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SHEAR STRENGTH PROPERTIES OF CLAY WITH SAND COLUMN

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IN
CIVIL ENGINEERING

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Shear Strength Properties of Clay with Sand Column

M.Sc. Thesis
in
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University of Gaziantep

Supervisor
Assoc. Prof. Dr. Hanifi ÇANAKÇI

by

Hakan İKİZ

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ABSTRACT

SHEAR STRENGTH PROPERTIES OF CLAY WITH SAND COLUMN

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M. Sc. in Civil Engineering
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Direct shear tests were performed on low plastic residual clay reinforced with different diameter and number of sand columns. The tests were carried out under unconsolidated undrained condition. Sand used in the tests is poorly graded passing No: 5 sieve and retaining on No: 40. Effects of sand column diameter and number of sand columns in sand-clay mixtures on shear strength parameters internal friction angle and cohesion were investigated. The test results showed that shear strength parameters of sand-clay mixtures were affected by diameter of sand column and sand content. Internal friction angle of sand-clay mixtures for the same sand column diameter increases with increasing number of sand columns but cohesion reduces. The test results also showed that internal friction angle with same sand content area of sand-clay mixtures increases with increasing sand column diameter. However, under the same condition, cohesion of the mixtures decreases. In addition, volume of the sand-clay mixtures was affected by number and diameter of sand column during shear of the mixture.

Key Words: Sand columns, sand-clay mixture, shear strength, internal friction angle, cohesion.

ÖZET

İÇERİSİNE KUM KOLON YERLEŞTİRİLMİŞ KİLİN KAYMA DAYANIMI ÖZELLİKLERİ

İKİZ, Hakan
Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Doç. Dr. Hanifi ÇANAKÇI
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Bu çalışmada, düşük plastisiteli kil içine farklı çaplarda ve sayılarda kum kolonları yerleştirilerek drenajsız direkt kesme deneyleri yapılmıştır. Kum-kil karışımında 5 numaralı eleğin altında kalan ve 40 numaralı eleğin üstünde kalan kum kullanılmıştır. Bu deneyde, kil zemin içine dikey olarak yerleştirilen kum kolonlarının çapının ve sayısının kum-kil karışımının kayma dayanımı parametreleri olan içsel sürtünme açısı ve kohezyon üzerindeki etkisi araştırılmıştır. Deney sonuçlarına göre, kil içerisine yerleştirilen kum kolonlarının karışımın kayma dayanımı parametrelerini etkilediği gözlemlenmiştir. Kil içerisine yerleştirilen kum kolonlarının çapları değiştirilmeden sayılarının artırılması ile içsel sürtünme açısının arttığı kohezyonun ise azaldığı görülmüştür. Kil içerisine yerleştirilen kum miktarı aynı tulup kolon çapları artırıldığında ise içsel sürtünme açısının arttığı ve kohezyonun azaldığı gözlemlenmiştir. Ayrıca, kum kil karışımının kayma esnasında hacim değişimlerinin kum kolonlarının sayısı ve çapları ile etkilendiği gözlendi.

Anahtar Kelimeler: Kum kolonlar, kum-kil karışımı, iç sürtünme açısı, kohezyon.

ACKNOWLEDGEMENTS

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LIST OF SYMBOLS

A	Cross sectional area of specimen in shear box test
CD	Consolidated drained test
c_u	Undrained shear strength
C_f	Function of clay content
c'	Cohesion intercept in terms of effective stresses
c	Cohesion intercept in terms of total stresses
D_r	Relative density
d	Diameter
e	Overall void ratio of soil
e_0	Initial void ratio
e_{cv}	Critical void ratio
e_{max}	Maximum void ratio
e_{sk}	Skeleton void ratio
f	Ratio of weight of fines to total weight of solids
h	Height
I_p	Plasticity index
LL	Liquid limit
N	Normal load
N_{60}	SPT blow count corrected for field procedures
OCR	Over consolidation ratio
P_h	Horizontal load
PI	Plasticity index

P_v	Vertical load
s_u	Undrained shear strength
T	Maximum torque
UU	Unconsolidated undrained
u	Pore water pressure
ψ	Angle of dilation
ε_v	Vertical strain
ϕ'	Friction angle in terms of effective stresses
ϕ	Friction angle in terms of total stresses
ϕ'	Friction angle in terms of effective stresses
ϕ'_{max}	Maximum angle of shearing resistance
ϕ'_{cv}	Critical angle of shearing resistance
ϕ'_u	True angle of friction
λ	Unit weight
σ_f	Normal stress on failure plane
σ'	Effective normal stress
σ'_3	Effective minor principal stress in triaxial test
τ	Shear stress in shear box test and drained shear strength
τ_f	Shear stress on the failure plane
τ_u	Undrained shear strength

CHAPTER 1

INTRODUCTION

1.1. General

In geotechnical engineering, there are so many problems with ground during built a construction. The main problems are bearing capacity, lateral earth pressure, settlement, and slope stability.

Soil at a construction site is not always completely useful for constructing structures for instance buildings, bridges, highways and dams. In clayey soil, there is usually large consolidation settlement depending on depth of layer and structural load. In loose granular soils, large elastic settlement is anticipated. Under these problems, soils needs ground modification such as reducing consolidation and elastic settlement and increasing shear strength of soils. If it is possible, the problematic soils must be removed and replace with better soil at site. If not possible different modification techniques need to be used such as hydraulic modification, inclusion, chemical modification, compaction, mixing etc. The main purposes of ground improvement are;

- Develop shear strength and bearing capacity of soil.
- Decrease settlement of soil under structural load.
- Decrease shrinkage and swelling of soil.
- Raise safety of slope stability.

Ground modification techniques have been used since many years. Every year, these improvements have become better than the past. General ground improvement techniques are numbered at the below.

- In-situ densification of soil.
- Grouting

In this thesis, it was aimed to improve shear strength of soil with inclusion of sand columns in low plastic residual clayey soil. Three different diameters of sand columns and four different sand to clay (s/c) area ratios were used. In all series of tests shear strength parameters internal friction angle and cohesion, stress-strain behavior, and vertical deformation were measured.

1.2. Organization of the Thesis

The present thesis is composed of seven chapters. In Chapter 2, a review of the literature survey about sand inclusion in clay is given. The theoretical background of shear strength, shear strength parameters, and shear strength tests are give Chapter 3. In Chapter 4, material and experimental methods are explained. These are material and material properties, procedures of preparing samples. The test results for in each experimental study are given chapter 5. The explanation and discussion of test results are represented in Chapter 6. Finally the conclusions obtained from the study are presented in chapter 7.

CHAPTER 2

LITERATURE SURVAY

2.1. Introduction

In the past different researchers studied the performance of clay-sand. Some of these studies are uniform mixture of sand and clay some of them are making sand column on clay layer. Summary of some of literature review is given below.

2.2. Literature Survey

Wasti and Alyanak (1968) have performed series of tests on sand-clay mixtures. They stated that when clay content is just enough to fill the voids of the sands at its maximum porosity. They found that structure of the mixture varies and there is a linear relationship between the Atterberg limits and mixtures behavior. The minimum value of kaolin content was found for the mixture at liquid limit of kaolin clay.

Novais and Ferreira (1971) performed consolidated-drained direct shear tests on artificial mixtures with increasing content of clay. The mixture has included two types of sand (fine and coarse) and a montmorillonitic clay. They found that shear stress decreasing as the clay content increased. They also described the existence of three zones of behavior of the mixtures as a function of clay content (C_f):

Incoherent behavior ($C_f \leq 28$ %) where the cohesion is null and the angle of friction is high (above 30^o) the effects of fluctuations in soil grain size variations are not significant.

- ($28 < C_f < 41$) where the soil is sensitive to grain size fluctuations.
- Coherent behavior ($C_f \geq 41$) where the cohesion is high and the angle of friction is low.

Georgiannou (1988) made a study on the behavior of clayey sands under monotonic and cyclic loading. He found that the fines content has an exceptional influence on the stress-strain response of the soil mass. As the fines content increases, the dilatants behavior of the soils is repressed, and the reaction gradually becomes controlled by the fine matrix at about % 40 fines content.

Georgiannou, et al., (1990) performed an experimental study about stress-strain behavior of anisotropically consolidated clayey sands using computer controlled triaxial cells. The specimens were arranged by sedimenting Ham River sand into a kaolin suspension. They studied the results of variations in clay content and initial granular void ratio. They invented that this method produces a material which is noticeably less stable, which has a higher granular void ratio and shows a higher undrained brittleness behavior. Furthermore they showed that a sand that has 30 % clay fraction the normally consolidated material is no more dilatant. They also expressed that for clay fractions up to 20 %, the clay does not meaningfully decrease the angle of shearing resistance of the granular component.

Pitman, et al., (1994) have performed an analysis to investigate the affect of fines and gradation on the behavior of loosely prepared sand samples. Loose sand samples, arranged by moist and consolidated to the same effective stress level, were prepared with different percentages of both plastic and nonplastic fines. Samples were isotropically consolidated and subjected to monotonic undrained triaxial compression. They stated that undrained brittleness decreased as the fines content increased. They also found that the undrained brittleness may not be controlled by the plasticity of the fines but more by the amount of fines (<74μm), at least for percentages greater than 10 %.

Tan, et al., (1994) studied an experimental investigation about shear strength of very soft clay-sand mixtures. Sand was added in clay slurry to improve shear strength and accelerate the consolidation. The fall cone penetration test was conducted to determine the shear strength of the mixture. The test showed that the clay slurry filled the voids in the sand increased shear strength of the mixture.

Bayoğlu (1995) carried out an experimental study. Shear strength and compressibility of sand and clay mixture was studied in this investigation. In the mixture were used fine particles (diameter < 0.074 mm). Soil mixtures having wide range of grain size from sand to silt-clay mixtures were studied. Drained shear box and consolidated-undrained triaxial tests were performed on normally consolidated clay-sand mixtures to find strength and compressibility parameters. The mixtures were containing 5 %, 15 %, 35 %, 50 %, 75 %, and 100 % fines. The results of direct shear tests are given in Figure 2.2 and Figure 2.2. The internal friction angles varied between 30-38 degrees. It increased a little until 10 % fines. However, the friction angle of more than 10 % fines content decreased from 38 to 10 degrees with increasing fine content. At fine contents higher than 50%, the reduction in the friction angle was significant and faster.

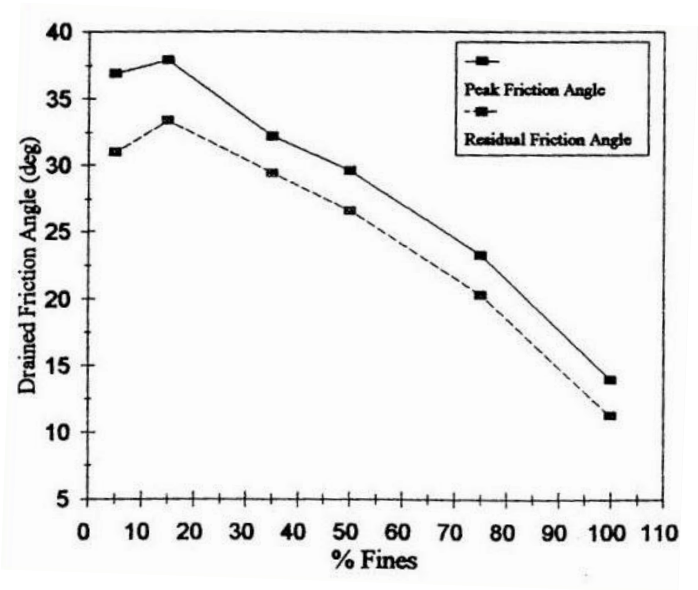

Figure 2.1. Drained friction angle - % fines relation (Bayoğlu, 1995)

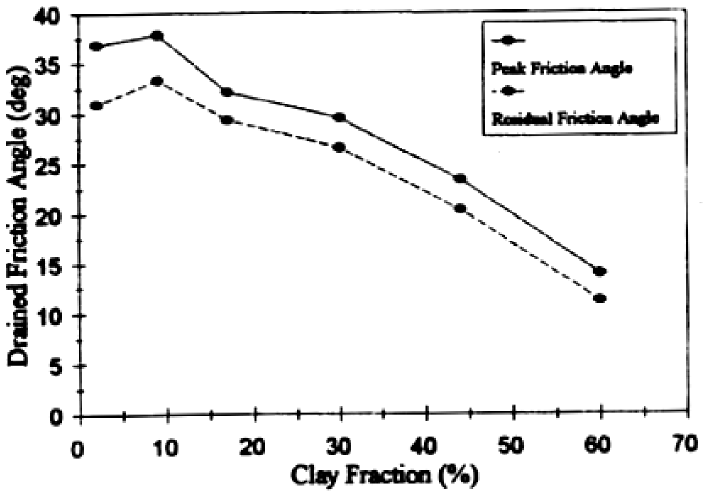

Figure 2.2. Drained friction angle - % clay fraction relation (Bayoğlu, 1995)

The relation between drained friction angle and plasticity index and liquid limit is given in Figure 2.3 and Figure 2.4 The behavior, which is about the same for the two properties, is more likely a linear decrease of the angles with increasing PI and LL.

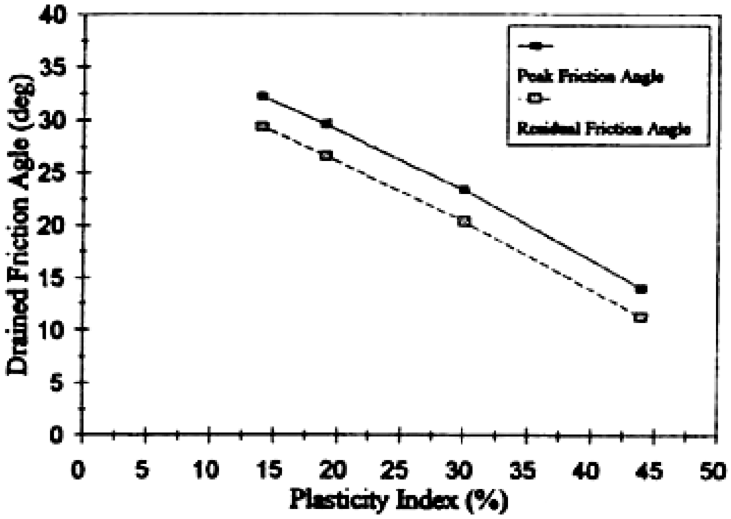

Figure 2.3. Drained friction angle plotted against plasticity index (Bayoğlu, 1995)

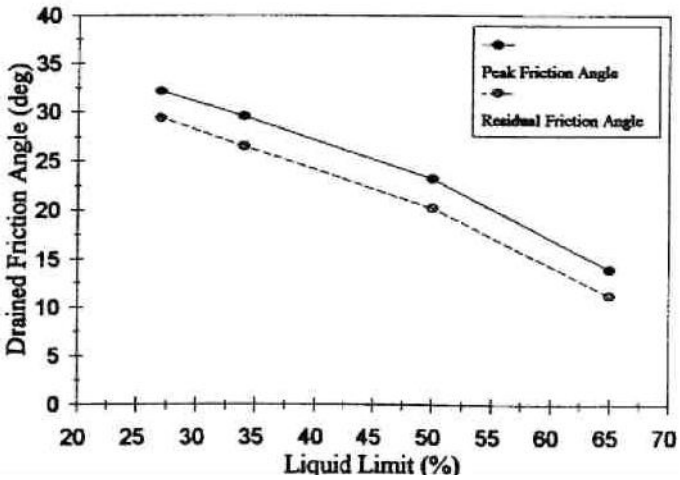

Figure 2.4. Drained friction angle plotted against liquid limit (Bayoğlu, 1995)

Consolidated-undrained triaxial tests were performed on 35%, 50%, 75%, and 100% fines content. Its results are presented in Figure 2.5; there was no a direct relation between undrained friction angle and percentage of fines and the measured angle of shearing resistances were in the same order of magnitude regardless of percent fines.

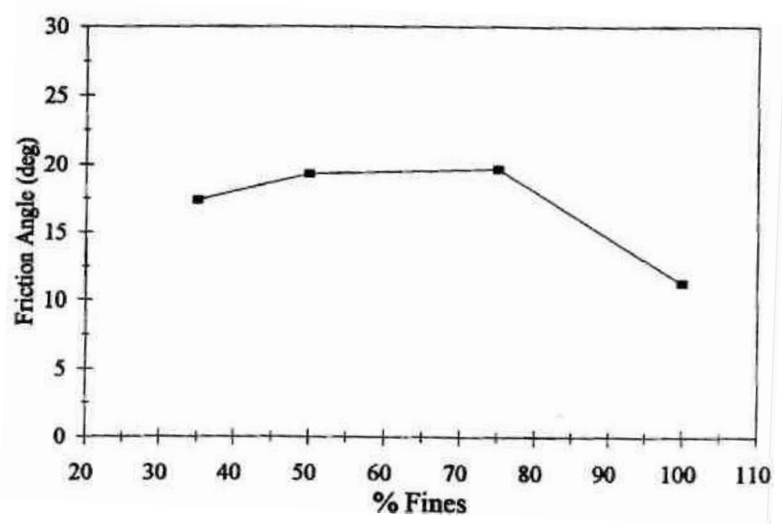

Figure 2.5. Undrained friction angle plotted against % fines (Bayoğlu, 1995)

Salgado (2000) carried out an experimental study on the effects of nonplastic fines on the shear strength of sands. A series of laboratory tests was made on samples of Ottawa sand. The fine content of sand was ranged of 5-20 % by weight. Triaxial tests were performed to axial strains in excess of 30 %. The concept of the skeleton void ratio e_{sk} (Kuerbis et al., 1988) is the void ratio of the silty sand calculated as if the fines were voids.

$$e_{sk} = \frac{1 + e}{1 - f} - 1 \tag{2.1}$$

where, e = overall void ratio of soil,
f = ratio of weight of fines to total weight of solids.

Whenever e_{sk} is greater than the maximum void ratio $(e_{max})_{f=0}$ of clean sand, the sand particles are not in contact and mechanical behavior is no longer controlled by the sand matrix. They suggested that silty sand with nonfloating fabric in the 5-20 % silt content of dilatancy is more than clean sands; dilatancy appears to peak around 5 % silt content, but even at 20 % silt content it remains above that of clean sand.

Wallejo and Mawby (2000) carried out a series of direct shear tests and porosity measurements on sand-clay mixtures. It was observed that the percentage of sand in the mixture had a marked effect on their shear strength. When the amount of the sand by weight in the mixture was more than 75%, the shear strength of the mixture was affected mainly by the frictional resistance between sand grains. When the amount of sand had between 75 and 40%, the shear strength of the mixture was provided by shear strength of clay and resistance between sand grains. When the sand was less than 75% by weight, the shear strength of mixture was affected mainly by shear strength of clay.

Wood and Kumar (2000) performed drained and undrained triaxial compression tests on normal and over consolidated mixtures of kaolin clay and coarse uniform sand. It was observed from tests that deviator stress, clay volumetric strain and pore pressure were not affected by amount sand until the clay content fell below 40%.

Prakasha and Chandrasekaran (2005) studied on reconstituted Indian marine soils having different proportions of sand and clay. Test results showed that sand grains in clay provide to decrease in void ratio and increase in friction and pore pressure response resulting in decrease in undrained shear strength.

Canakci and Gullu (2007) carried out an experimental study on sand-clay mixture in different ratio and found that increase in amount of sand in the mixture reduces the internal friction angle up to 15% of sand content. When the sand content is higher than 15%, internal friction angle increases and approaches sand friction angle.

Black, et al., (2007) reported triaxil test on clay reinforced with vertical sand columns having two different diameters. In their study, they used transparent medium instead of clay in order to observe deformations of column under real time loading. Authors used two sets of test. In the first set single 32mm diameter sand column was inserted in the claylike material, and in the second set of test three 20mm diameter sand column were used. Test were carried both in drained and undrained conditions. Their test results showed that undrained behavior of the composite was improved considerably. They also found that the undrained strength of the composite material was not particularly affected by the number of columns.

Ölmez (2008) carried out an experimental study about shear strength behavior of sand-clay mixtures. The mixtures were fine clean sand and kaolin clay. The ratio of sand in the mixture was varied 5 to 40%. Aim of the study to observe the affects of fine materials in the soil mixture on the behavior of shear strength. Undrained triaxial compression tests, and drained direct shear tests were performed on sand-clay mixtures.

Three series experiment were studied. In the 1.series, triaxial unconsolidated – undrained tests (UU) were performed under 35 kPa, 60 kPa, and 85 kPa cell pressures on 5%, 10%, 15%, %20, and 25% of soil mixture dry weight. The result of first series of experiment in, Figure 2.6 there was not important difference in undrained strength until 20% kaolin content of the mixture. But after 20% kaolin, strength decreased significantly with the increase in kaolin content.

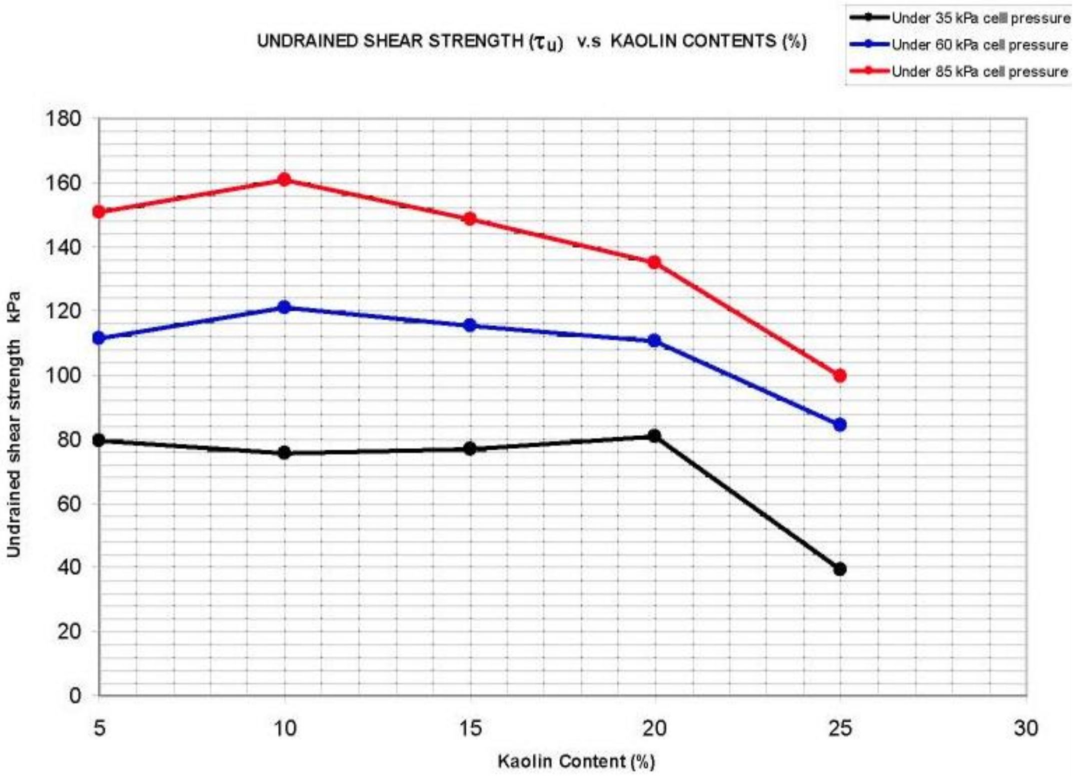

Figure 2.6. Relations between undrained shear strength and kaolin contents, in series1 (Ölmez, 2008)

In the second series, the behavior of the specimens under unconsolidated -undrained conditions in triaxial compression test was studied under both 50 kPa and 100 kPa cell pressures. The result of study is shown in Figure 2.7, similarly like first series, up to 20% kaolin content strength was nearly constant. After 20 % kaolin, strength decreased with increasing kaolin content.

