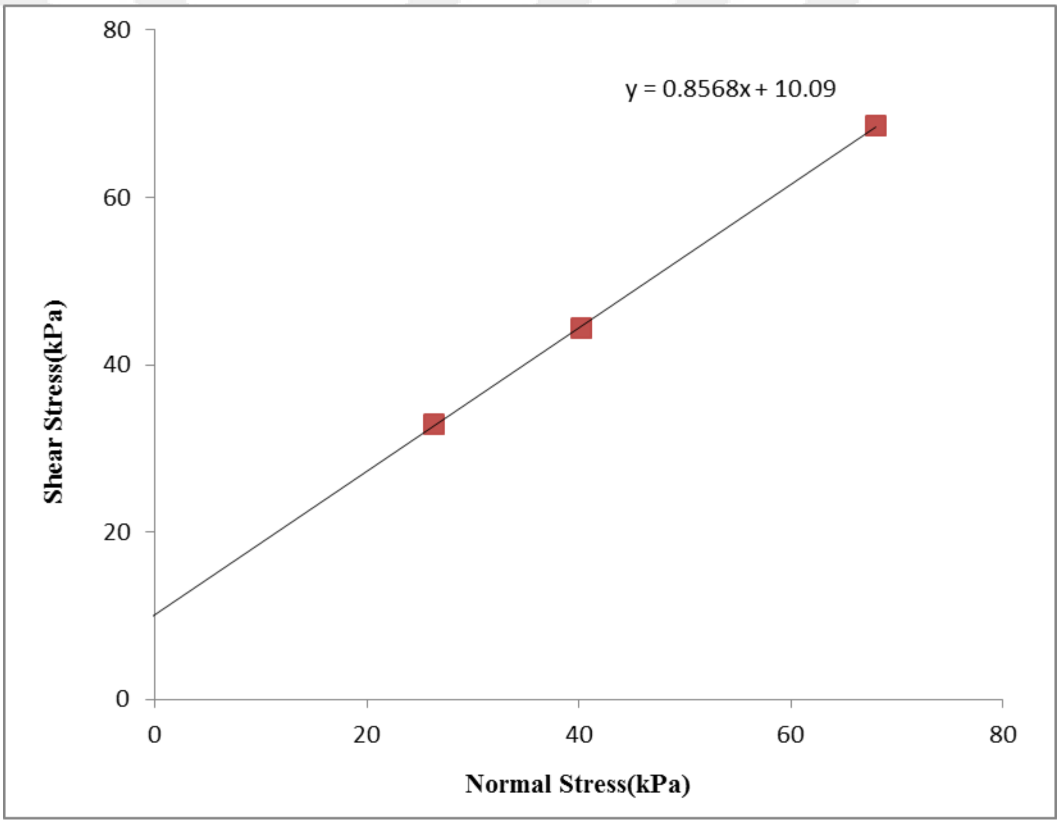
Sample description: Sand colum 2.5 (CU)



C15: Horizontal displacement versus shear stress for sand colum 2.5 (CU)

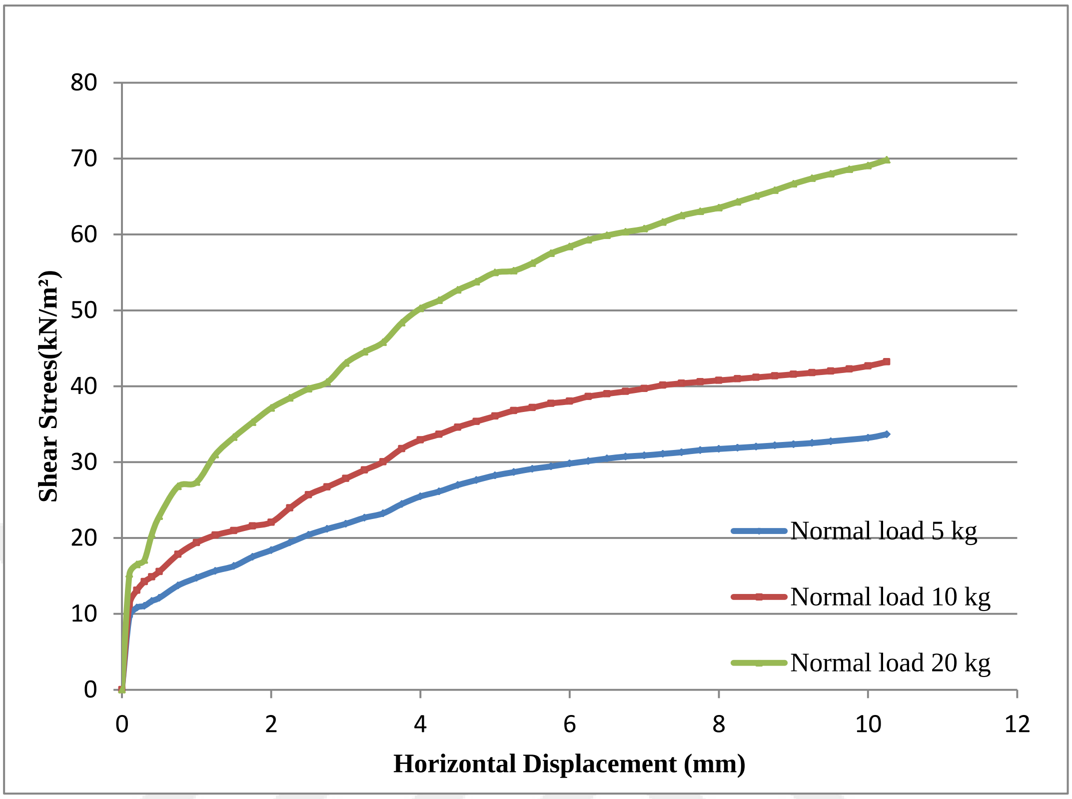
Normal Load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	32.9
10 kg	40.27778	44.3
20 kg	68.05556	68.5