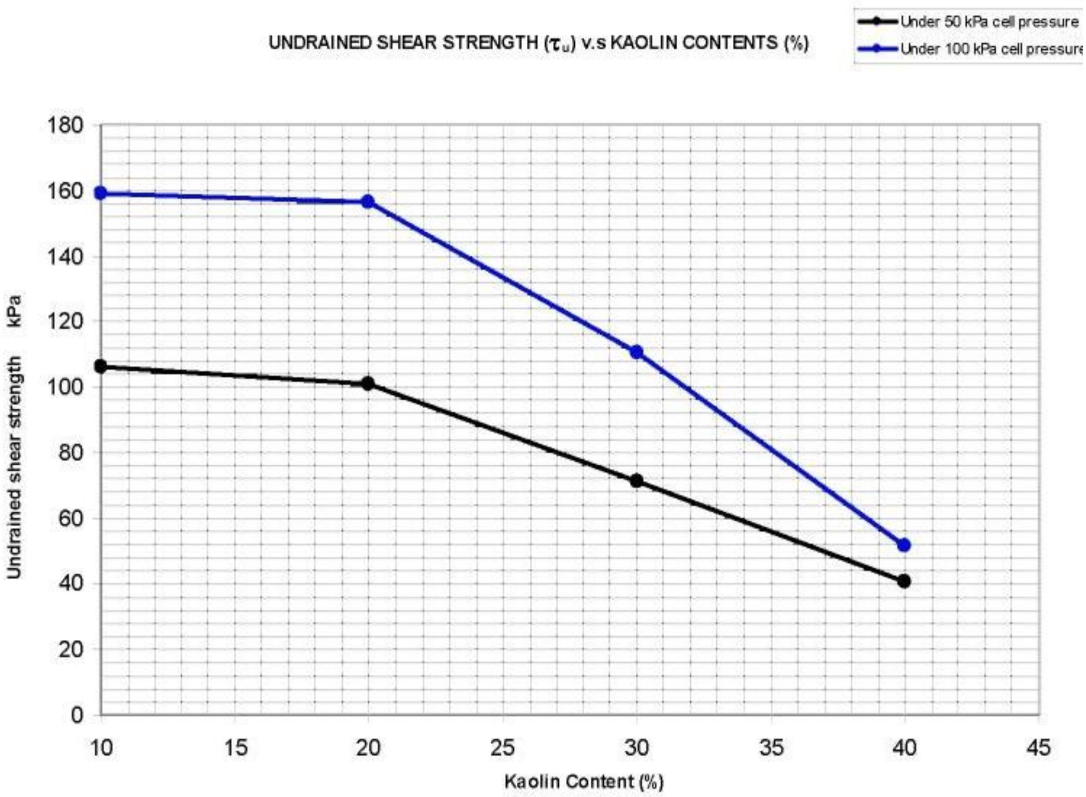

Figure 2.7. Relations between undrained shear strength and kaolin contents, in series 2 (Ölmez, 2008)

In third series, drained direct shear tests were performed on under 50, 100, and 150 kPa vertical pressures and repeated each pressures. Tests were performed on 10, 20, 30, and 40% kaolin contents on dry weight of soil mixture. Test results of this study are shown in Figure 2.8, when the kaolin content increased, effective cohesion increased. The increase rate started decrease after 20 % kaolin content and the rate decreased after 30 % kaolin content. The effective friction angle of mixture during shear decreased with increasing kaolin content in relation between drained shear strength at 50, 100, and 150 kPa vertical pressures and kaolin content was given in Figure 2.9 Drained shear strengths decreased with increasing kaolin contents shown in Figure 2.10.

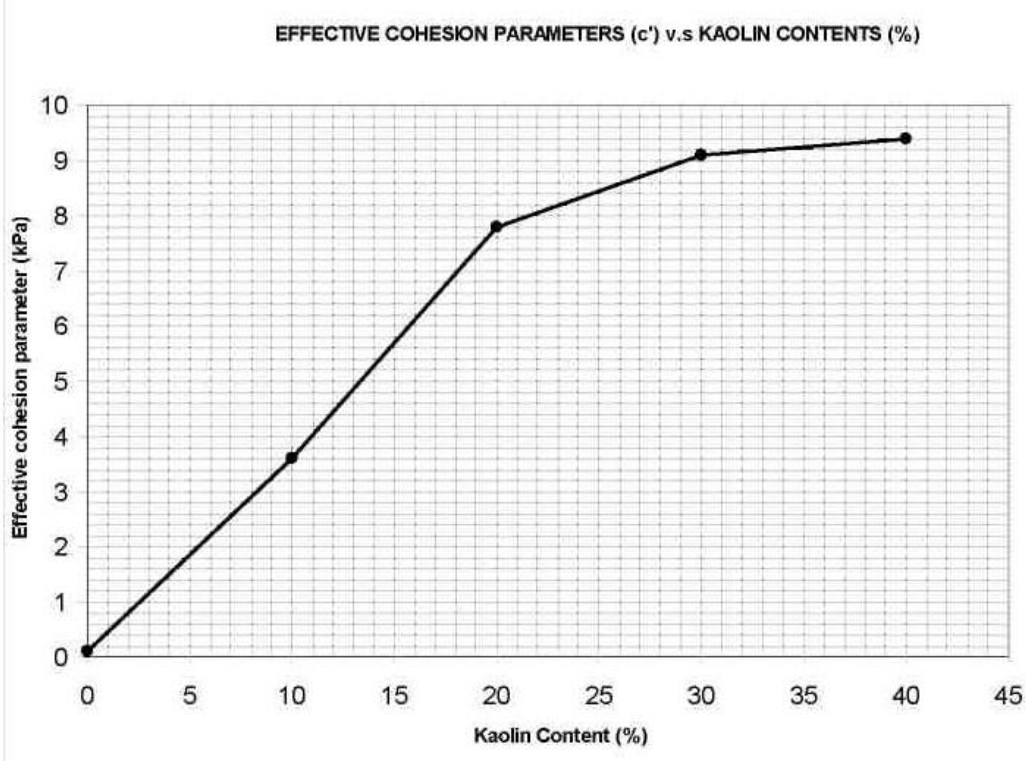

Figure 2.8. Relation between drained cohesion and kaolin content, in series 3 (Ölmez, 2008)

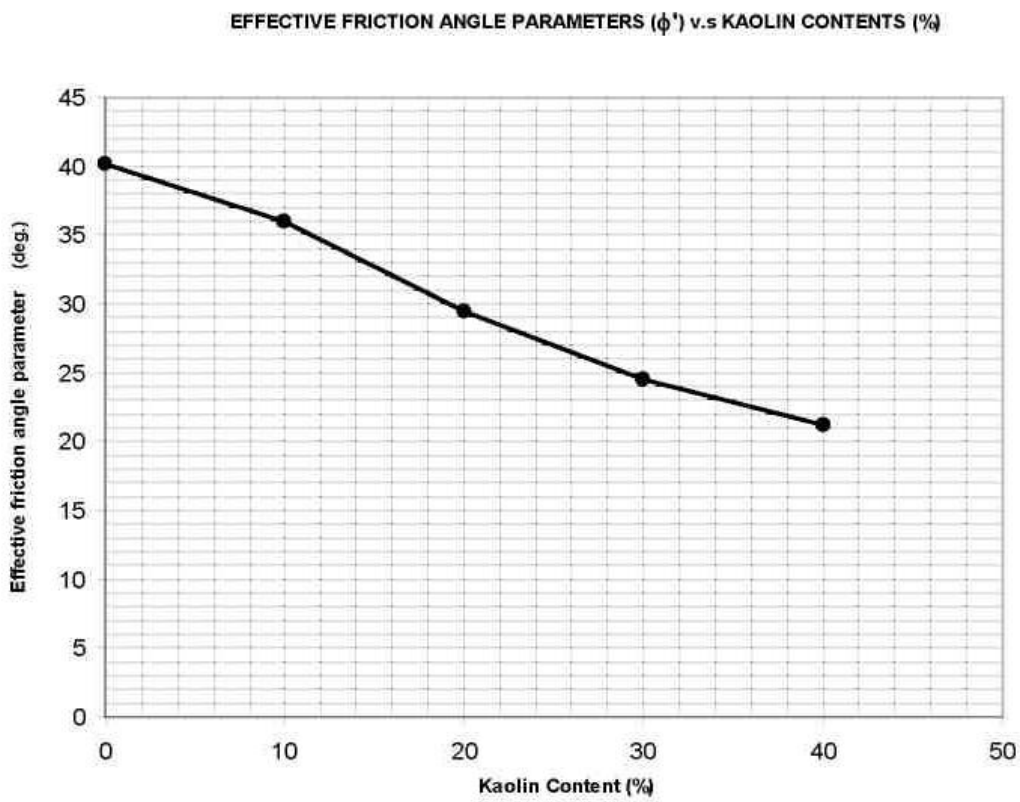

Figure 2.9. Relation between drained friction angle and kaolin content, in series 3 (Ölmez, 2008)

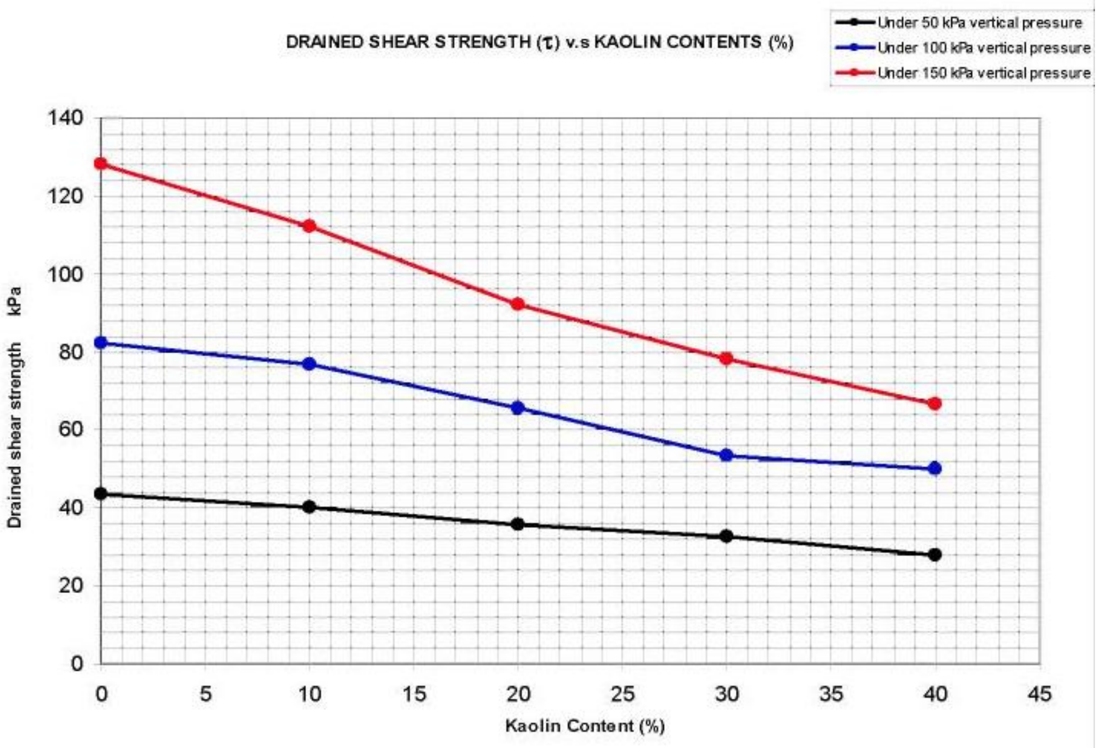

Figure 2.10. Relations between drained shear strength and kaolin contents, in series 3 (Ölmez, 2008)

In this thesis, it was aimed to investigate shear strength of soil with inclusion of sand columns in low plastic residual clay. In the study, three different diameters of clay columns and four different sand to clay area ratios were used. In all series of tests shear strength parameters internal friction angle and cohesion, stress-strain behavior, and vertical deformation were observed.

CHAPTER 3

SHEAR STRENGTH OF SOIL

3.1. Introduction

The shear strength of soil is a most important aspect of geotechnical engineering. The bearing capacity of shallow or deep foundations, slope stability, retaining wall design and, indirectly pavement design are all affected by the shear strength of the soil in a slope, behind a retaining wall or supporting a foundation or pavement. Structures and slopes must be stable and secure against total collapse when subjected to maximum anticipated applied loads. Thus limiting equilibrium methods of analysis are conventionally used for their design, and these methods require determination of the ultimate or limiting shear resistance of the soil. (Holtz and Kovacs, 1982)

3.2. Definition of Shear Strength

Mohr presented a theory for rupture in materials in 1910. According to this theory, a material fails by a critical combination of normal stress and shearing stress, and not by normal or shear stress alone. The relationship between normal stress and shear stress on a failure plane can be given by fallowing function

$$\tau_f = \sigma_f \qquad (3.1)$$

where τ_f shear stress at failure and σ is the normal stress on the failure palane. The failure envelope defined by equation 3.1 is a curved line; see in Figure 3.1 Coulomb defined the function in 1776

$$\tau_f = c + \sigma \tan \phi \qquad (3.2)$$

where c is cohesion and ϕ is the angele of friction of the soil. Equation 11.2 is Mohr-Coulomb failure creation in as shown in Figure 3.2.

In equation 3.2 defined for the total stress parameters. (c, σ, and ϕ). Equation 3.3 defined for the effective stress parameters. The effective stress σ' is carried by the soil solids. The Mohr-Coulomb failure criterion, the equation 3.3 is given in terms of effective stress,

$$\tau_f = c' + \sigma' \tan \phi' \tag{3.3}$$

where c' is cohesion of effective stress and σ′ is effective stress of soil (σ′ = σ - u, u is the pore water pressure)

Effective parameters are generally used under conditions where effects of the drained conditions on the shear strength are more critical than that of undrained conditions such as the slope stability problems. However total parameters are usually taken undarined conditions if they are more critical in shear strength problems like bearing capacity problems of shallow foundations. (Head, 1982)

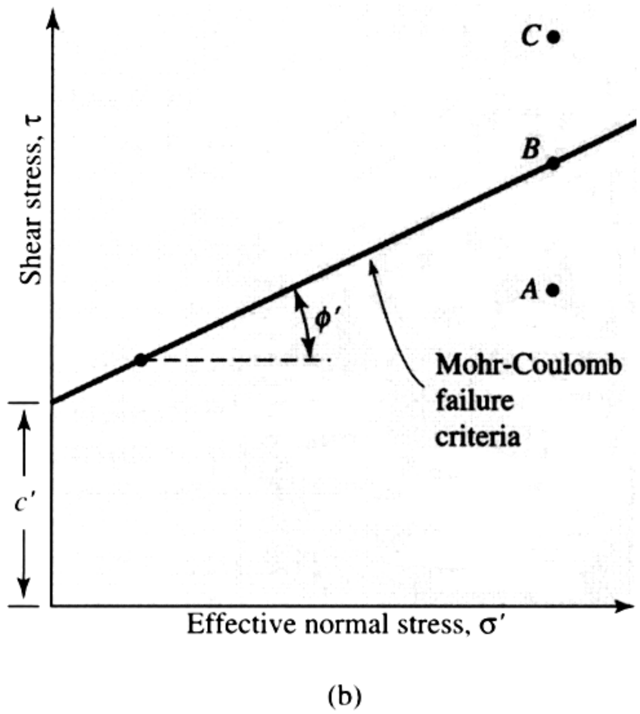

Figure 3.1. Mohr-Coulomb failure criterion (Das, 2008)

In Figure 3.1 shows the plot of the failure envelope defined by Eq. (3.3). If the values of σ′ and τ plot as point A in Figure 3.1, shear failure will not occur along the plane. Shear failure will occur, if the effective normal stress and the shear stress plot as point B. A state of stress plotting as point C con not exist since falls above the failure envelope. Shear failure would have occurred before this condition was reached.

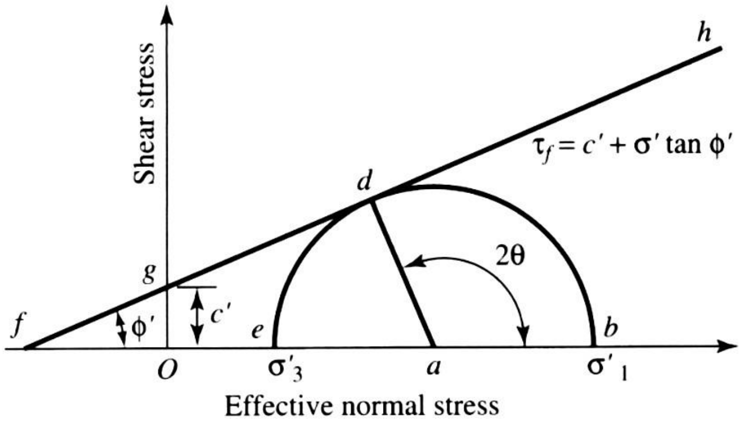

Figure 3.2. Mohr’s circle and failure envelope (Das, 2008)

3.3. Shear Strength of Cohesive Soil

Cohesive soils are generally called problematic soil by geotechnical engineers. The shearing strength of a clay soils depends on many factors. Mainly, the shearing stresses of clays are related with type of clay mineral and water content of clay. Effective stress and consolidation takes very important role on shear strength of clays. The importance of effective stress and drainage can be explained during shearing on saturated clays.

The shear strength of cohesive soils can generally defined by laboratory tests. These tests are either direct shear tests or triaxial shear tests. The triaxial tests are generally performed on cohesive soils. The shear strength with using effective stress can be given in equation 3.3. The effective cohesion, $c' \cong 0$ for normally consolidated clays, and $c' > 0$ for overconsolidated clays.

There are many differences between cohesive and cohesionless soils. These differences have an effect on the shear strength of cohesive soils. The important differences are given below:

- The frictional resistance of cohesionless soils is more than cohesive soil.
- The permeability of clay soils is less than sandy soils, and the water drainage of clays is also slower.
- Changes of volume in clays are slower than granular soil such as consolidation.

The properties of drained and undrained condition with normally consolidated and overconsolidated clays are shown in Figure 3.3 In consolidated-undrained tests, axial stress and pore water pressure are performed against axial strain. For normally consolidated clays, axial stress and pore water pressure arrive an ultimate value at relatively large strain to a steady value in Figure 3.3(a).

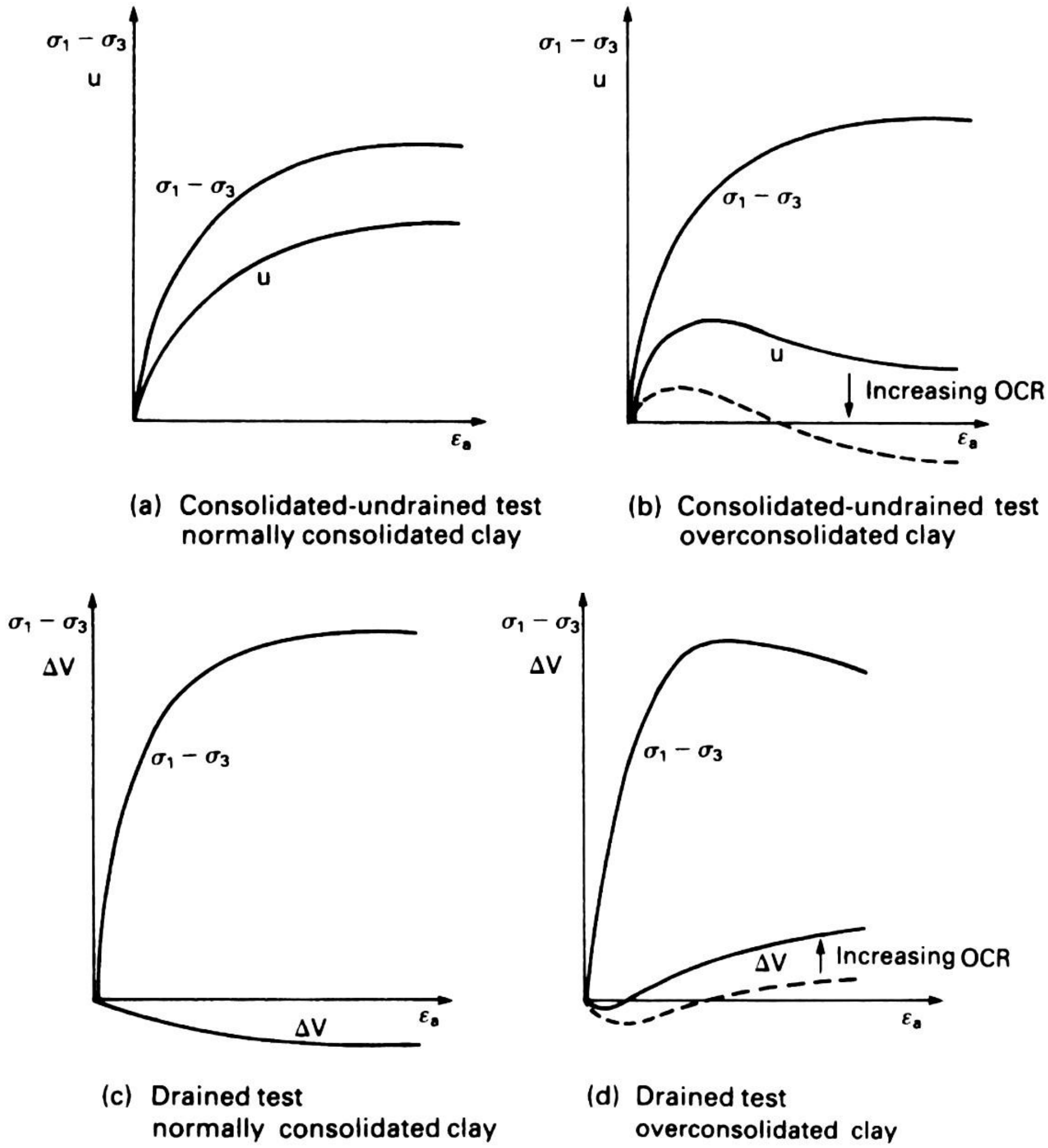

Figure 3.3. Typical results from consolidated-undrained and drained triaxial tests (Craig, 1997)

For overconsolidated clays, axial stress increases until a peak value and then decreases with following increase in strain. However, it is not usually possible to reach the ultimate stress due to excessive specimen deformation. Pore water pressure increases initially and then decreases, the higher the overconsolidation ratio the greater the decrease. Pore water pressure may become negative in the case of heavily overconsolidated clays as shown by the dash line in Figure 3.3(b). In drained tests, axial stress and volume change are plotted against axial strain. For normally consolidated clays an ultimate value of stress is again reached at relatively high strain. A decrease in volume takes place during shearing shown in Figure 3.3(c). For over-consolidated clays a peak value of axial stress is reached at relatively low strain.

Subsequently, axial stress decreases with increasing strain and the volume of an overconsolidated clay initially decrease and increases to and after peak stress and the clay softens shown in Figure 3.3(d). For overconsolidated clays the decrease from peak stress towards the ultimate value becomes less pronounced as the overconsolidation ratio decreases.

In practical situations, if the stress in a particular soil element becomes equal to the peak shear strength, any further increase in strain will result in a reduction in strength. Consequently, additional stress will be transferred to adjacent elements, perhaps resulting in peak strength being also reached in these elements. A sequence of progressive failure could thus be set in train within a soil mass. Therefore, unless it is certain that strains throughout the soil mass will remain less than that corresponding to peak strength, it is necessary to use the critical-state strength in design. (Craig, 1997)

Undrained Strength

Same void ratio and composition of unconsolidated - undrained tests results were same for different pressure shown in Figure 3.4, the failure envelope being horizontal, i.e. $\phi_u = 0$ and the shear strength is given by $\tau_f = c_u$ for fully saturated clay. Unconfined compression test would lie to the left of the effective stress circle in Figure 3.4 because of the negative pore water pressure in the specimen.

If the test sample had reduction in void ratio during each tests, the samples were not fully saturated so internal friction angle $\phi_u > 0$. The unsaturated samples have different values of internal friction angle.

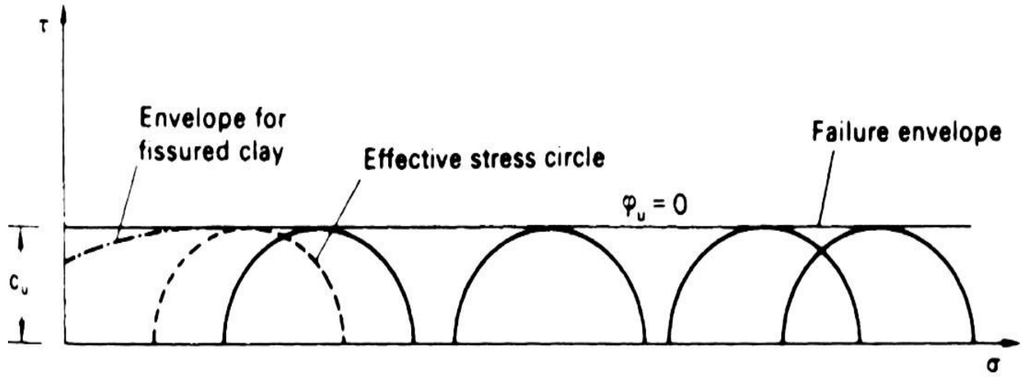

Figure 3.4. Unconsolidated-undrained triaxial test results for saturated clay (Craig, 1997)

The results of unconsolidated-undrained tests are usually represented as a plot of c_u with the suitable depth from which the sample obtained. Considerable disturbance can be expected on results plots. Undrained strength is generally increased with increasing effective vertical stress σ'_v for normally consolidated clays. This is comparable to the change of c_u with σ'_3 shown in Figure 3.5 in consolidated-undrained triaxial tests. The undarined strength value of clay taken between water table and surface is higher than that the instantaneously below the water table because of drying soil when the water table is below the surface of clay.

The consolidated undrained triaxial test shows the undrained strength of the clay to be obtained after the void ratio has been changed from the first value by consolidation. The undrained strength is thus a function of this void ratio or of the corresponding all-round pressure (σ'_3) under which consolidation took place. The

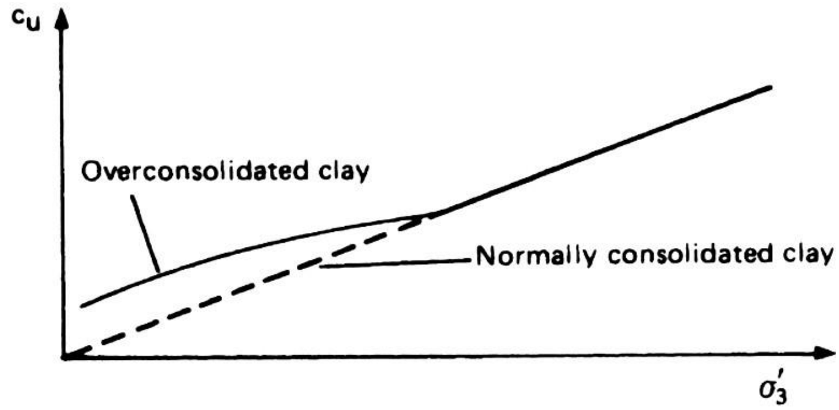

Figure 3.5. Consolidated-undrained triaxial test: variation of undrained strength with consolidation pressure (Craig, 1997)

Table 3.1. Undrained strength classification (Craig, 1997)

Stiffness state	Undrained strength(kN/m^2)
Hard	> 300
Very stiff	150 - 300
Stiff	75 - 150
Firm	40 - 75
Soft	20 - 40
Very soft	< 20

all-round pressure during the undrained part of the test (i.e. when the principal stress difference is applied) has no influence on the strength of the clay, although it is normally the same pressure as that under which consolidation took place. The results of a series of tests can be represented by plotting the value of c_u (ϕ_u being zero) against the corresponding consolidation pressure σ'_3 as shown in Figure 3.5. For clays in the normally consolidated state the relationship between c_u and σ'_3 is linear, passing through the origin. For clays in the overconsolidated state the relationship is non-linear, as shown in Figure 3.5

Clays may be classified on the basis of undrained shear strength as in Table 3.1.

Drained Strength

The hydraulic conductivity of clay is very low compared with that of sand. When a normal load is applied to a clay soil specimen for direct shear test, a sufficient length of time must elapse for full consolidation — that is, for dissipation of excess pore water pressure. For this reason, the shearing load must be applied very slowly. The test may last from two to five days. Figure 3.6 shows the results of a drained direct shear test on an over-consolidated clay. Figure 3.7 shows the plot of τ_f against σ' obtained from a number of drained direct shear tests on a normally consolidated clay and an overconsolidated clay. Note that the value of c' — 0 for a normally consolidated clay. (Das, 2008)

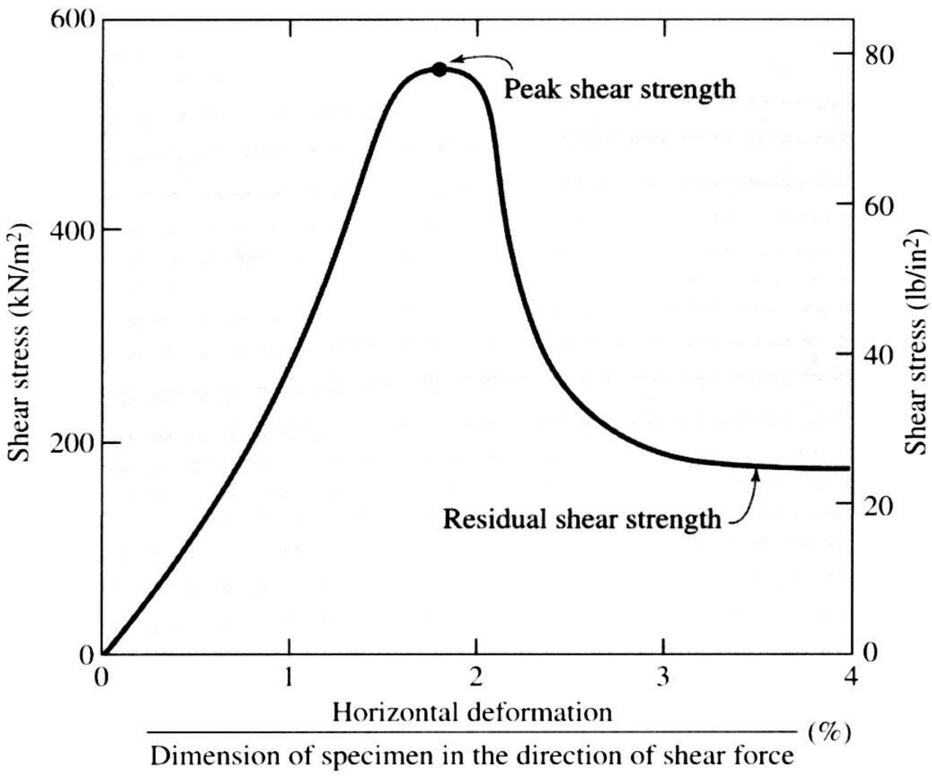

Figure 3.6. Results of a drained direct shear test on an overconsolidated clay (Das, 2008)

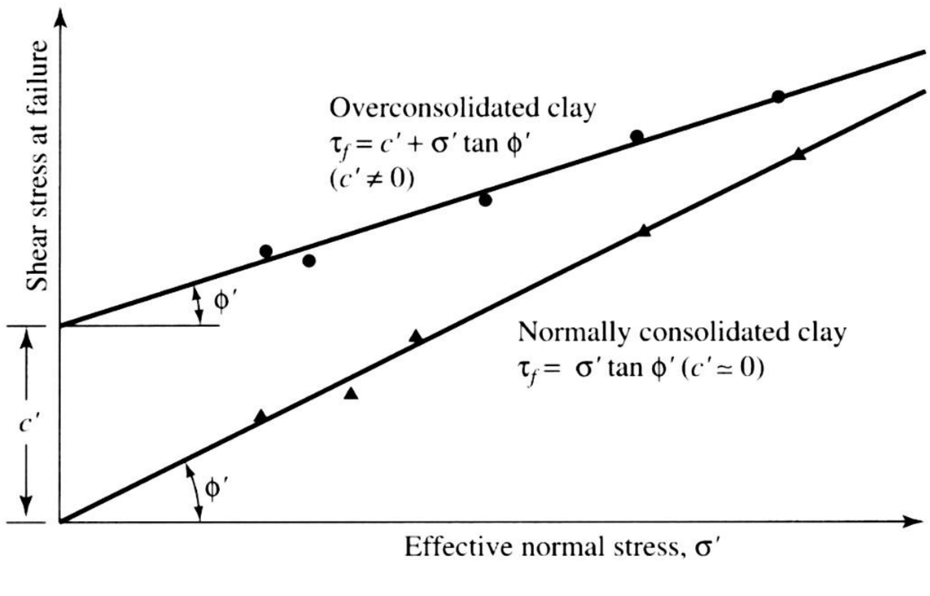

Figure 3.7. Failure envelope for clay obtained from drained direct shear tests (Das, 2008)

3.4. Shear Strength of Sand

The shear strength properties of a sand can be calculated from the results of either direct shear tests or triaxial tests. Behavior of shear stress and shear strain for dense and loose sand was shown in Figure 3.9(a) from direct shear tests. The results of triaxial compression tests results are similar too.

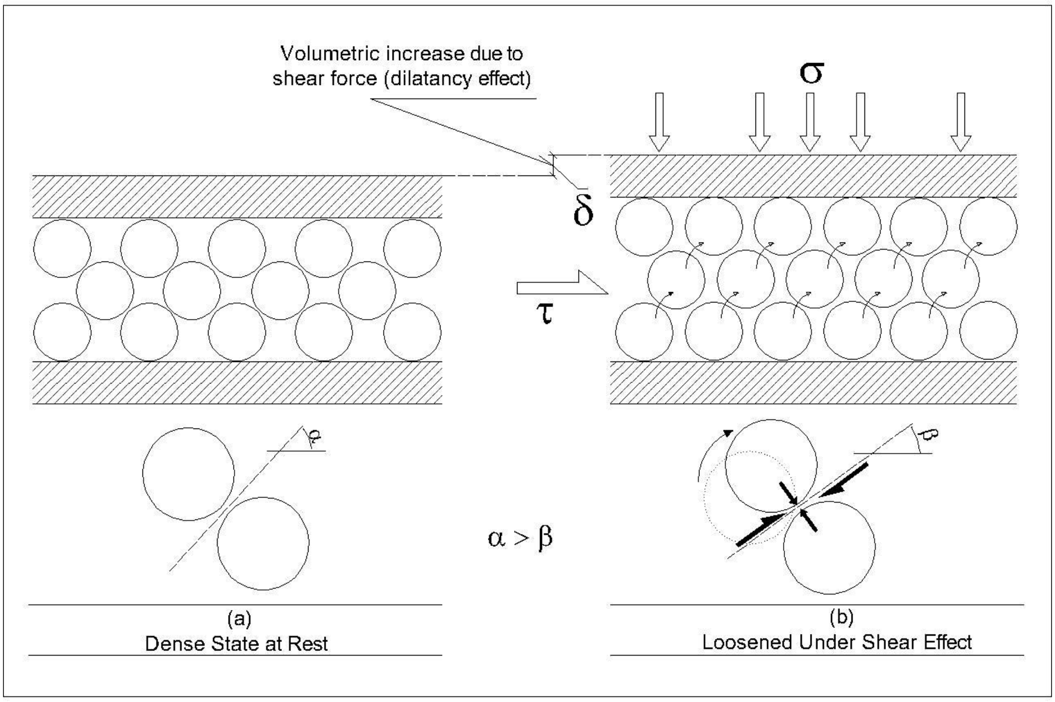

Figure 3.8. Illustration of soil behavior for dense soil state

Dense sands have a significant degree of interlocking between particles, which needs to be overcome by the frictional resistance before shear failure. Degree of interlocking is the biggest in very dense sands. The characteristic stress strain curves have peak stress and low strain initially in dense sand. After the overcoming interlocking, stress decrease with increasing strain. The decrease of interlocking causes an increase in the volume of the sample during shearing, see in Figure 3.9(c). The change in volume shows change in void ratio (e) in Figure 3.9(d).

The increasing volume during shearing in dense sand is called dilatancy (Figure 3.8). It is defined that shear plane is horizontal but sliding between individual particles takes place on inclined at various angles above the horizontal, as the particles move up and over their neighbours during shearing of dense sands.

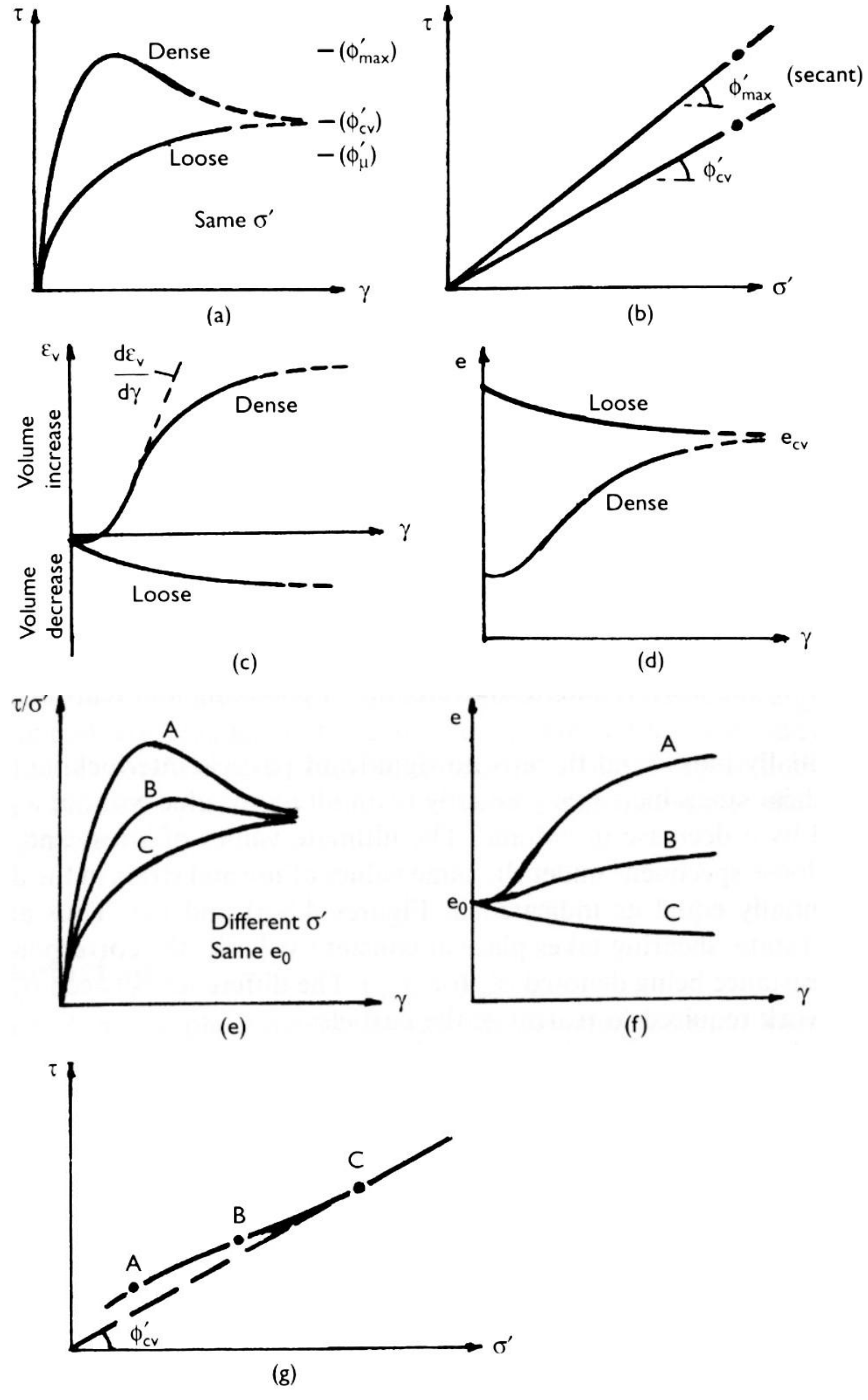

Figure 3.9. Shear strength characteristics of sand (Craig, 1997)

The angle of dilation (ψ) defines an average value of this angle for whole specimen. The loading is forced upwards; work being done towards the normal stress. The maximum angle of shearing resistance (ϕ'_{max}) calculated from peak stresses for dense sand (Figure 3.9 (b)), which is greater than the true angle of friction (ϕ_u) between the surfaces of individual particles. The difference showing the work required to pass interlocking and reorganize the particles.

Different method of defining the results from direct shear tests is to plot the stress ratio τ/σ' against shear strain. Stress ratio towards to shear strain performing tests on three sample of sand at different effective normal stress. Everyone has initially same void ratio, are shown in Figure 3.9(e). The effective normal stress being lowest in test A and highest in test C. Void ratio against shear strain curve are shown in Figure 3.9(f). This results show that void ratio decrease with increasing effective normal stress. Ultimate stress reduces with increasing effective normal stress shown in Figure 3.9(g). ϕ'_{max} values for each test can be defined by a secant parameter, the value reducing with rising effective normal stress until it becomes equal to ϕ'_{cv}. The decrease in the difference between maximum and ultimate shear stress with increasing normal stress is mainly with from decrease in ultimate void ratio. The lower the ultimate void ratio the less scope for dilation so less soil particle can overcame to interlocking and than the other soil particles fractures and crushes at high stress thus, causes of less value of ϕ'_{max}.

In practice, the routine laboratory testing of sands is not applicable because obtaining undisturbed sample is very difficult. The parameters ϕ'_{max} and ϕ'_{cv} is given in certain codes of practice. In the case of dense sands it has been shown that the value of ϕ'_{max} under conditions of plane strain can be 4° or 5° higher than the corresponding value obtained by conventional triaxial tests. The increase in the case of loose sands is negligible.

3.5. Shear Strength Tests

Shear strength tests were explained in two main groups. These are laboratory and in situ tests.

3.5.1. Laboratory Tests

The laboratory tests have two parts for calculating of shear strength. These laboratory tests are direct shear test and triaxial shear test.

3.4.1.1. Direct Shear Test

Direct shear test is used to evaluate the shear strength parameters, namely; ϕ' and c' or shear strength of a soil under drained condition. It has been one of mostly used tests in geotechnical engineering applications because of its simplicity and repeatability. The main drawback of the shear box test is that the specimen is not failing along its weakest plane but along a predetermined or induced failure plane i.e. horizontal plane separating the two halves of the shear box.

The reliability of the friction angle of sands obtained from the test was questioned by Cerato (2006). The research concluded that the friction angle is affected by the scale of the shear-box used in the tests. The tests indicated that for well-graded, angular sands, ϕ′ decreases as box size increases.

Moreover, the reproducibility of the test is claimed to be weak (Bareither, 2008), i.e. the test results may remarkably change due to laboratory conditions and to particularly sample preparation techniques employed. Nevertheless, the repeatability of the test is reported to be high which means that the test results are highly consistent with each other as long as the test is carried out in the same laboratory, under same conditions and supervision of the same operator. Therefore, the test is still to be accepted as an applicable way of shear strength determination unless a better method of testing to replace it is pioneered.

The test equipment consists of a metal shear box in which the soil specimen is placed. The soil specimens may be square or circular in plan. The size of the specimens generally used is about 60 mm X 60 mm and about 25 mm high. A square specimen of soil is placed in a rigid box that is divided horizontally into two pieces. The specimen is confined under a vertical stress, and a horizontal force is applied so as to fail the specimen along a horizontal plane. (Figure 3.10)

On this plane there are two stresses acting; a normal stress (σ) due to applied vertical load (P_v), and a shearing stress due to the applied horizontal load (P_h). These stresses are simply computed as

$$\sigma = P_v / A \tag{3.4}$$

$$\tau = P_h / A \tag{3.5}$$

where; A is the nominal area of the sample (or of the shear box) and is not corrected for lateral displacement under shear force P_h. (Bowles, 1988)

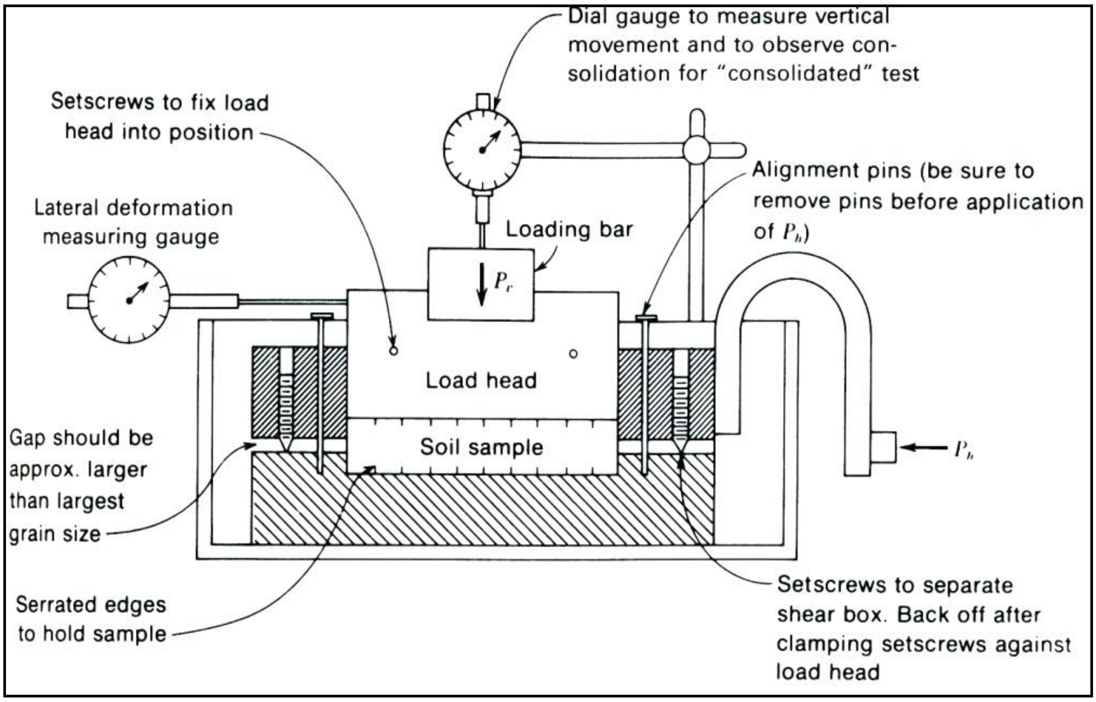

Figure 3.10. Details of the direct shear test method and its apparatus

Direct shear tests are repeated on similar specimens at various normal stresses. The normal stresses and the corresponding values of τ_f obtained from a number of tests are plotted on a graph from which the shear strength parameters are determined.

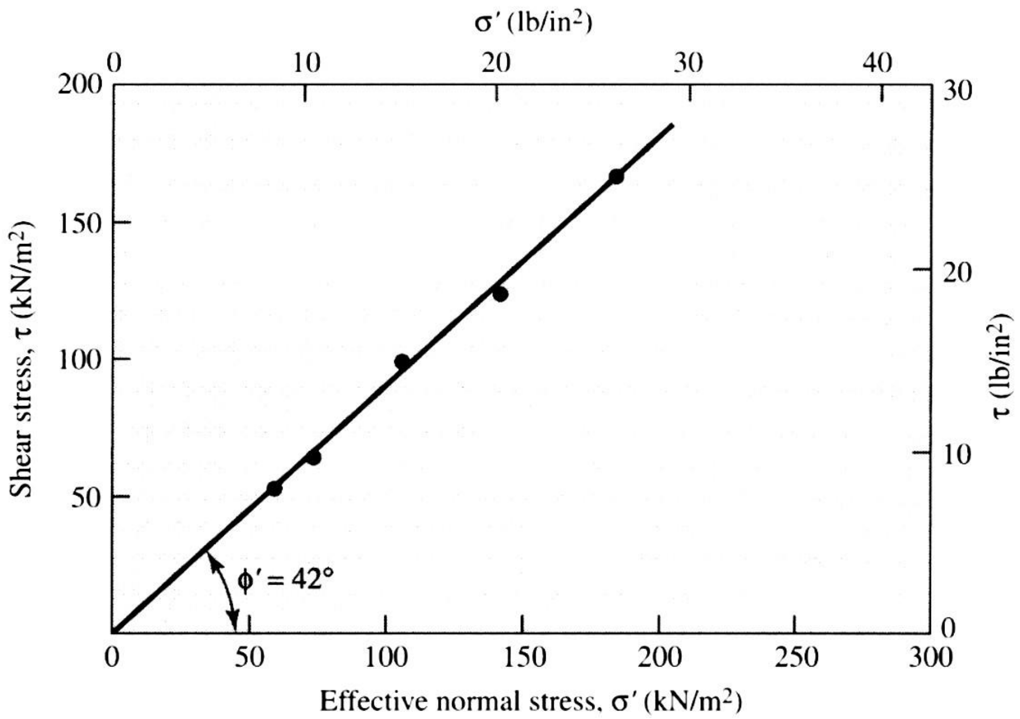

Figure 3.11. Determination of shear strength parameters for a dry sand using the results of direct shear tests (Das, 2008)

Figure 3.11 shows such a plot for tests on a dry sand. The equation for the average line obtained from experimental results is

$$\tau_f = c' + \sigma' \tan \phi' \tag{3.6}$$

So, the friction angle can be determined as follows:

$$\phi' = \tan^{-1}(\tau_f / \sigma') \tag{3.7}$$

It is important to note that in situ cemented sands may show a c' intercept.

3.4.1.2. Triaxial Shear Test

The triaxial shear test is one of the most reliable methods available for determining shear strength parameters. It is common used for exploration and general testing. A diagram is defined the triaxial test equipment in Figure 3.12.

In this test, a soil sample has 36 as a diameter and 76 mm long. The sample is closed by a thin rubber cylindrical box and placed inside a plastic cylindrical chamber that is usually filled with water or glycerin. The specimen is exposed to a confining pressure by compression of the fluid in the chamber. This compression is causes shear failure in the sample. This stress can be performed in one of two ways:

- Using of dead weights or hydraulic pressure in equal increase until the specimen fails.
- Using of axial deformation at a constant rate by means of a geared or hydraulic loading press. This is a strain-controlled test.