C (kPa) = 10.09
φ (degree)= tan invers 0.8568 = 40.5



C16: Normal stress versus shear stress for sand colum 2.5 (CU)

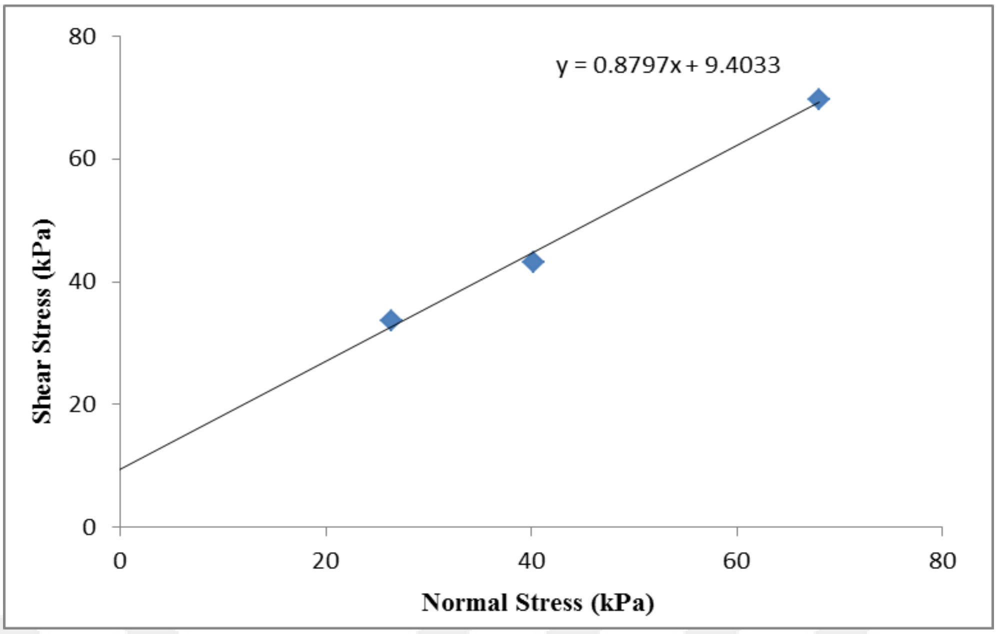
Sample description: Sand colum 3.5 (CU)



C17: Horizontal displacement versus shear stress for sand colum 3.5 (CU)