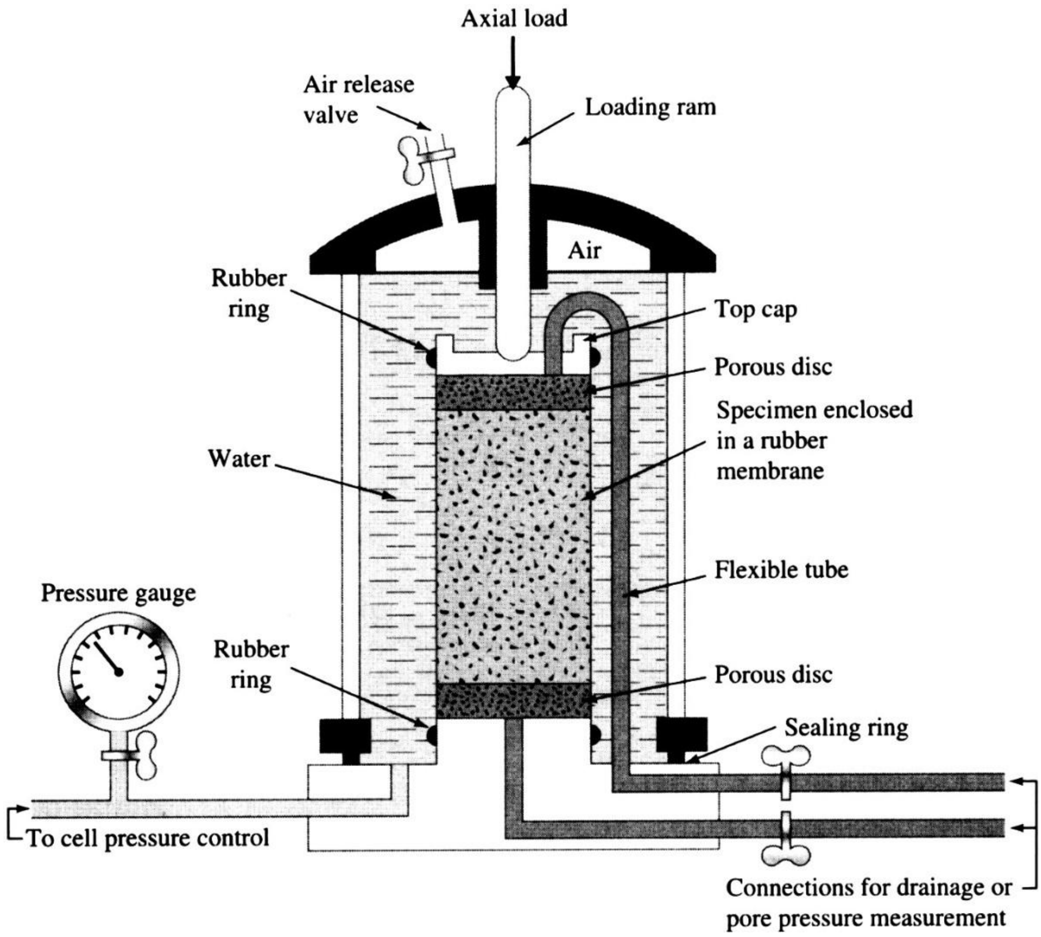

Figure 3.12. Diagram of triaxial test equipment (after Bishop and Bjerrum, 1960) (Das, 2008)

Many kinds of tests procedures can be performed with the triaxial apparatus. The three most common types of triaxial tests are as fallows.

1. **The unconsolidated–undrained (UU) test:** The test sample is performed on horizontal and vertical stresses. The stresses generally equal to vertical stress. The stress was present in the field, which are applied to the sample.

Consolidation is not permitted, and the soil sample is under drained condition.

2. **The consolidated drained (UU) test:** The horizontal and vertical stresses are generally equal or bigger than the vertical stress, which was present in the field. The soil is permitted to on consolidate. The test is performed under undrained condition s. The test is carried out to find the drained c and ϕ.
3. **The consolidated-undrained test:** The initial stresses are applied like CD test, The soil is permitted to consolidate but the test is performed umder undrained condition. However, the drained c and ϕ values can be calculated from this test and obtained pore water pressures during the test and calculating the effective stresses. (Coduto,2001)

3.5.2. In Situ Tests

In situ tests were explained in three different tests for calculating shear strength. The tests are shear vane tests, standard penetration test, and cone penetration test.

3.4.1.3. Shear Vane Test

The shear vane device consists of four bladed vane welded to a rod shown in Figure 3.13. The vane is generally inserted into the ground to the defined depth. A steadily increasing (6° per minute) torque is applied until the soils shear failure. The undrained shear strength is calculated from the maximum torque. The torque is obtained during shear failure. This test is used very soft and soft clays and silts at the bottom of boring. The undrained shear strength is computed from equation 3.8

$$S_u = \frac{2T}{\pi d^3 \left(\frac{h}{d} + \frac{1}{3}\right)} \qquad (3.8)$$

where *T* is the maximum torque, *h* is the height, and *d* is the diameter of the vane.

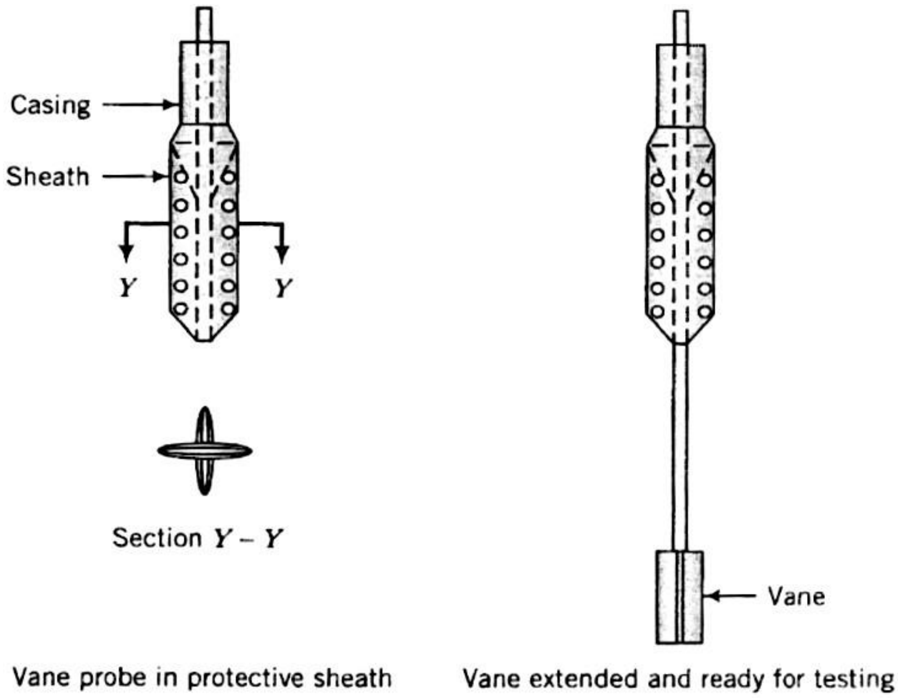

Figure 3.13. Shear vane tester (Budhu, 2000)

3.4.1.4. The Standard Penetration Test

The SPT is performed by using a standard split spoon sampler into the ground by blows from a drop hammer of mass 64 kg falling 760 mm in Figure 3.14 sampler is used 150 mm into the soil at the bottom of a borehole, and the number of blows (N) required to use it an additional 300 mm is counted. The number of blows (N) is called the standard penetration number.

Different corrections are applied to the N values to find for energy losses, overburden pressure, rod length. It is usual to correct the N values to a rod energy ratio of 60%. The rod energy ratio is the ratio of the energy transferred to the split spoon sampler to the free-falling energy of the hammer. The corrected N values are shown as N60. The N value is used to calculate the relative density, friction angle, and settlement in coarse-grained soils. The test is very simple, but the results are difficult to interpret.

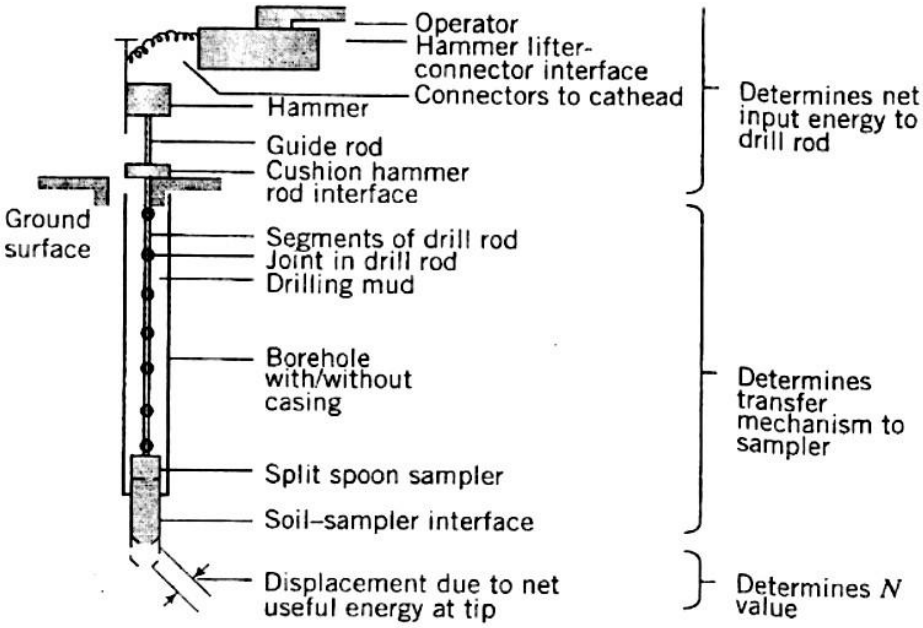

Figure 3.14 Standard penetration test. (Budhu, 2000)

Table 3.2. Correlelation of N, N_{60}, γ, D_r, and ϕ' for coarse grained soils (Budhu, 2000)

N	N_{60}	Description	γ (kN/m^3)	D_r (%)	ϕ' (°)
0-5	0-3	very loose	11-13	0-15	26-28
5-10	3-9	loose	14-16	16-35	29-34
10-30	9-25	medium	17-19	36-65	35-40
30-50	25-45	dense	20-21	66-85	38-45
>50	>45	very dense	>21	>86	>45

Table 3.3. Correlation of N_{60} and s_u for saturated fine grained soils (Budhu, 2000)

N_{60}	Description	s_u (kPa)
0-2	very soft	< 10
3-5	soft	10-25
6-9	medium	25-50
10-15	stiff	50-100
15-30	very stiff	100-200
>30	extremely stiff	>200

Typical correlation in *N* values, relative density, and ϕ' are given in Table 3.2 and Table 3.3. You should be careful in using the correlation in Table 3.2 and Table 3.3 to determine the properties of soils and to design foundations because the data is generally between large intervals and the correlation coefficients are low.

3.4.1.5. The Cone Penetration Test

The cone penetration is a cone with a maximum area of 10 cm^2 in Figure 3.15that is attached to a rod. An outer sleeve covers the rod. The thrusts required to use the cone and the sleeve into the ground are measured independently so that the end resistance or cone resistance and side friction or sleeve resistance may be estimated separately. Although the test actually was improved for the design of piles, the cone penetration is used to calculate the bearing capacity and settlement of foundations.

The cone penetration has porous elements inserted into the cone or sleeve to allow for pore water pressure measurements in Figure 3.15. The measured pore water pressure depends on the location of the porous elements. A load cell is often used to measure the force of penetration.

The cone resistance q_c is normally correlated with the undrained shear strength. Several adjustments are made to q_c. One correlation equation is

$$s_u = \frac{q_c - \sigma_z}{N_k} \tag{3.9}$$

where N_k is a cone factor that depends on the geometry of the cone and the rate of penetration. Average values of N_k as a function of plasticity index can be estimated from

$$N_k = 19 - \frac{(I_p - 10)}{5}; \quad I_p > 10 \tag{3.10}$$

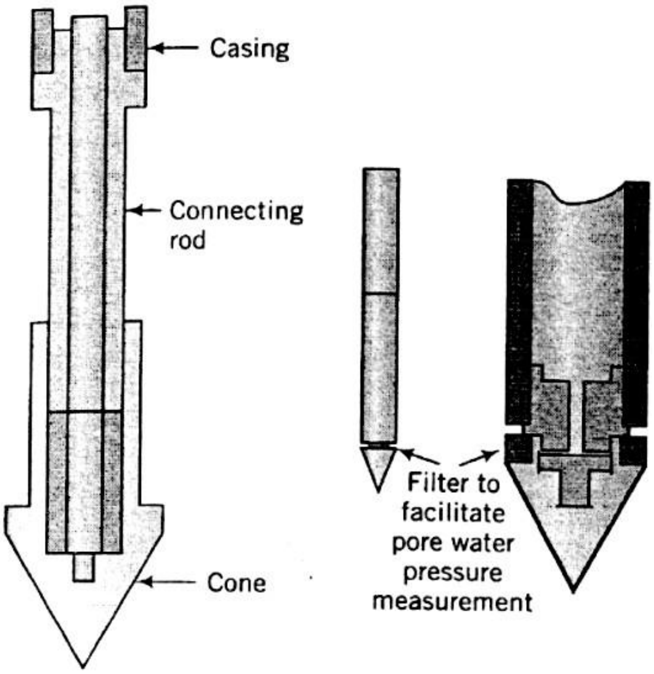

Figure 3.15 Dutch cone and piezocone (Budhu, 2000)

CHAPTER 4

EXPERIMENTAL STUDY

4.1. Introduction

This research is mainly based on the interpretation of the results particularly of the direct shear box test. As mentioned earlier, the study, in very general terms, aims to explore the change in shear strength parameters of clay soil when it is reinforced with sand soil filled columns. Therefore there were mainly two soil materials involved in the study. These materials were not ordinarily selected but they were rather required to ensure some physical properties such as grain size and density. In order clearly to define such properties, some other principal laboratory experiments, such as Sieve Analysis, Atterberg Limits, Unit Weight and Standard Proctor Test were also performed during the study.

Here, in this chapter the materials and the methods utilized for the experiments throughout the study will be defined first. Not only the definitions but also the other supplementary tests to reveal their physical properties will be explored under subsections of Clay, Sand and Water. This will include the test procedures followed, the results achieved and their evaluations for these descriptive tests.

Later in this chapter, the direct shear box test as the backbone of the study will be explained in detail under an individual section. The main characteristics of the test, study-specific application details, presentation of the results as well as their interpretation will be covered in that section. These will later make the foundation of the conclusions.

4.2. Materials and Methods

It was used clay, sand, and water as materials for the tests in this study.

4.2.1. Clay

Tests were performed on low plastic red residual clay collected from University of Gaziantep Campus. Some tests were performed on clay to define its geotechnical properties. These tests were sieve analysis test, atterberg limits (liquid and plastic limit tests) tests, and standard proctor tests. The sieve analysis test was performed on clay adapted from ASTM D422. The collected soil was passed from the 0.075 mm sieve. The soil percentage of passing from the sieve was approximately more than 80 % so the soil was admitted in the name of clay. The atterberg limits tests were performed on clay conforming ASTM D 427 and D 4318. The result of the liquid limit and plastic limit of clay were calculated 40% and 24% respectively. Standard proctor tests were carried out on clay to find maximum dry density and optimum water content of clay. The tests were adapted from ASTM D698. The obtained values from the tests were shown in Figure 4.1. Maximum dry density and optimum water content calculated from standard proctor tests were 1.69 t/m^3 and 18.8 % respectively. Thus, maximum unit weight was 16.6 kN/m^3.

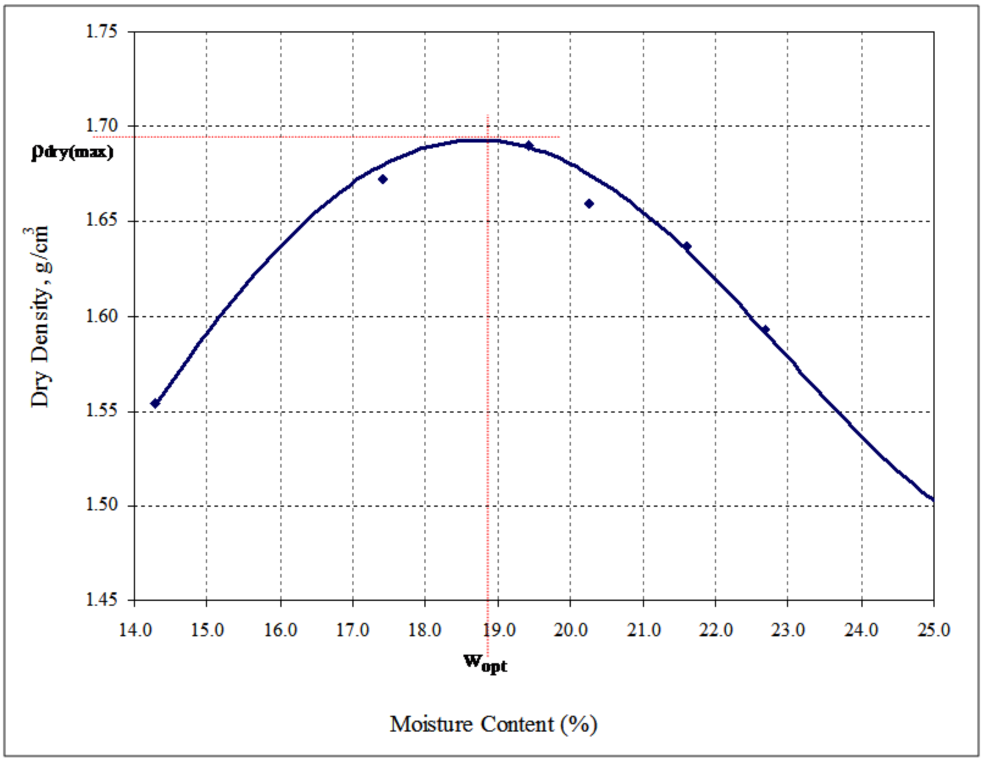

Figure 4.1. Determination of maximum dry density and optimum water content

4.2.2. Sand

The sand material was selected from those used for ordinary constructional purposes. However, for the sake of precision, the soil was sieved and those passing through #5 and remaining on #40, of which particle sizes range between 0.850 and 0.600 mm were separated to be used for the experiments. The unit weight test was performed on loose and dense sand. The unit weights of loose and dense sand obtained from tests was 13.9 kN/m^3 and 16.3 kN/m3.

4.2.3. Water

Tap water was used for all kind of works.

Figure 4.2. Direct shear test machine

4.3. The Methods of Direct Shear Test

In this research the direct shear test was executed in accordance with ASTM D 3080-90. A square box is rigid, metal, opens at the top, and divided horizontally into two halves. The upper half of the box is able to move horizontally under a vertical stress and a horizontal force so the specimen is shared along the horizontal plane. The shear force is applied horizontally to the sample by direct shear machine in Figure 4.2. The horizontal movement is a free movement of the lower and upper halves of the shear box, the box is attached on ball-bearing slides. When the shear force started to be applied to sample, the horizontal deformation of sample is recorded from dial gauge attached on machine. In addition, there is a dial gauge vertically attached the machine on the sample which measures vertical deformation. The details of direct shear test apparatus is shown in Figure 4.3.

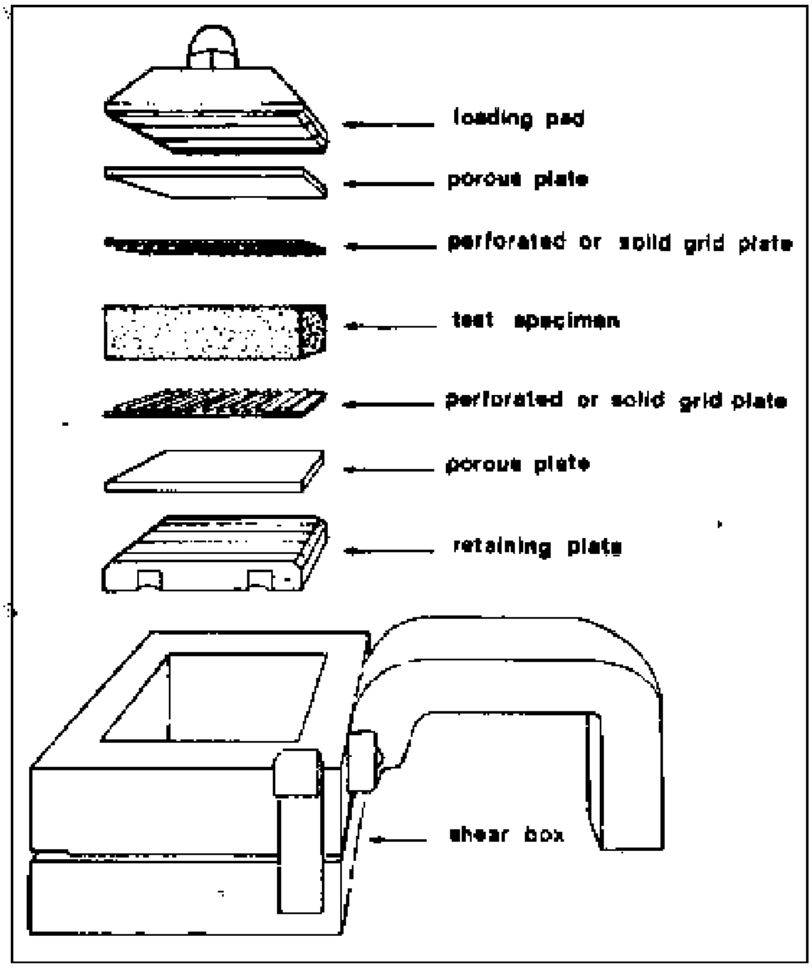

Figure 4.3. Details of the direct shear test apparatus

The research was mainly based on the results obtained from direct shear box test which makes the conclusions highly dependent on the procedure applied for this specific study. Therefore, it needs some clarifications to be made on the procedure adopted for the series of tests performed in the laboratory.

4.3.1. The Test Procedure Adopted

The results obtained from the direct shear box test are reported to be remarkably affected by the test conditions, sample preparations and the test procedure adopted. Therefore, for the sake of calibration, it seems appropriate to give details of the test procedure followed throughout the study in order effectively to interpret the results.

- The tests were conducted inside a shear box of 60 x 60 mm in plane and 25 mm in depth with approximately 150 grams of soil,
- The clay soil was taken directly from standard proctor mould as compacted at its optimum water content whereas the sand soil was compacted in place by the aid of a vibrator.
- Application of normal stress acting on the shearing surface was achieved by adding standardized loads to ensure total stresses of σ_n= 50, 100 and 150 kPa for three different stages,
- Before shearing load is applied the upper half of the shear box was raised about 0.6 mm by using the screws provided for this purpose.
- The tests was performed in unconsolidated and undrained conditions
- The loading rate was selected to be 1.0 mm/min.
- Shear stresses were recorded as a function of horizontal displacement up to a total displacement of 12.0 mm to observe the post-failure behaviour as well.
- The shear forces and the vertical displacements were recorded at every 0.2 mm of the shear displacement for the first 2.0 mm of total displacement. The reading span, later, was increased to 0.5 mm of the shear displacement for the rest of the test. This is mainly because that the difference between the vertical displacement readings becomes less remarkable after 2.0 mm of the total shear displacement,
- As the test is completed for 12.0 mm of total shear displacement the machine was reversed to release load.

4.3.2. The Test Schedule Applied

As mentioned earlier, the study, in very general terms, aims to explore the change in shear strength parameters of clay soil when it is reinforced with sand columns. Therefore, it was required first to have the shear strength parameters separately available for the two distinct materials of clay and sand. In this sense, the test was scheduled such that firstly the shear strength parameters of pure clay and secondly that of pure sand was determined. Having these results, another direct shear-box test setup was prepared to reveal the effects sand-filled columns in a clay matrix. This was repeated for different sized and number of sand columns to also reveal the size effect within the same setup.

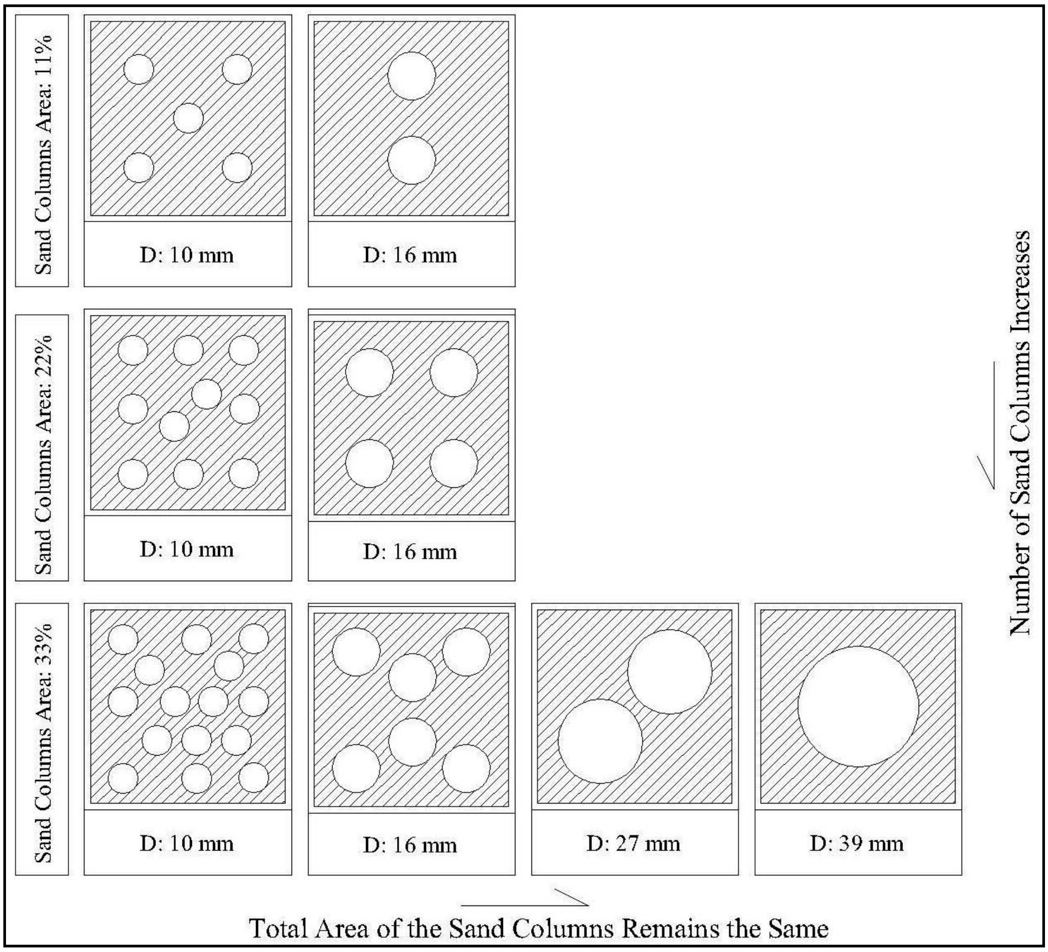

Figure 4.4. Illustration of Sand Column placement in the shear box test setup

The most important point to consider in this schedule is the selection of size and number of the sand columns used in the third setup. This makes up the foundation of the research undertaken. The main approach was to keep the total sand columns cross-sectional area, within the total surface of the shear box, constant while interchanging between the diameter and the number of the sand columns. This setup was tried to be illustrated as given in Figure 4.4. The total cross-sectional areas for the sand columns, therefore, were expressed as the ratio of total sand area over the total surface area being tested. This ratio was planned to be 11, 22, and 33% for three different test setups. Percent values of sand, therefore, represent the proportion of the section area of the sand columns to the whole shearbox area including both sand and clay surface areas together.

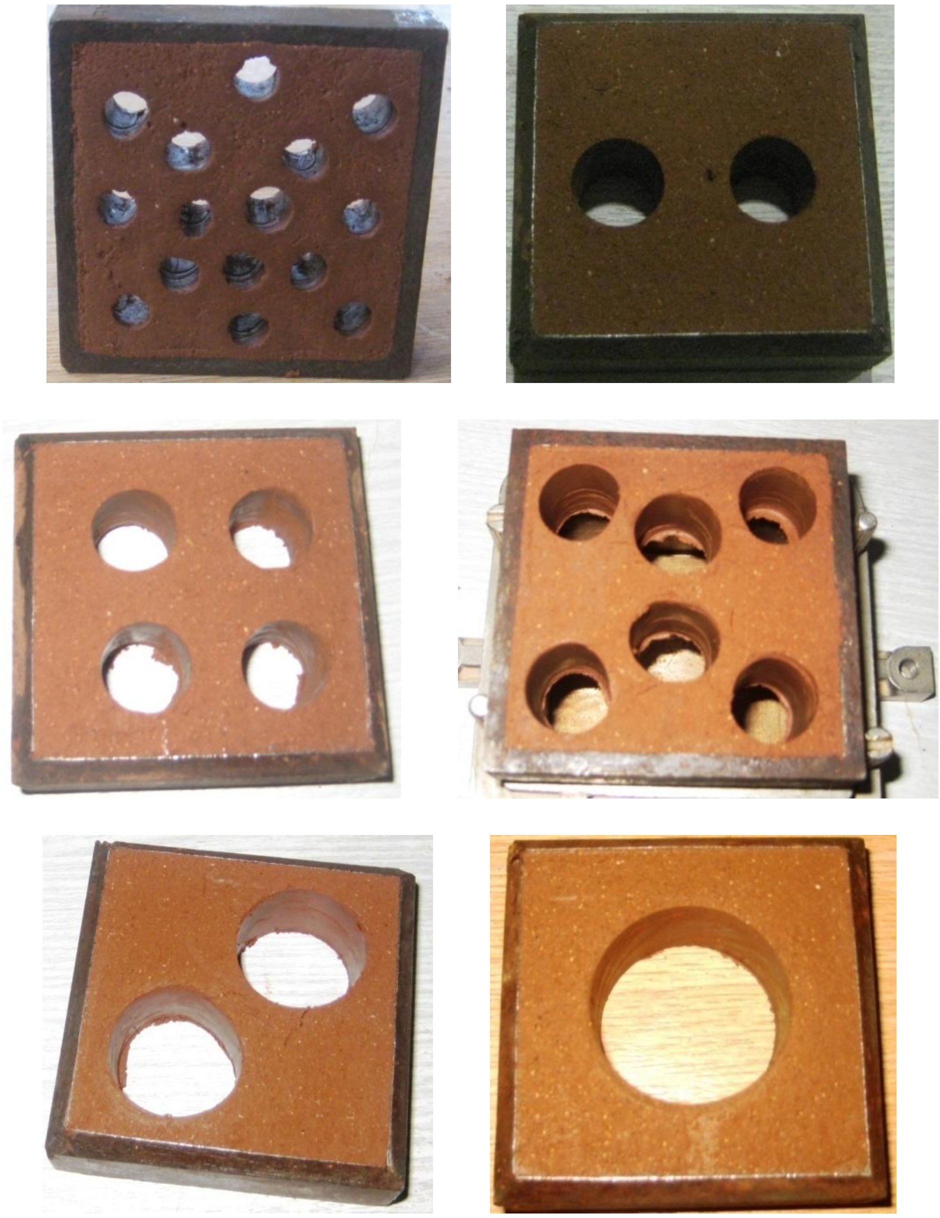

Figure 4.5. Opening of the sand column place into sample

Samples were prepared in three stages. In the first part clay was compacted by standard proctor test at its optimum water content. The compacted clay was placed in the sear box apparatus.

In the second part a thin pipes with its different diameters were used to open holes in the compacted clay samples shown in figure. Diameters of the holes opened in the clay were 10, 16, 27, and 37 mm. in Figure 4.5.

Dry sand was placed in the holes. Sand was compacted with high frequency vibratory hand compactor because sand had a loose density at the first. Vibration was taken 30 second for all the holes to keep density of the sand constant. In order to investigate effect of number of sand columns, different number of hole were opened in the clay. Sand columns had different diameters placed in clay specimen in the shear apparatus and compacted shown in Figure 4.6

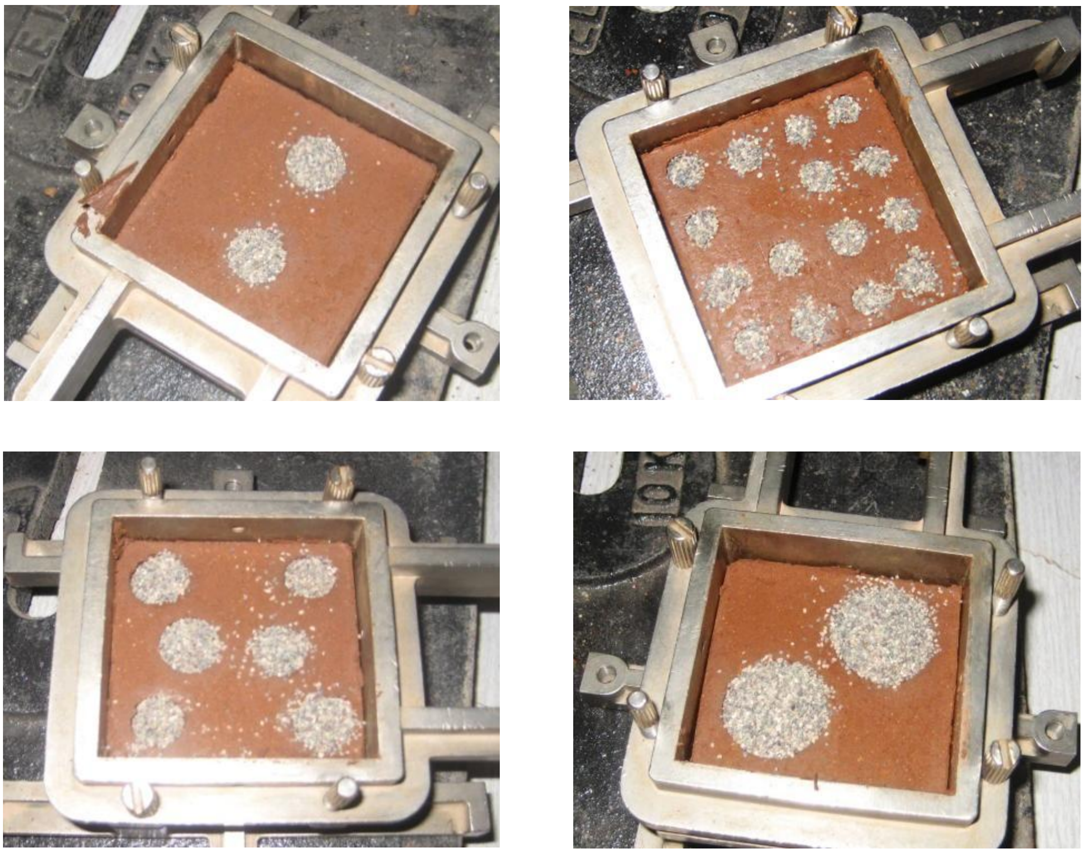

Figure 4.6. Clay with sand columns having different diameter in the direct shear apparatus

In this way, not only the effect of intensity of the sand area but also the scale effect of the sand area was investigated. Keeping the total sand area constant while changing the diameter of the sand columns has, therefore, revealed the size effect phenomena on shear strength parameters for this composite surface. With this setting, 2 different test cases for 10% and 20% of sandy specimens and 4 other test cases for 30% of sandy specimens were planned and performed.

The results and their interpretations will be given in more detail in the following section.

CHAPTER 5

TEST RESULTS

The friction between sand particles is due to sliding and rolling friction as well as interlocking action (Figure 3.8). It also depends on state of compaction, coarseness of grains, particle shape and roughness of grain surface and grading. This statement keeps valid for all types of soils as long as they are tested in dry conditions (Lambe and Whitman, 1969). That is to say that friction angle of sands as well as clays is defined by the resistance of the soil particles to shear force as they are rearranged by sliding and rolling on each other and particularly by breaking through each other. In the absence of water all soil particles regardless of their size are known to behave the same. Friction angle reported to vary between 25^{o} for clay soils and about 45° for well graded sands with angular grains in dense state. (Budhu, 2000)

The particular tests performed in the laboratory were assumed to be unconsolidated and undrained due to high loading rate (1mm/min). However, the soil specimens were not fully saturated. The maximum water content was around the optimum water content of 18.8%.

The results obtained will be presented in the following order;

1. The shear strength parameters obtained for pure clay at water content (WC) of 18.8%
2. The shear strength parameters obtained for loose sand
3. The shear strength parameters obtained for dense sand
4. The shear strength parameters for composite soil body at different sand columns.

5.1. Shear Strength Parameters Pure Clay

The tests were performed at a water content of 18.8% which is optimum water content for this particular clay. The direct shear test was aimed to be performed at optimum water content in all of the study.

Clay was first compacted at its optimum water content of 18.8%. It was later extracted from the mould to be tested in the shear box. Interpretation of results will be given here in this section. Results were plotted as shear stress versus strain. A typical curve for such a clay soil was obtained as given in Figure 5.1. According to the result, shear stress increases with increasing strain until peak point of the shear stress after this point shear stress slightly decreases. In addition, shear stress increases with increasing normal load so that shear stress reached 136 kPa for the highest loading shown in the figure for the pure clay.

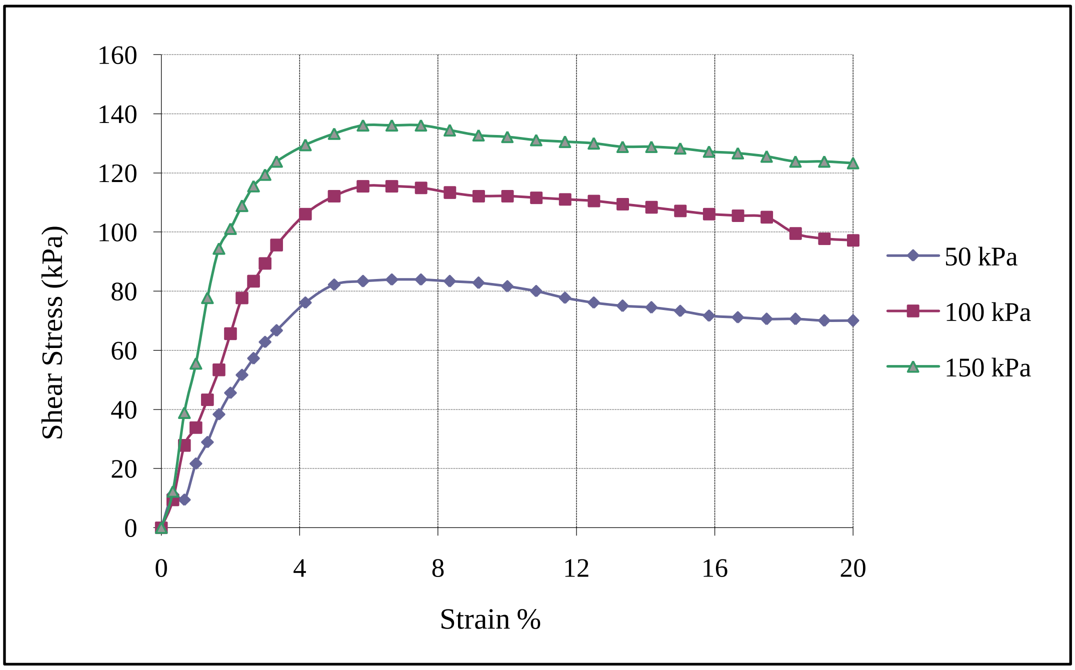

Figure 5.1. Plot of shear stress vs. strain for pure clay soil

The test was repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.1.

Table 5.1. Peak and Residual Shear Stress values for pure clay soil

Normal load (kPa)	Peak shear stress (kPa)	Residual shear stress (kPa)
50	84	70
100	116	97
150	136	121

The volumetric change during the shear within the soil specimen can be best revealed by plotting vertical displacement versus horizontal displacement recorded during the test. The plot is given in Figure 5.2. As shown in the figure, for each loading case the soil is first subjected to slight consolidation. For the highest loading case the consolidation goes further while for the others the soil shows slight expansion up to 0.73 mm.

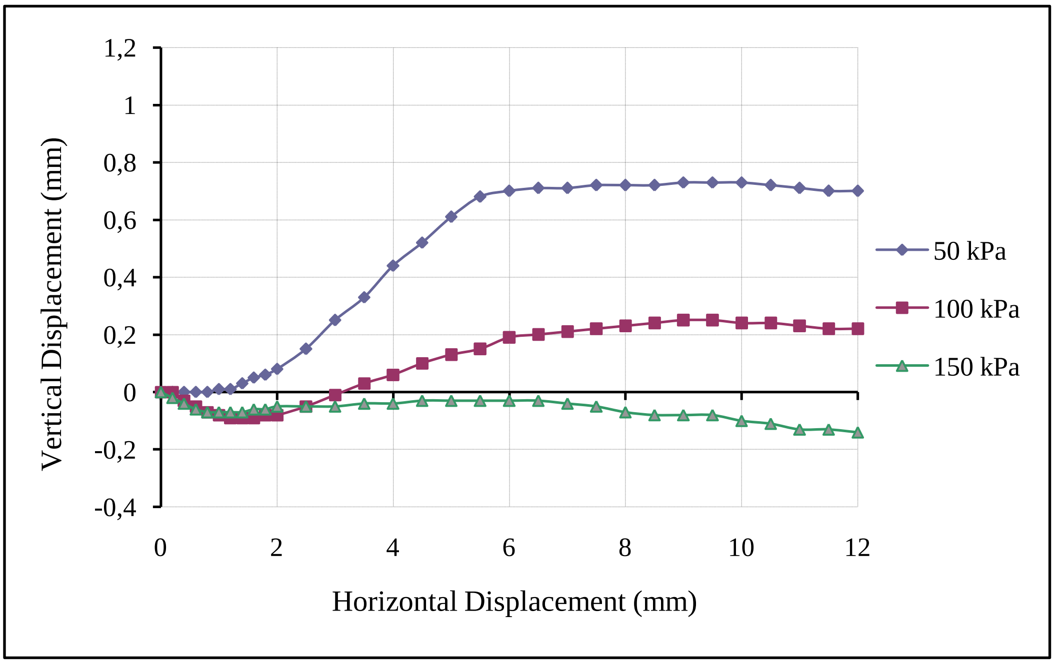

Figure 5.2. Volumetric change obtained for clay soil at water content of 18.8%

The shear strength parameters, therefore, for this given clay specimen can be revealed by plotting shear stress values versus that of normal stress as given in Figure 5.3. The results were tabulated as given in Table 5.2.

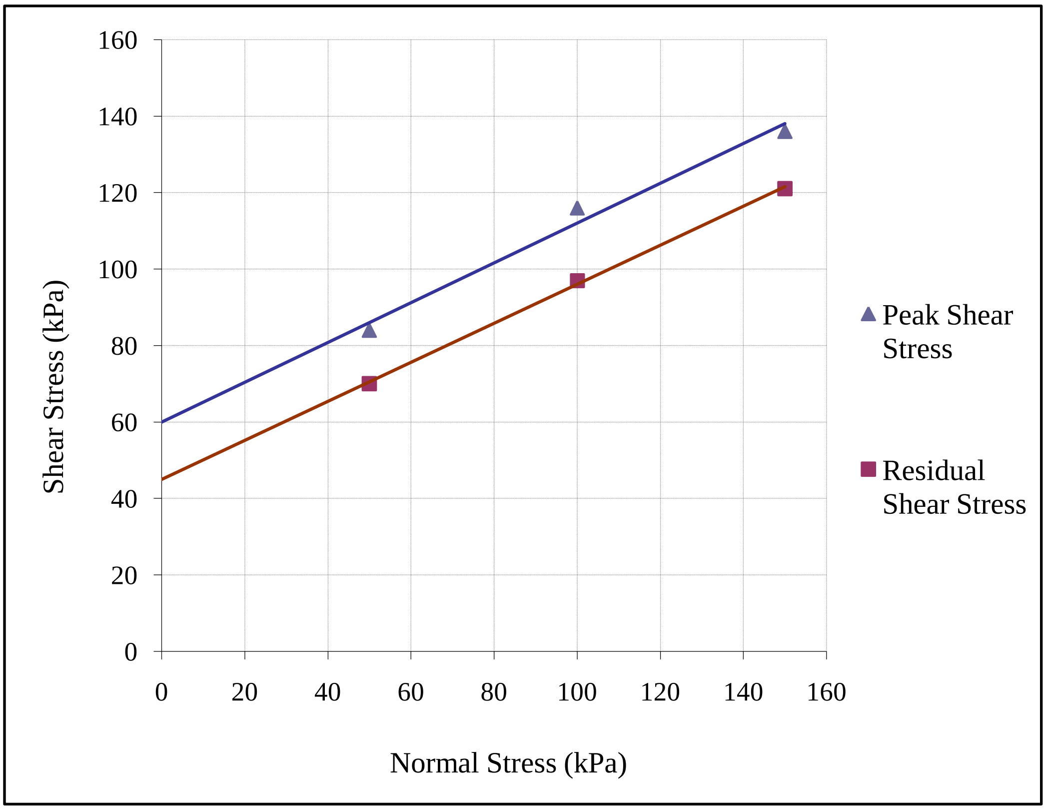

Figure 5.3. Shear strength parameters for pure clay soil

Residual and peak shear stress increases with increasing normal stress shown in the figure. Residual values are less then peak values for the pure clay.

Table 5.2. Shear strength parameters for pure clay soil

Shear Strength Parameter	Peak Values	Residual Values
c, kPa	60	26
ϕ, °	27.5	27

As shown above the internal friction angle for clay soil decreased for the residual case whereas the cohesion values decreased.

5.2. Shear Strength Parameters for Sands

The aim of performing these tests on loose and dense sand was comparing dense and loose sand shear strength properties.

5.2.1. Shear Strength Parameters for Loose Sand

The test was performed on sand soil of which the physical properties were given previously. Oven dried sand was poured into the shear box without any compaction effort. Interpretation of results will be given here in this section. Firstly, the results were plotted as shear stress versus strain. A typical curve for such a clay soil was obtained as given in Figure 5.4. As shown below the figure, shear stress increases with increasing strain until peak point of the shear stress after this point shear stress have no more chance. In addition, shear stress increases with increasing normal load so that shear stress reached 101 kPa for the highest loading shown in the figure for the loose pure sand.

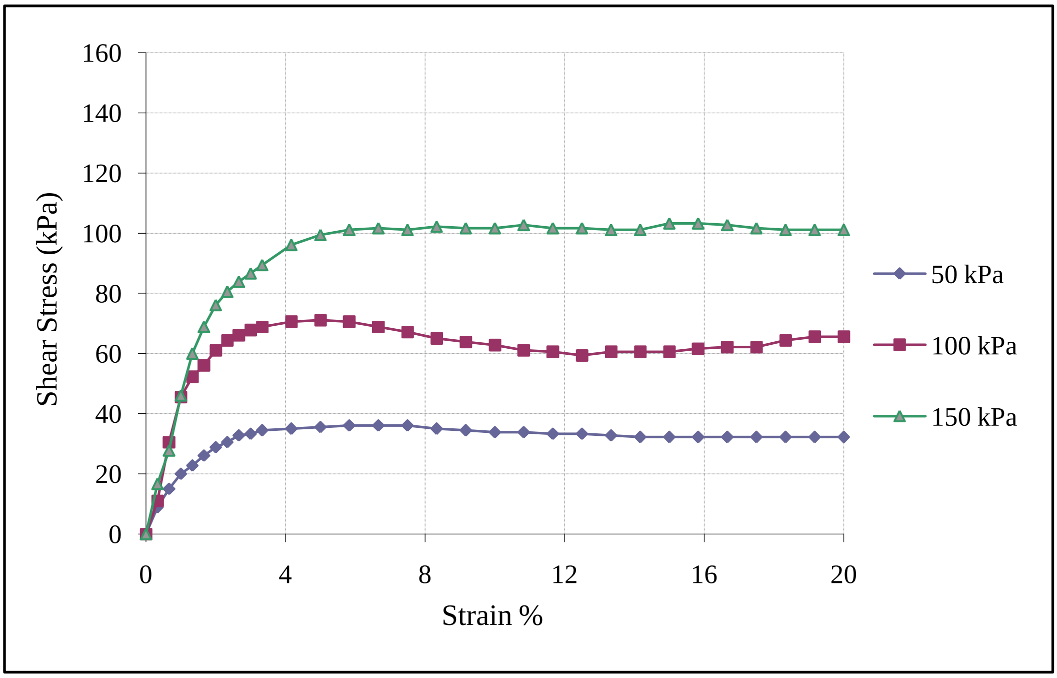

Figure 5.4. Plot of shear stress vs. strain for loose sand

The test was repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.3.

Table 5.3. Peak and Residual Shear Stress values for loose sand

Normal load (kPa)	Peak shear stress (kPa)	Residual shear stress (kPa)
50	36	32
100	70	62
150	101	101

As it can be seen from the table there is no much difference between peak and residual shear stress values for the loose sand. The soil is assumed to deform continuously under a constant stress at a constant volume. In fact, all soils under shearing affect tend to deform under constant volume. Reorientation of soil particles such that the soil particles can roll and translate over each other is the result of this tendency. Reorientation of particles for loose sand is easier since the particles are more or less free to move. Therefore, no obvious distinction between peak and residual shear strength values is expected for loose sands.

The volumetric change plot can be regarded as a supporting view for the given statements above. The test results were plotted for vertical displacement versus horizontal displacement (Figure 5.5). As shown in the figure, for each loading case the soil is first subjected to slight compaction and later it expands. However, the expansion was 0.4 mm at most for the lightest loading case of 50 MPa. In other words the soil might be assumed to be sheared at a constant volume.