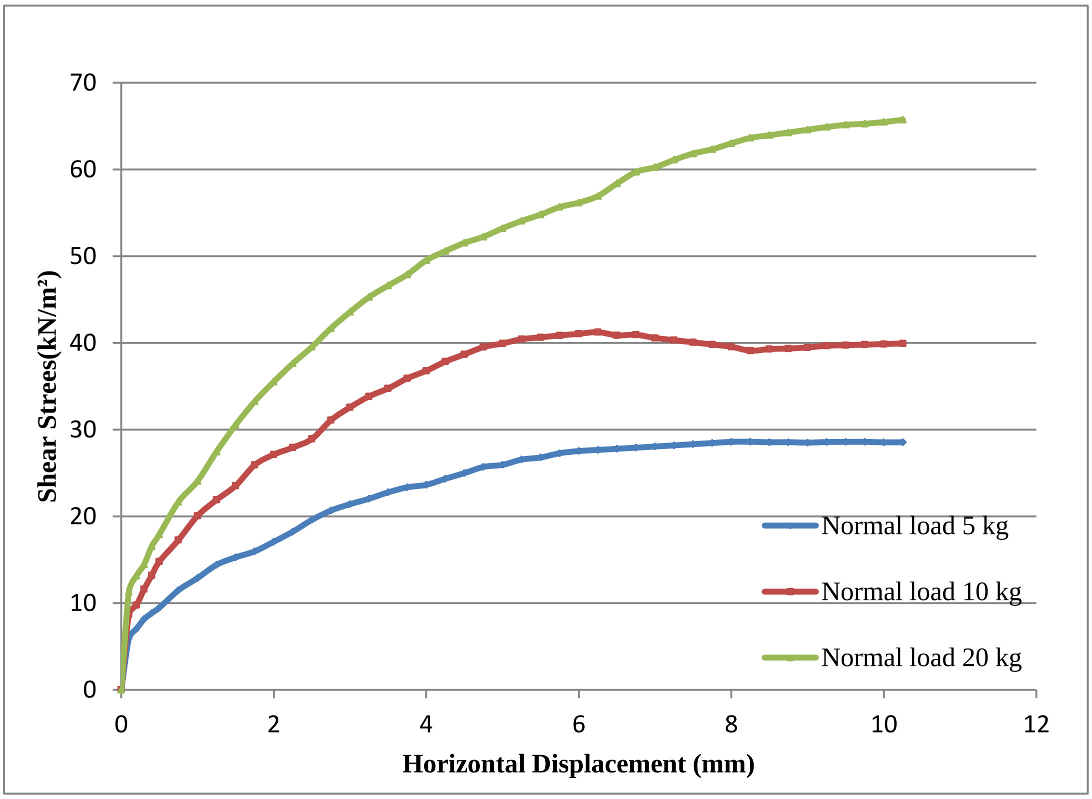
Normal Load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	33.7
10 kg	40.27778	43.3
20 kg	68.05556	69.8

C (kPa) = 9.4033
φ (degree)=tan invers 0.8797 = 41.3



C18: Normal stress versus shear stress for sand colum 3.5(CU)

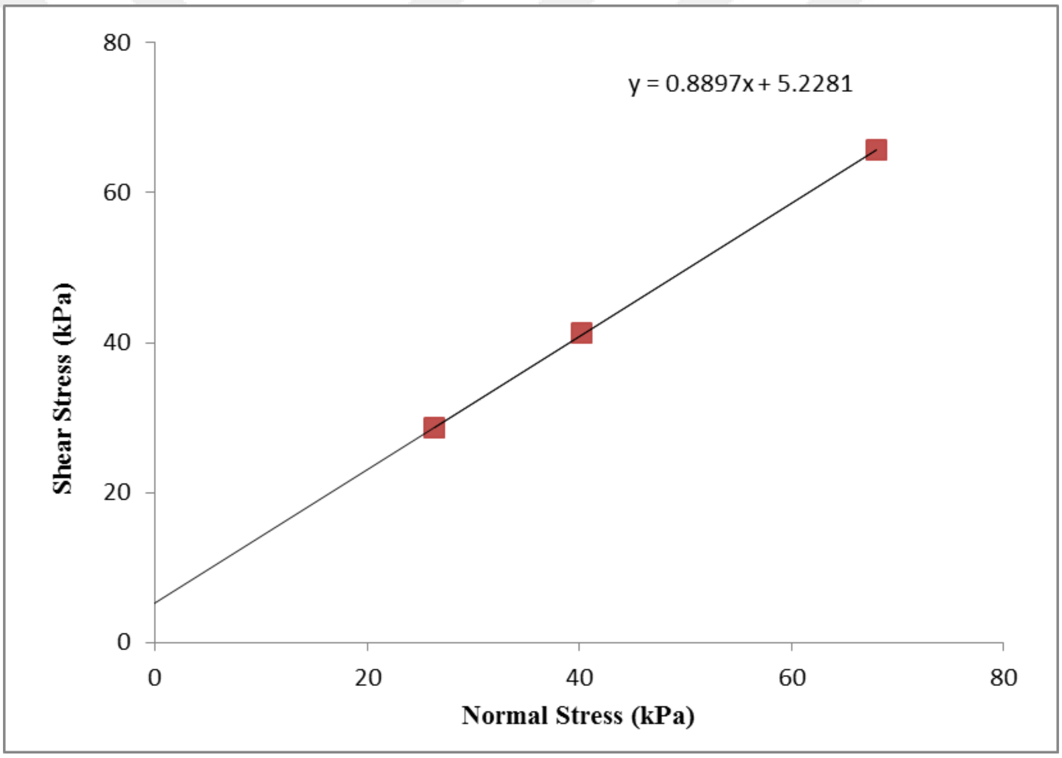
Sample description: Sand colum 4.7 (CU)



C19: Horizontal displacement versus shear stress for sand colum 4.7 (CU)

Normal Load	Normal Stress (kPa)	Shear Stress (kPa)
5 kg	26.38889	28.6
10 kg	40.27778	41.2
20 kg	68.05556	65.7

C(kPa) = 5.2281
φ (degree)=tan invers 0.8897= 41.659



C20: Normal stress versus shear stress for sand colum 4.7 (CU)

C21: Preparing samples for direct shear test

C22: Samples after direct shear test for different sand column