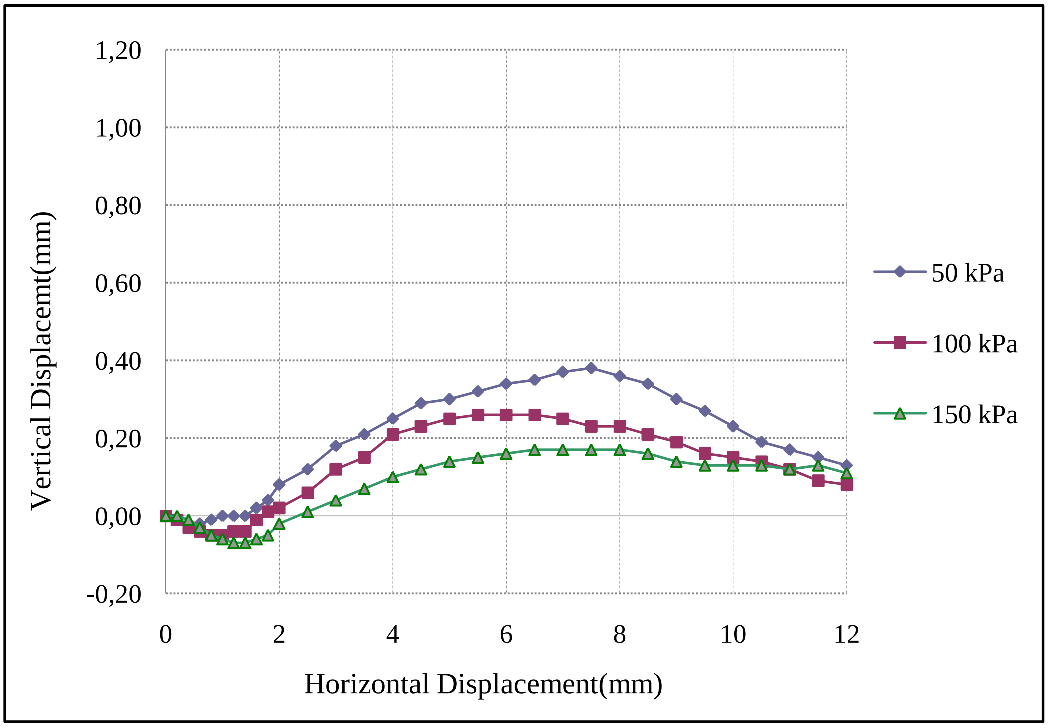

Figure 5.5. Volumetric change obtained for loose sand

The shear strength parameters, therefore, can be revealed from the plot for shear stress versus normal stress values as shown in Figure 5.6. The tabulated results are given in Table 5.4.

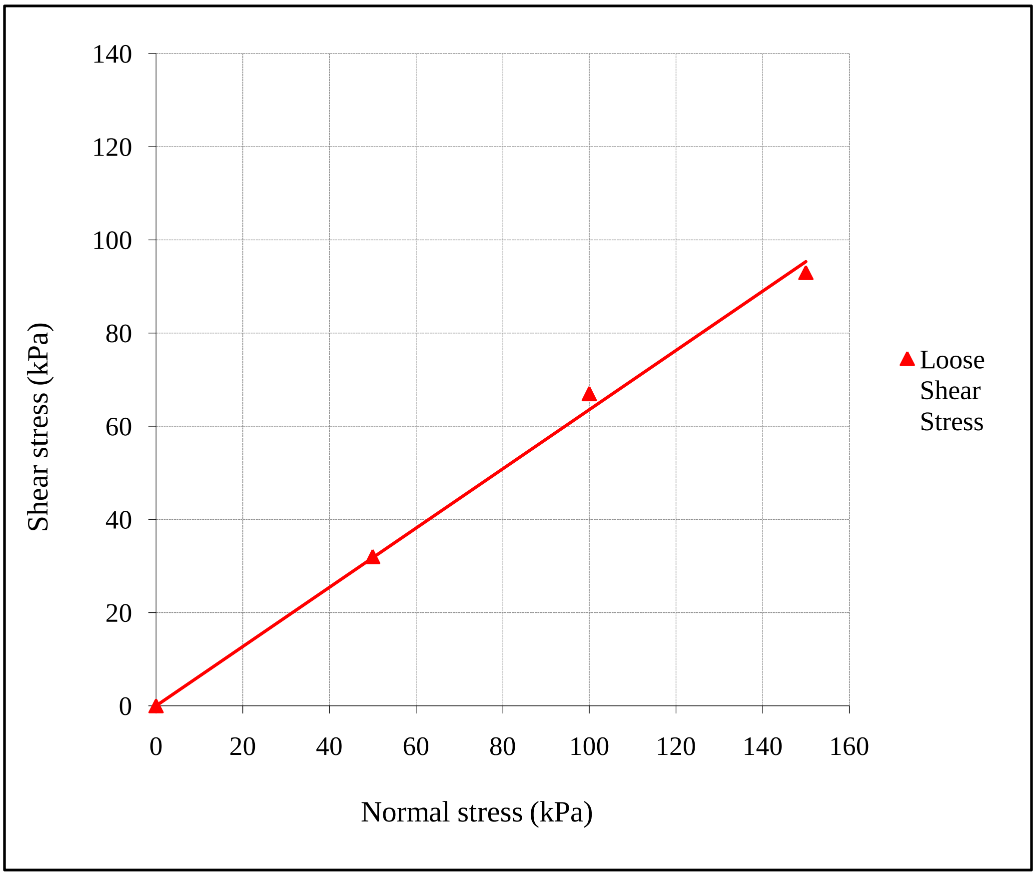

Figure 5.6. Shear strength parameters for loose sand

Table 5.4. Shear strength parameters for loose sand

Shear Strength Parameter	Peak Values	Residual Values
c, kPa	0	0
ϕ, °	33	33

As shown above the shear strength parameters are given in terms of effective stress parameters. Therefore, the cohesion intercept was taken to be zero. Plotting the best fitted line through scattered values, the internal friction angle for this loose sand specimen was calculated to be 33°. As the volumetric change during shearing was not high, there was no remarkable difference recorded for peak and residual values of shear strength parameters.

5.2.2. Shear Strength Parameters for Dense Sand

Oven dried sand was poured into the shear box layer by layer. Each layer was compacted by the aid of a high frequency vibratory hand compactor. Vibration was applied for 30 seconds. This was required to simulate the actual test conditions to be applied for the composite soil test cases. Interpretation of results will be given here in this section. Firstly, the results were plotted as shear stress versus strain. A typical curve for such a clay soil was obtained as given in Figure 5.7. As it can be below the figure, shear stress for the dense sand increases steeply with increasing strain until peak point of the shear stress after this point shear stress decreases and the decrease stops and continue constant values of shear stresses for the each loading. The decrement of shear stress is nearly same the shear stress for the loose sand. In addition, shear stress increases with increasing normal load so that shear stress reached 101 kPa at peak point for the highest loading whereas shear stress decreases until 93 kPa for the highest loading shown in the figure for the dense sand.

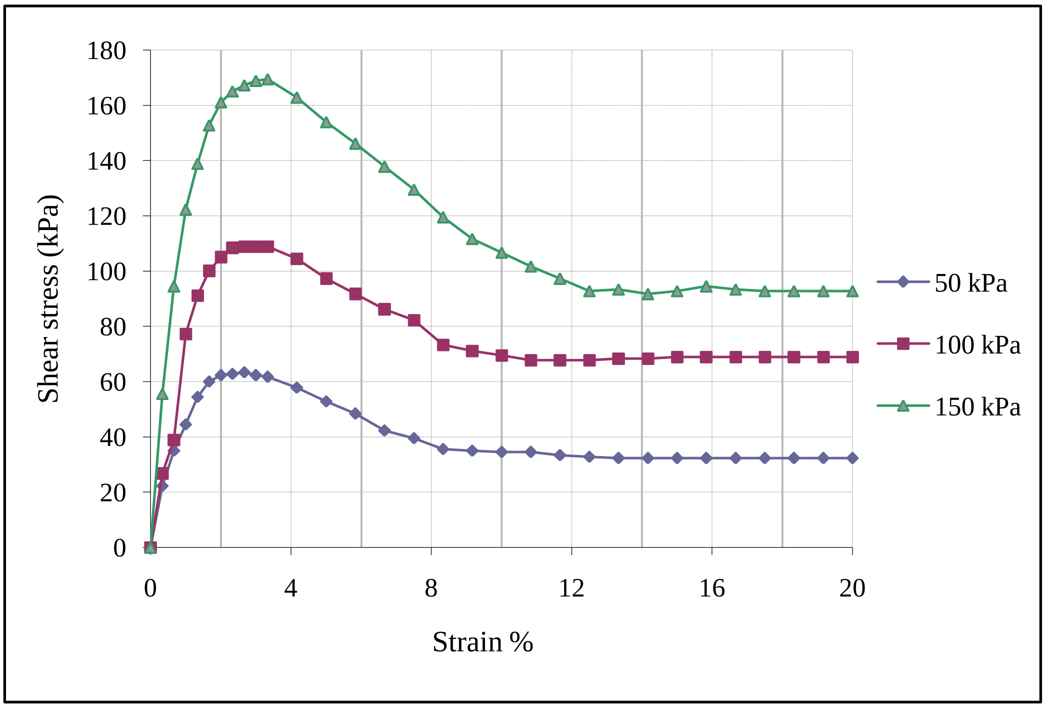

Figure 5.7. Plot of shear stress vs. strain for dense sand

The test was repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.5.

Table 5.5 Peak and residual shear stress values for dense sand

Normal load (kPa)	Peak shear stress (kPa)	Residual shear stress (kPa)
50	63	32
100	109	67
150	170	93

As it can be seen from the table there is some remarkable difference between peak and residual shear stress values for the dense sand.

The volumetric change plot can be regarded as a supporting view for the given statements above. The test results were plotted for vertical displacement versus horizontal displacement (Figure 5.8). As shown in the figure, for each loading case the soil expands remarkably. The magnitude reaches up to 1.2 mm. It worths to consider that the difference in loading magnitude resulted in no considerable change in magnitude of expansion. This clearly supports the idea that the soil particles try to override each other which requires great deal of energy compared to the normal stresses applied. Once it is achieved the soil continues to deform under constant volume.

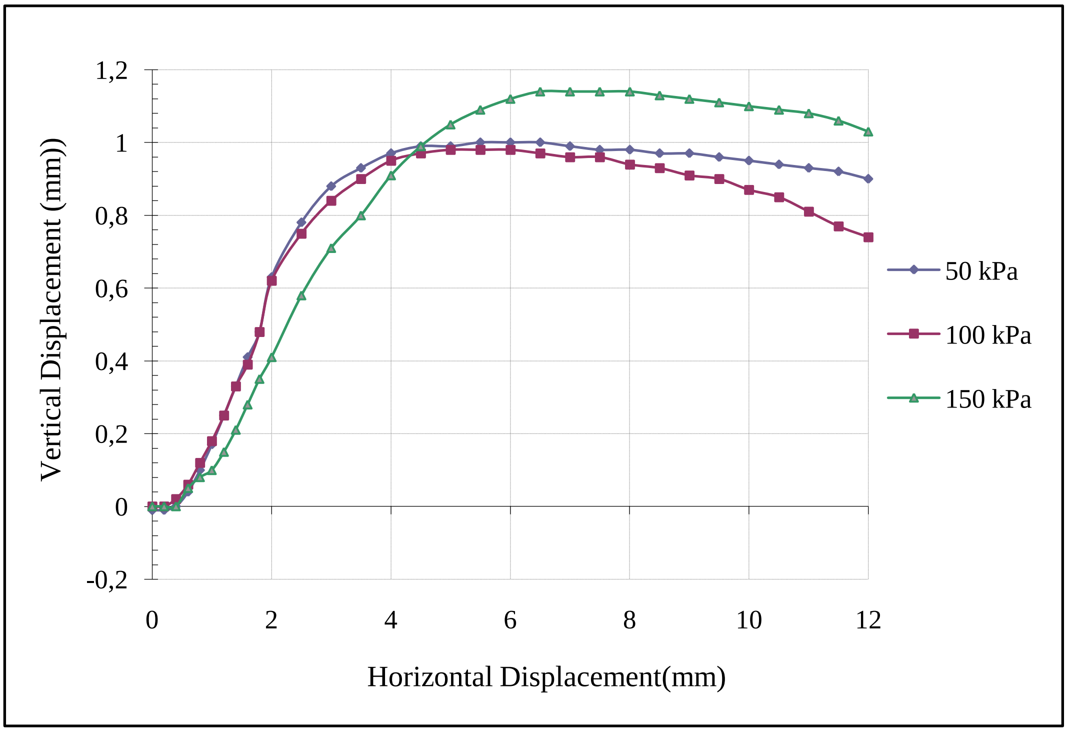

Figure 5.8. Volumetric change obtained for dense sand

The shear strength parameters, therefore, can be revealed from the plot for shear stress versus normal stress values as shown in Figure 5.9. The tabulated results are given in Table 5.6.

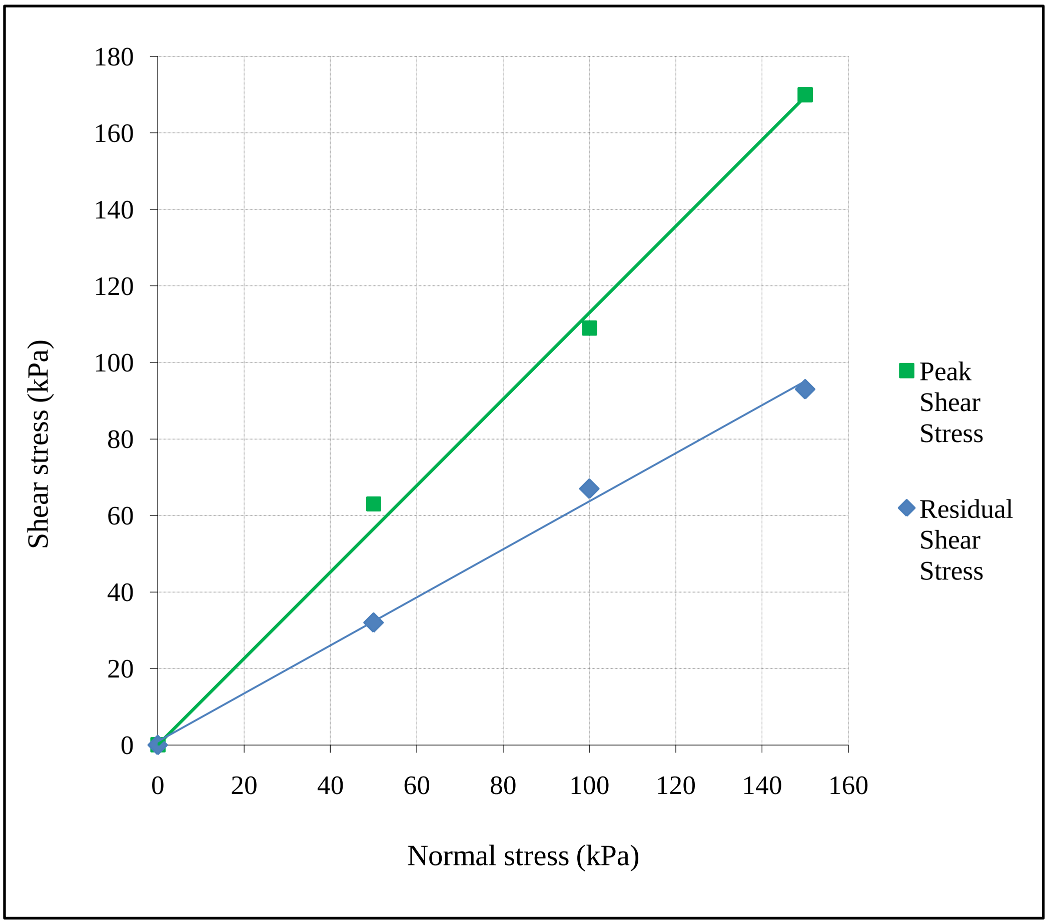

Figure 5.9. Shear strength parameters for dense sand

Table 5.6. Shear strength parameters for dense soil

Shear Strength Parameter	Peak Values	Residual Values
c, kPa	0	0
ϕ, °	48.4	32.4

As shown above the shear strength parameters are given in terms of effective stress parameters. Therefore, the cohesion intercept was taken to be zero. Plotting the best fitted line through the scattered values, the internal friction angle for this dense sand specimen was calculated to be 48°.

5.3. Shear Strength Parameters for Clay with Sand Columns

The test schedule with its details was given in the relevant section and it was tried to be illustrated as given in Figure 4.4. Accordingly, the tests were set up to investigate the effect of sand columns on the shear strength parameters of clay soils. The test was repeated for different sized and number of sand columns to also reveal the size effect within the same setup.

In this set up the main approach was to keep the total sand columns cross-sectional area, within the total surface of the shear box, constant while interchanging between the diameter and the number of the sand columns. The total cross-sectional areas for the sand columns, therefore, were expressed as the ratio of total sand area over the total surface area being tested. This ratio was planned to be 11, 22, and 33% for three different test setups. Each will be achieved by the use of different size and number of sand columns.

Firstly, sand columns with 10 mm of the diameter were tested. 10% of sand area was achieved by the use of 5 sand columns. Number of sand columns was later increased to 10 and 15 at different stages to make sure that 22% and 33% of sandy area respectively is achieved. The same set up was repeated for sand columns with 16 mm diameter. 2, 4 and 6 sand columns were used to achieve 11%, 22% and 33% sand area respectively.

The test was repeated for two sand columns with16mm of diameter, 27mm of diameter and only one sand column with 37 mm of diameter at same water content of 18.8%. In each case the total sandy area was equal to 33%. Later tests were performed to make comparison with the previous tests of 33% of sand, to investigate the scale effect of the sand columns in the clay matrix.

Direct shear tests for different proportion and diameter of sand columns were performed on sand-clay mixtures and then after shearing of the results of sand clay mixtures tests specimens' shapes chanced and its some photos were given in Figure 5.10.

Figure 5.10. The result of direct shear tests of sand-clay mixtures sample photos

5.3.1. Clay Soil Reinforced with Sand Columns of 10mm

The tests were performed in compacted clay soil with 10mm diameters of compacted sand column. Firstly, clay was compacted at water content of 18.8%. It was later extracted from the mould to be tested in the shear box. The holes were opened into clay in shear box to be formed sand columns in clay layer. Number of 5, 10, and 15 holes were opened vertically in clay layer which were used to achieve 11%, 22%, and 33% surface respectively. The prepared dry sand were poured the holes and compacted by the aid of a high frequency vibratory hand compactor. Vibration was applied for 30 seconds. Interpretation of results will be given here in this section. Results were plotted as shear stress versus strain. A typical stress strain curve for each clay sand mixture of specimen was obtained as given in Figure 5.11, Figure 5.12, and Figure 5.13. As it can be below the figure, shear stress for sand-clay mixtures increases with increasing strain until peak point of the shear stress after this point shear stress slightly decreases for the each loading and all sand content. In addition, shear stress increases with increasing normal load, however, shear stress increasing with increasing sand content.

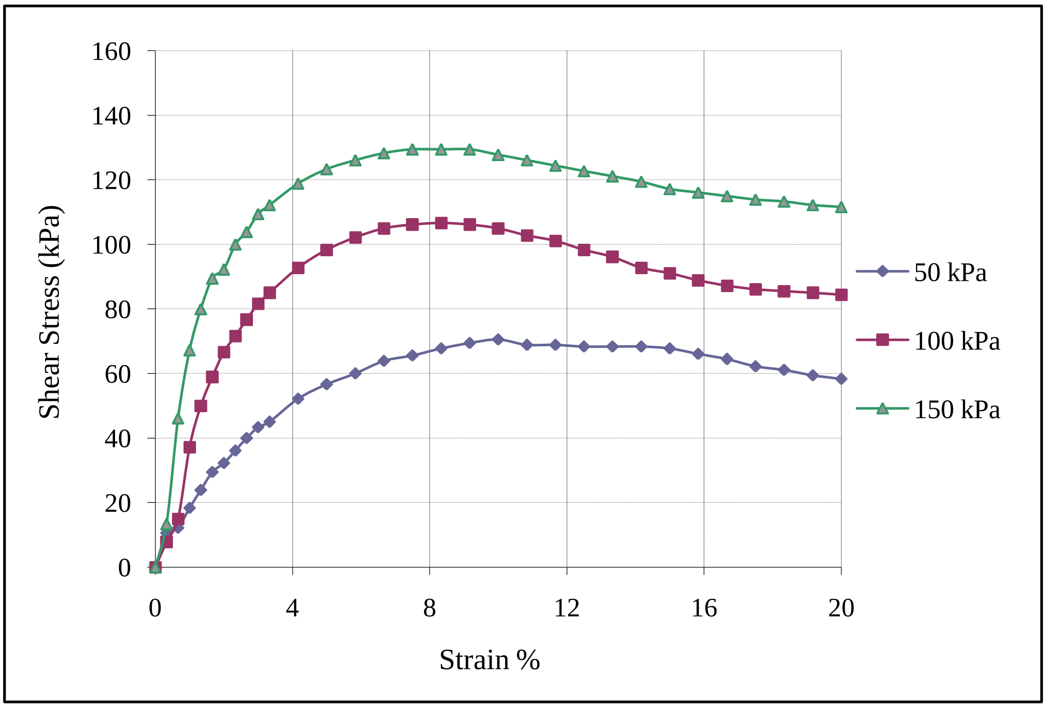

Figure 5.11. Shear stress – strain curve for 11% sand content of sand clay mixture

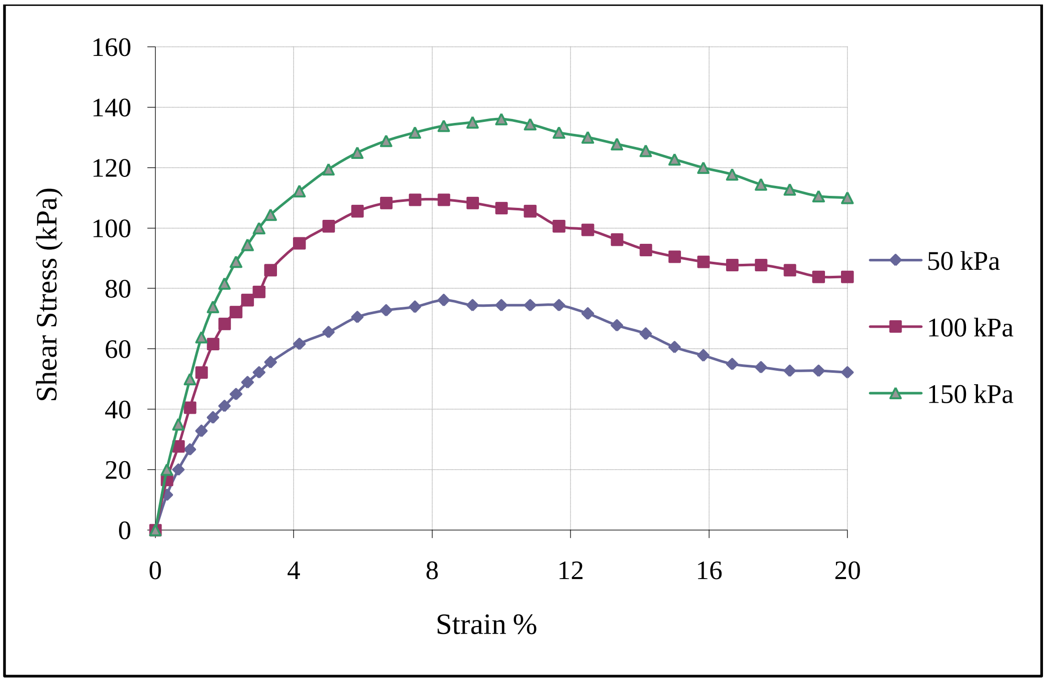

Figure 5.12. Shear stress – strain curve for 22% sand content of sand clay mixture

Shear stress for 10 mm 5 sand columns in sand-clay mixture increases with increasing strain up to peak point but after that point, the stress slightly decrease, however, shear stress of the direct sheer tests for the sand-clay mixtures increases with increasing normal load so that the stress reached at 129 kPa for the highest loading shown in Figure 5.11.

According to direct shear tests for 10 mm 10 sand columns in sand-clay mixture, shear stress increases with increasing strain until peak stress and then decrease slightly decrease. In addition, the stress increases with increasing normal load and reached up to 136 kPa for highest load shown in Figure 5.12.

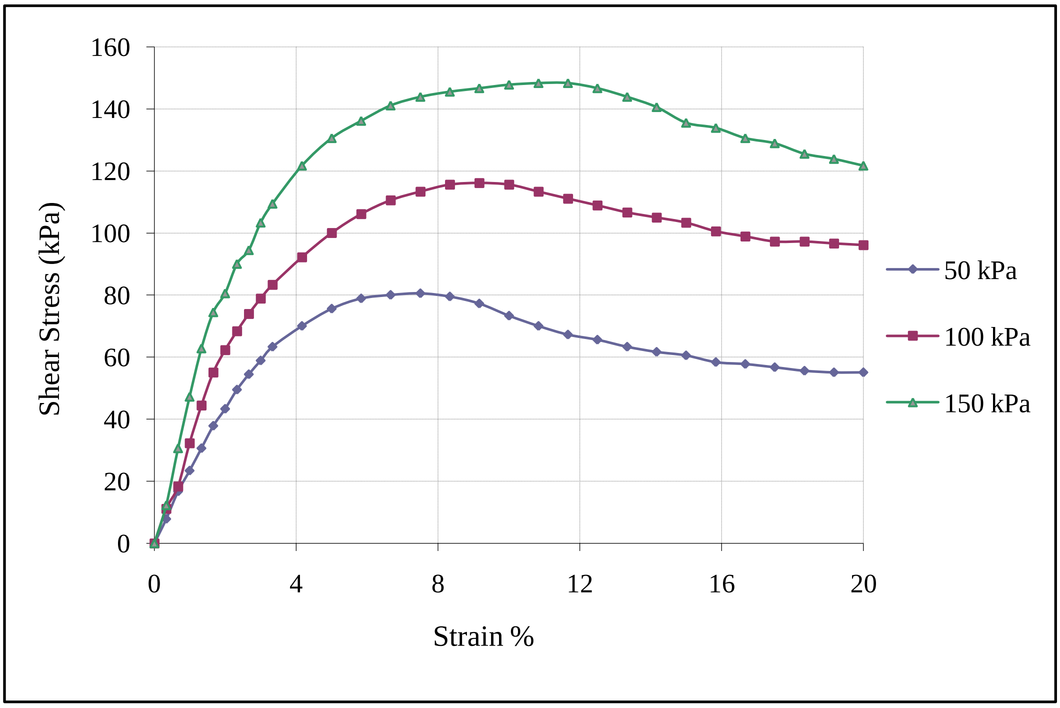

Figure 5.13. Shear stress – strain curve for 33% sand content of sand clay mixture

As it can be in Figure 5.13, Shear stress increases with increasing until peak point but after that, the stress slightly decrease, however, the stress increases with increasing normal load so that the stress reached at 148 kPa for the highest loading for the direct shear tests for 10 mm 15 sand columns in sand-clay mixture.

The tests were repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.7.

Table 5.7 Peak and Residual Shear Stress values for clay sand mixture

Normal Load (kPa)	Peak Shear Stress (kPa)	Residual Shear Stress (kPa)	Sand Sample Area, %
50	71	58	11
100	107	84	11
150	129	112	11
50	76	52	22
100	109	84	22
150	136	110	22
50	81	55	33
100	116	96	33
150	148	122	33

The volumetric change within the soil specimen can be best revealed by plotting vertical displacement versus horizontal displacement recorded during the test. The plot is given in Figure 5.14, Figure 5.15, and Figure 5.16. According to the direct shear tests, volume of the specimen decrease slightly at first and then volume of the sand-clay mixture increase during shearing and decreasing normal load.

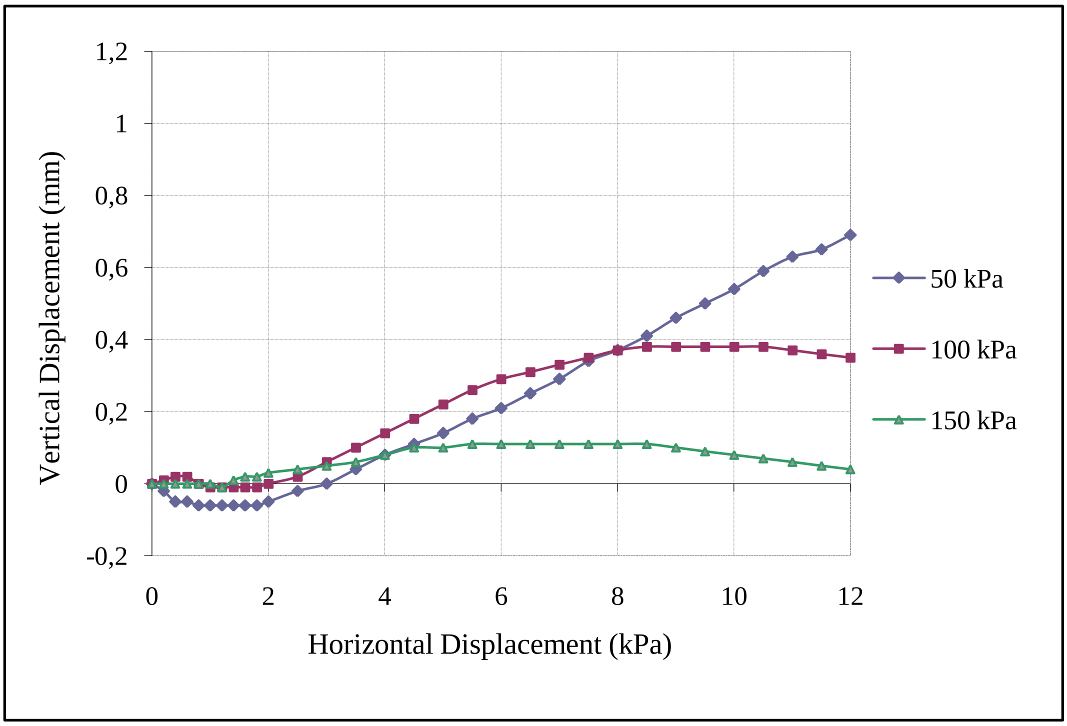

Figure 5.14. Volumetric change obtained for 11% sand content of sand-clay mixture

As shown in Figure 5.14, for each loading case the soil is first subjected to slight consolidation but after that volume of sand-clay mixture for 10 mm 5 sand columns increases up to 0.7 mm with increasing horizontal displacement. The increment of volume behaves inverse proportion with normal loading case.

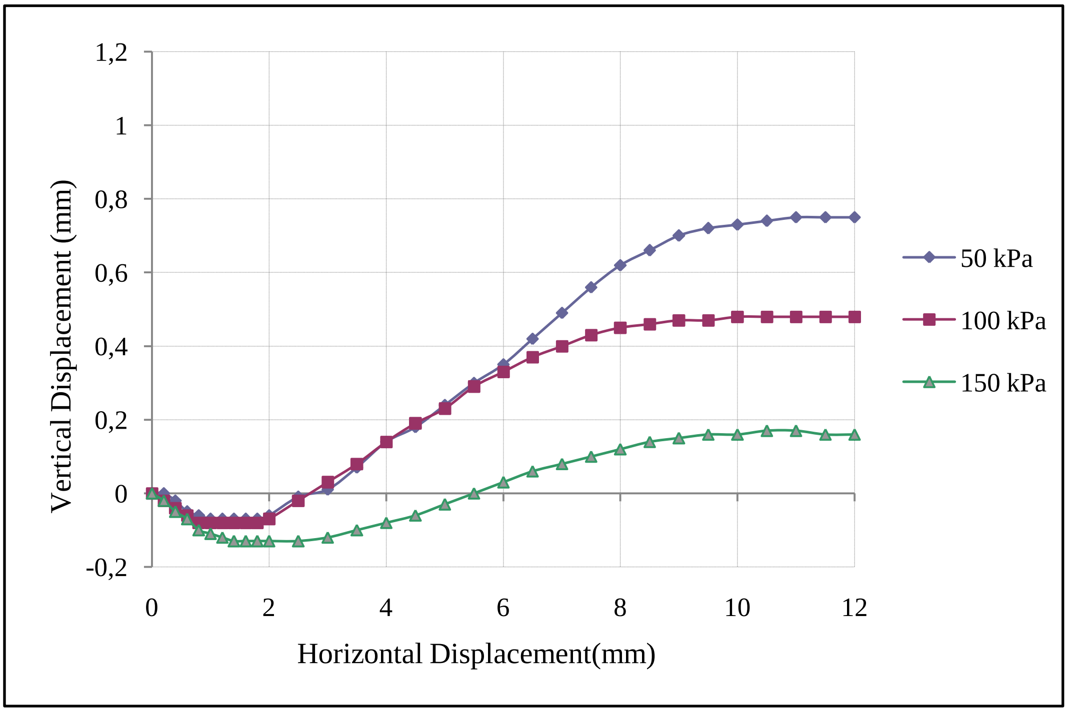

Figure 5.15. Volumetric change obtained for 22% sand content of sand-clay mixture

The volumes of 22% sand content of sand-clay mixtures decrease slightly at first subsequently, the volume increase during shear. In addition, volume of sand-clay mixture with 10 mm 10 sand columns increases with decreasing loading which was reached up to 0.78 mm for the lowest loading shown in Figure 5.15.

As it can be in Figure 5.16, the volumes of 10 mm 15 sand columns (33% sand content) decreases slightly at first after that, the volume increase during shear. In addition, volume of 33% sand content of sand-clay mixtures increases with decreasing loading which was reached up to 1.1 mm for the lowest loading shown in Figure 5.16

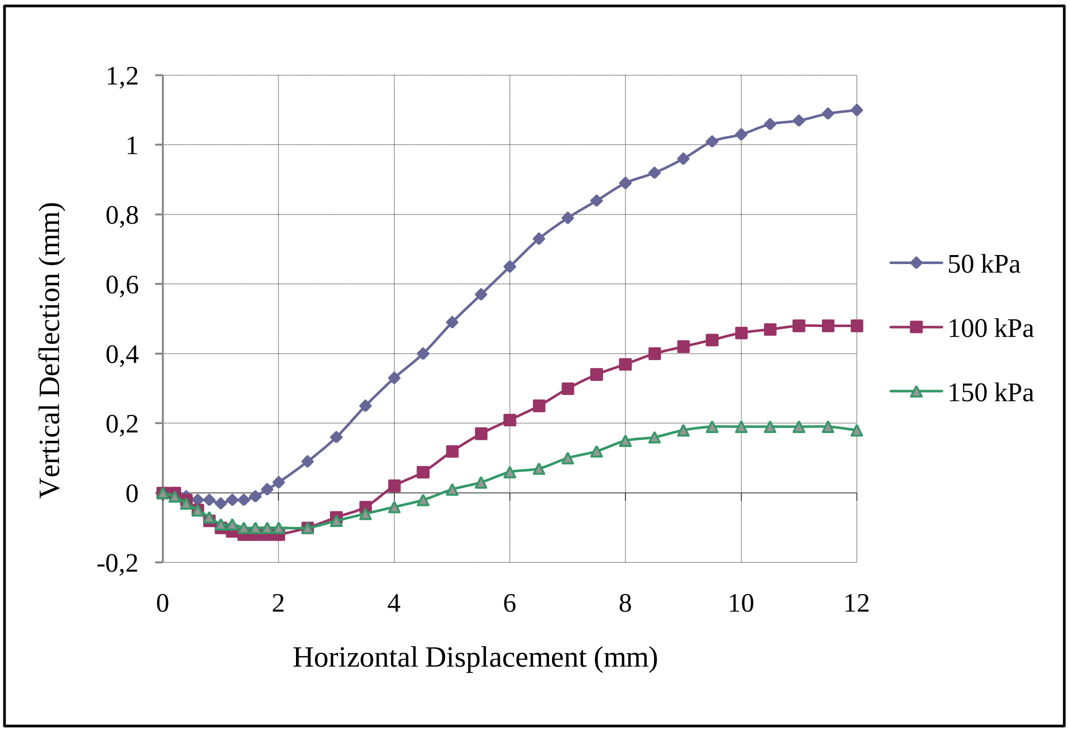

Figure 5.16. Volumetric change obtained for 33% sand content of sand-clay mixture

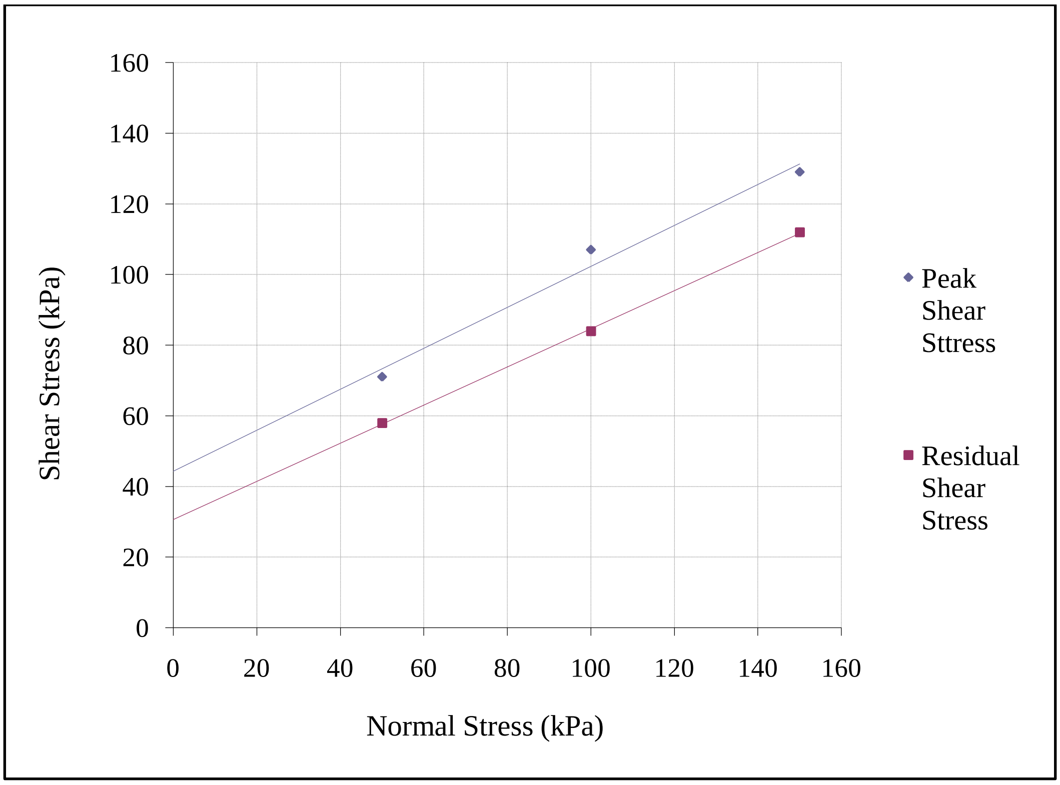

Figure 5.17. Shear strength parameters for 11% sand content of sand-clay mixture

The shear strength parameters, therefore, for this given sand-clay mixtures specimens can be revealed by plotting shear stress values versus that of normal stress as given in Figure 5.17, Figure 5.18, and Figure 5.19. The results were tabulated as given in Table 5.8. As shown in the figures, shear stress for 10 mm sand columns in sand-clay mixtures increases with increasing normal stress.

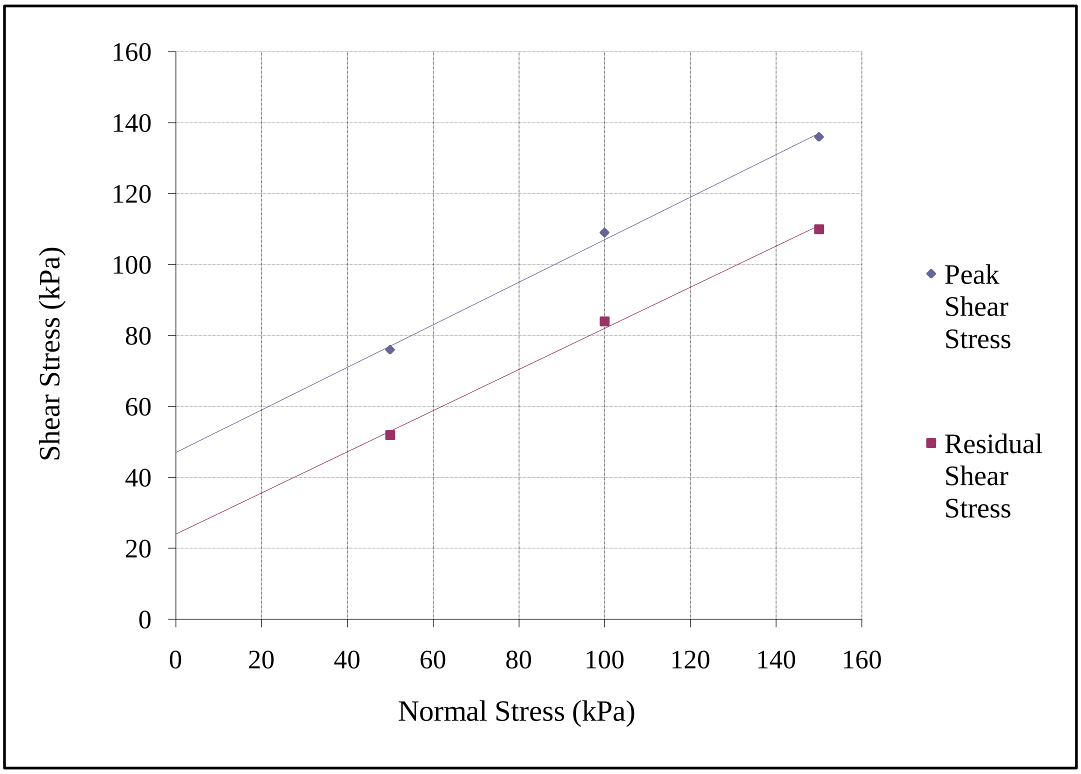

Figure 5.18. Shear strength parameters for 22% sand content of sand-clay mixture

As shown in Figure 5.17, peak and residual shear stress for 10 mm 5 sand columns in sand-clay mixtures increase with increasing normal stress. In addition, shear strength parameters internal friction angle and cohesion were calculated respectively 30° and 44 kPa for peak stress.

Shear stresses for 22% sand content of sand-clay mixtures increase with increasing normal stresses shown in Figure 5.18. Shear strength parameters (internal friction angle and cohesion) for 10 mm 10 sand columns in sand-clay mixtures were calculated and given in Table 5.8.

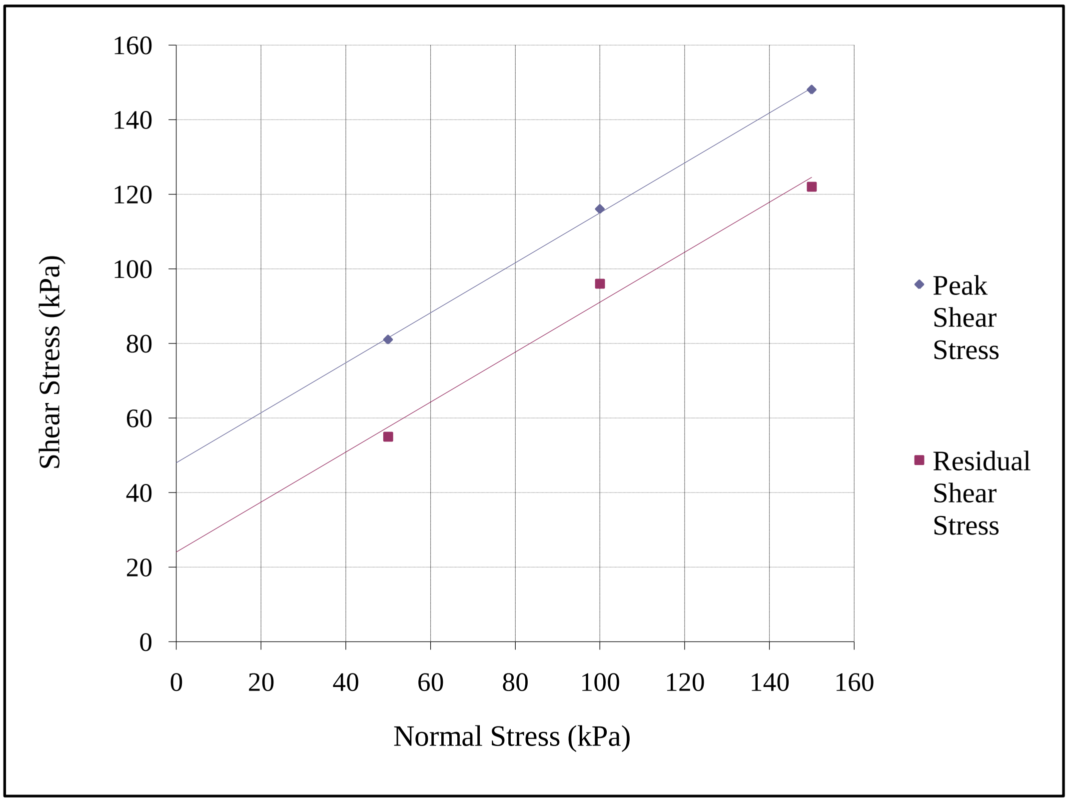

Figure 5.19. Shear strength parameters for 33% sand content of sand-clay mixture

According to direct shear tests for 10 mm 15 sand columns (33% sand content) in sand-clay mixture, shear stress increases with increasing normal stress shown in Figure 5.19. In this result, internal friction angle and cohesion were calculated 34° and 48 kPa respectively.

Table 5.8 Shear strength parameters for sand-clay mixture

Shear Strength Parameters	Peak Values	Residual Values	Sand Content by Sample Area, %
c, kPa	44	31	11
ϕ, °	30	28	11
c, kPa	47	24	22
ϕ, °	31	30	22
c, kPa	48	24	33
ϕ, °	34	34	33

5.3.2. Clay Soil Reinforced with Sand Columns of 16mm

The tests were performed in compacted clay soil with 16 mm diameters of compacted sand column. Firstly, clay was compacted at water content of 18.8%. It was later extracted from the mould to be tested in the shear box. The holes were opened into clay in shear box to be formed sand columns in clay layer. Number of 2, 4, and 6 16mm diameter holes were opened vertically in clay layer which were used to achieve 11%, 22%, and 33% area by sample area respectively. The prepared dry sand were poured the holes and compacted by the aid of a high frequency vibratory hand compactor. Vibration was applied for 30 seconds. Interpretation of results will be given here in this section. Results were plotted as shear stress versus strain. A typical stress strain curve for each clay sand mixture of specimen was obtained as given in Figure 5.20, Figure 5.21, and Figure 5.22. As it can be below the figure, shear stress for sand-clay mixtures with 16 mm sand columns increases with increasing strain until peak point of the shear stress after this point shear stress slightly decreases for the each loading and all sand content. In addition, shear stress increases with increasing normal load, however, shear stress increasing with increasing sand content.

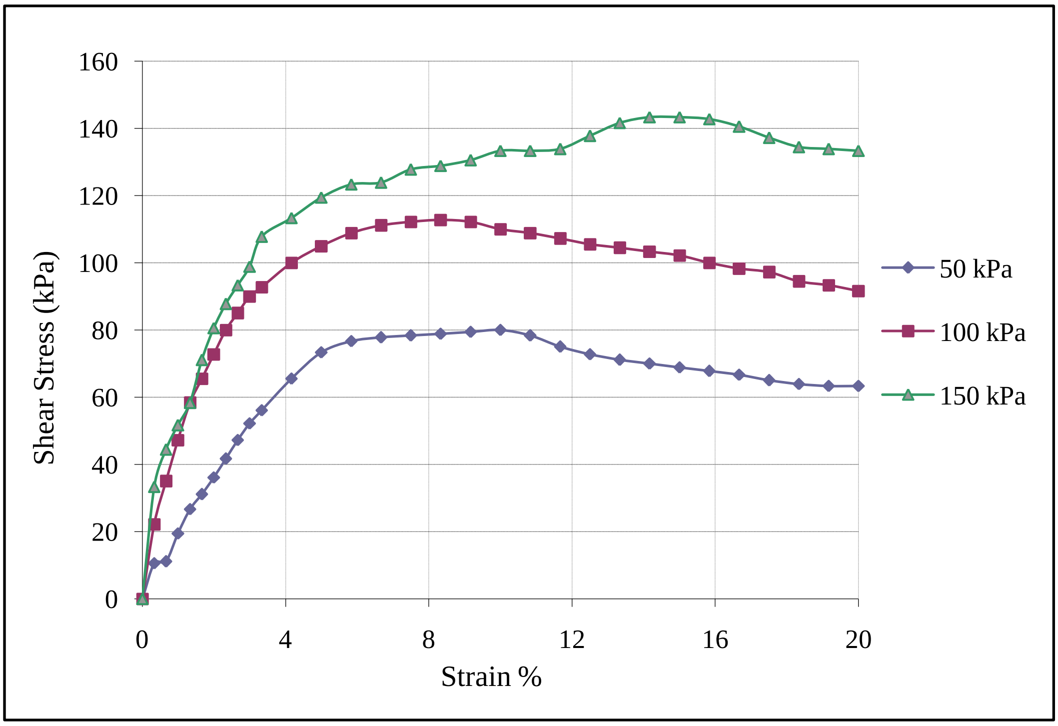

Figure 5.20. Shear stress – strain curve for 11% sand content of sand clay mixture

Shear stress for 16 mm 2 sand columns in sand-clay mixture increases with increasing strain up to peak point but after that point, the stress slightly decrease, however, shear stress of the direct sheer tests for the sand-clay mixtures increases with increasing normal load so that the stress reached at 129 kPa for the highest loading shown in Figure 5.20.

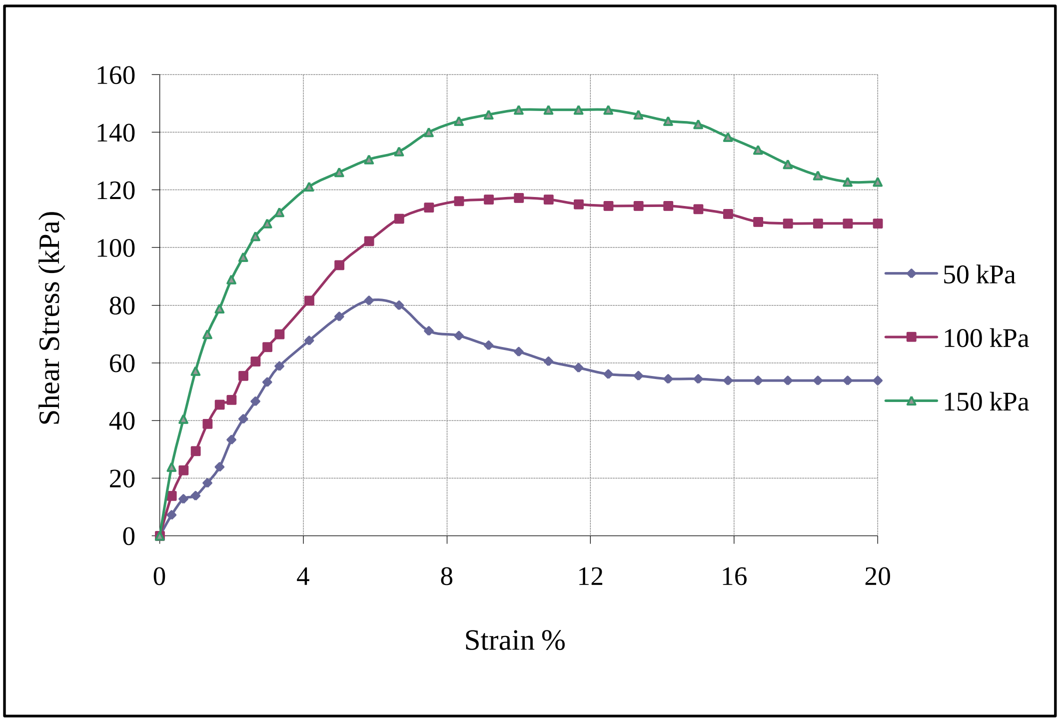

Figure 5.21. Shear stress – strain curve for 22% sand content of sand clay mixture

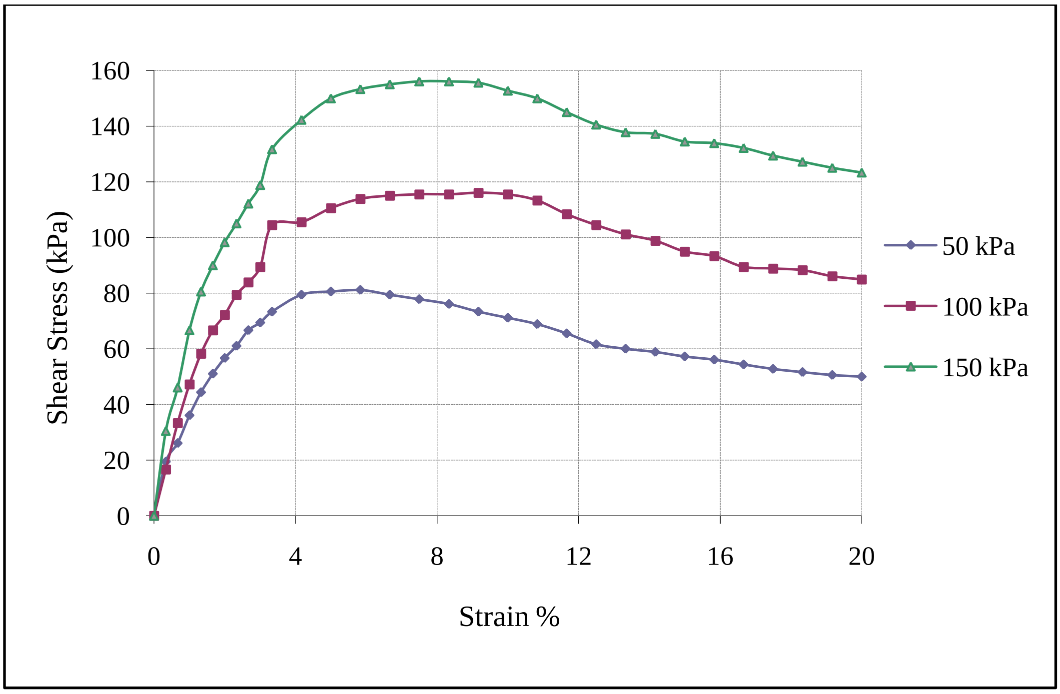

Figure 5.22. Shear stress – strain curve for 33% sand content of sand clay mixture

According to direct shear tests for 16 mm 4 sand columns (22% sand content) in sand-clay mixture, shear stress increases with increasing strain until peak stress and then decrease slightly decrease. In addition, the stress increases with increasing normal load and reached up to 148 kPa for highest load shown in Figure 5.21.

As it can be in Figure 5.22, Shear stress increases with increasing until peak point but after that, the stress slightly decrease, however, the stress increases with increasing normal load so that the stress reached at 156 kPa for the highest loading for the direct shear tests for 16 mm 6 sand columns in sand-clay mixture.

The tests were repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.9.

Table 5.9 Peak and Residual Shear Stress values for clay sand mixture

Normal Load (kPa)	Peak Shear Stress (kPa)	Residual Shear Stress (kPa)	Sand Sample Area, %
50	80	63	11
100	113	92	11
150	143	133	11
50	82	54	22
100	117	108	22
150	148	123	22
50	81	50	33
100	116	85	33
150	156	123	33

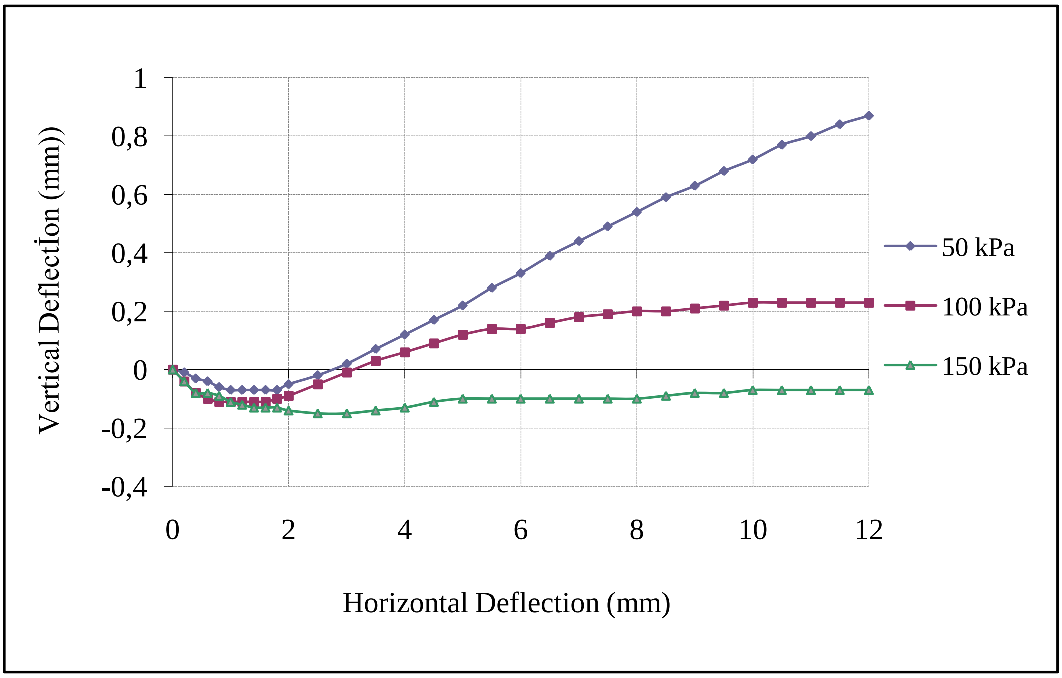

Figure 5.23. Volumetric change obtained for 11% sand content of sand-clay mixture

The volumetric change within the soil specimen can be best revealed by plotting vertical displacement versus horizontal displacement recorded during the test. The plot is given in Figure 5.23, Figure 5.24, and Figure 5.25. According to the direct shear tests for 16 mm sand columns in the mixture, volume of the specimen decrease slightly at first and then volume of the sand-clay mixture increase during shearing and decreasing normal load.

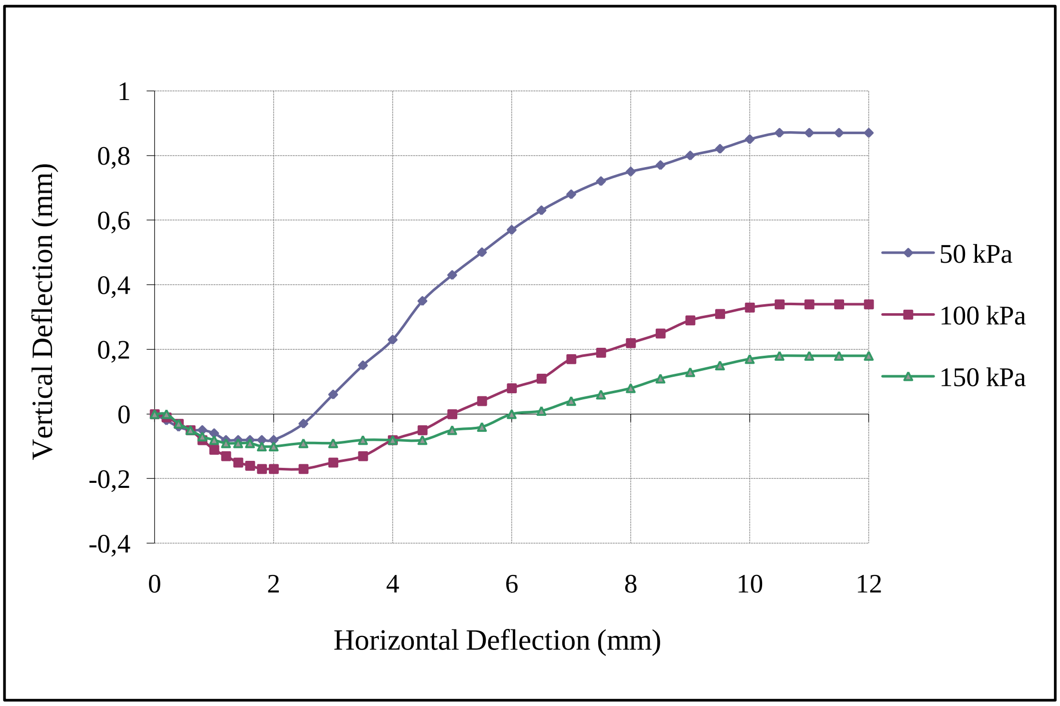

Figure 5.24. Volumetric change obtained for 22% sand content of sand-clay mixture

As shown in Figure 5.23, for each loading case the soil is first subjected to slight consolidation but after that volume of sand-clay mixture for 16 mm 2 sand columns increases up to 0.85 mm with increasing horizontal displacement. The increment of volume behaves inverse proportion with normal loading case.

The volumes of 22% sand content of sand-clay mixtures decrease slightly at first subsequently, the volume increase during shear. In addition, volume of sand-clay mixture with 16 mm 4 sand columns increases with decreasing loading which was reached up to 0.85 mm for the lowest loading shown in Figure 5.24.

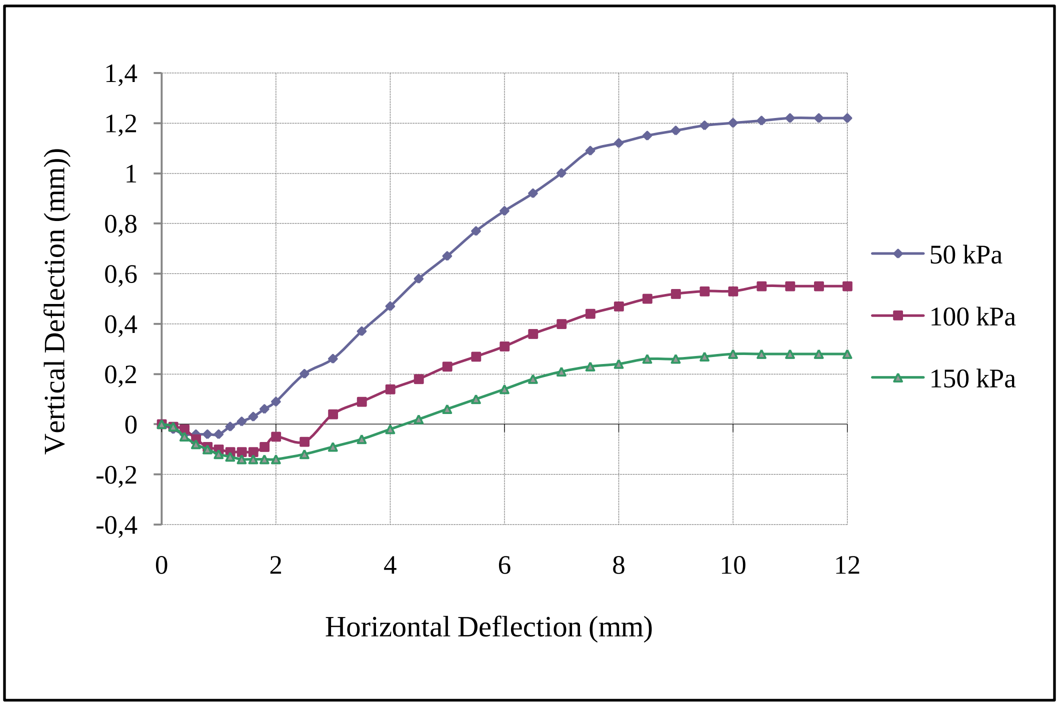

Figure 5.25. Volumetric change obtained for 33% sand content of sand-clay mixture

As it can be in Figure 5.25 the volumes of 16 mm 6 sand columns (33% sand content) decreases slightly at first after that, the volume increase during shear. In addition, volume of 33% sand content of sand-clay mixtures increases with decreasing loading which was reached up to 1.2 mm for the lowest loading.

The shear strength parameters, therefore, for this given mixtures specimens can be revealed by plotting shear stress values versus that of normal stress as given in Figure 5.26, Figure 5.27, and Figure 5.28. The results were tabulated as given in Table 5.10. As shown in the figures, shear stress for 16 mm sand columns in sand-clay mixtures increases with increasing normal stress.

As shown in Figure 5.26, peak and residual shear stress for 16 mm 2 sand columns in sand-clay mixtures increase with increasing normal stress. In addition, shear strength parameters internal friction angle and cohesion were calculated respectively 32° and 49 kPa for peak stress.

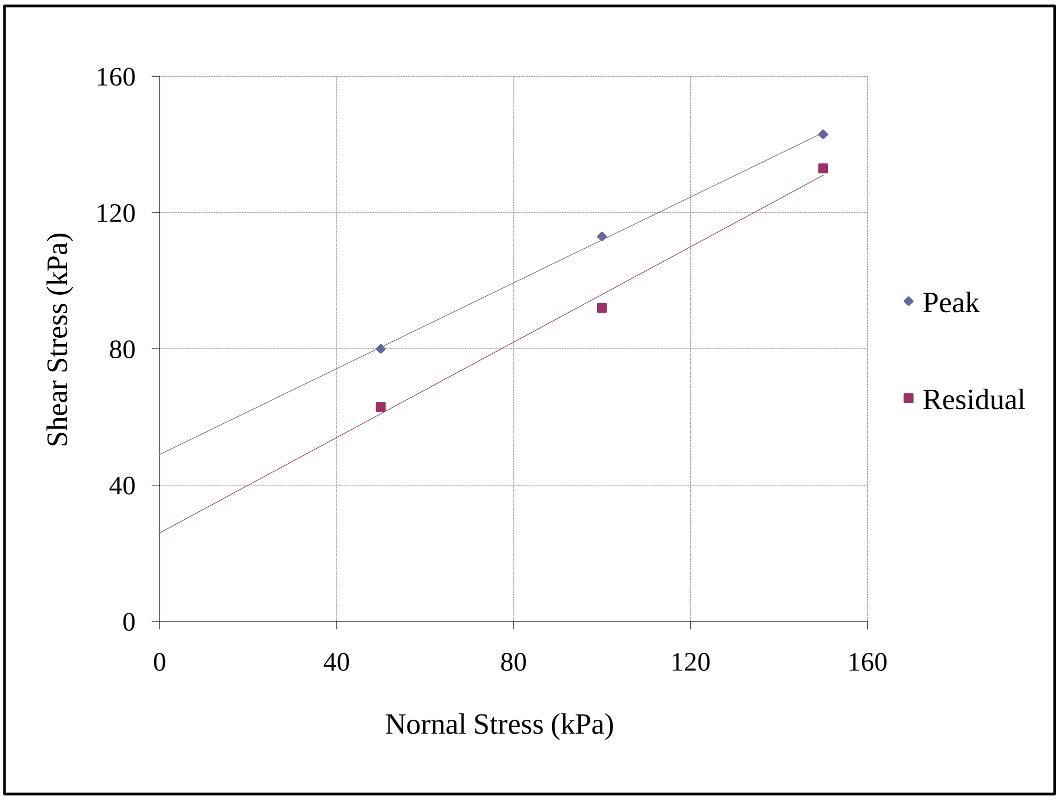

Figure 5.26. Shear strength parameters for 11% sand content of sand-clay mixture

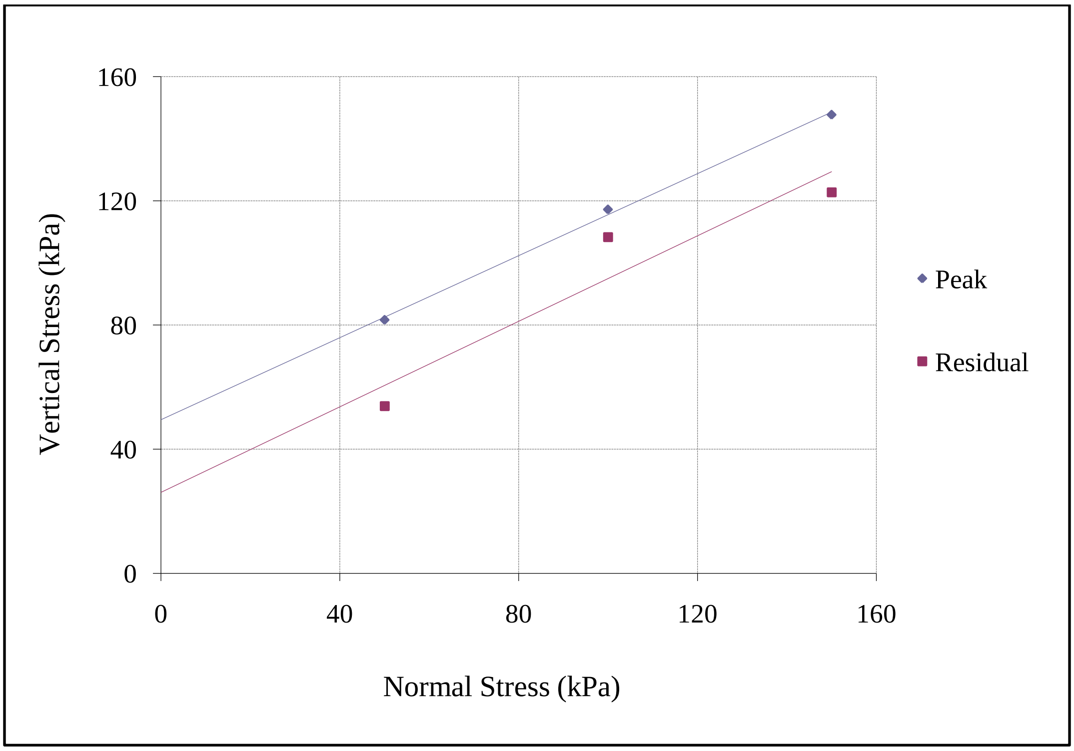

Figure 5.27. Shear strength parameters for 22% sand content of sand-clay mixture

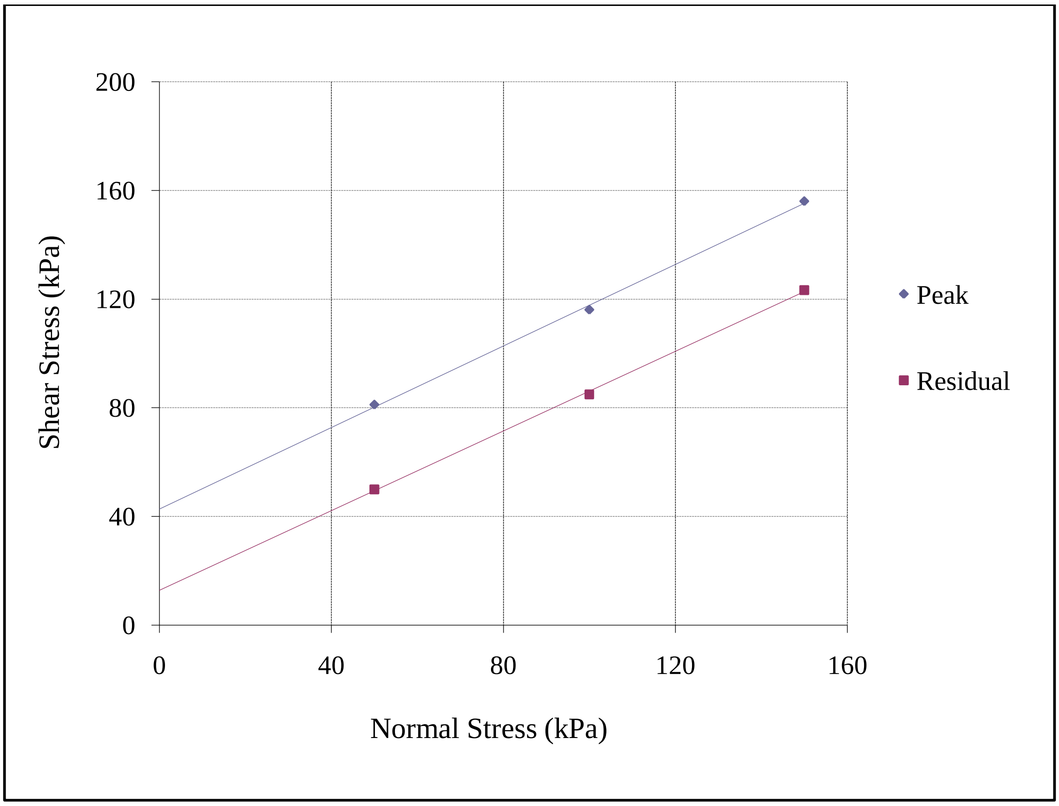

Figure 5.28. Shear strength parameters for 33% sand content of sand-clay mixture

Shear stresses for 22% sand content of sand-clay mixtures increase with increasing normal stresses shown in Figure 5.27. Shear strength parameters (internal friction angle and cohesion) for 16 mm 4 sand columns in sand-clay mixtures were calculated and given in Table 5.10.

According to direct shear tests for 16 mm 6 sand columns (33% sand content) in sand-clay mixture, shear stress increases with increasing normal stress shown in Figure 5.28. In this result, internal friction angle and cohesion were calculated 37° and 43 kPa respectively.

Table 5.10 Shear strength parameters for sand-clay mixture

Shear Strength Parameters	Peak Values	Residual Values	Sand Sample Area, %
c, kPa	49	26	11
ϕ, °	32.2	35	11
c, kPa	50	26	22
ϕ, °	33.4	34.6	22
c, kPa	43	13	33
ϕ, °	36.9	36	33

5.3.3. Clay Soil Reinforced with Sand Columns of 16 mm, 27, and 37 mm Diameter

The tests were performed in compacted clay soil with 16, 27, and 37mm diameters of compacted sand columns. Firstly, clay was compacted at water content of 18.8%. It was later extracted from the mould to be tested in the shear box. The holes were opened into clay in shear box to be formed sand columns in clay layer. Number of 6, 2, and 1 respectively 16, 27, and 37 mm diameter holes were opened vertically in clay layer which were used to achieve only 33% area by sample area. The prepared dry sand were poured into the previously opened holes in the clay and compacted by the aid of a high frequency vibratory hand compactor. Vibration was applied for 30 seconds. Interpretation of results will be given here in this section. Results were plotted as shear stress versus strain. A typical stress strain curve for each clay sand mixture of specimen was obtained as given in Figure 5.29, Figure 5.30 and Figure 5.31. As it can be below the figure, shear stress for sand-clay mixtures increases with increasing strain until peak point of the shear stress after this point shear stress slightly decreases for the each loading and different diameter of sand columns at 33% sand content. In addition, shear stress increases with increasing normal load.

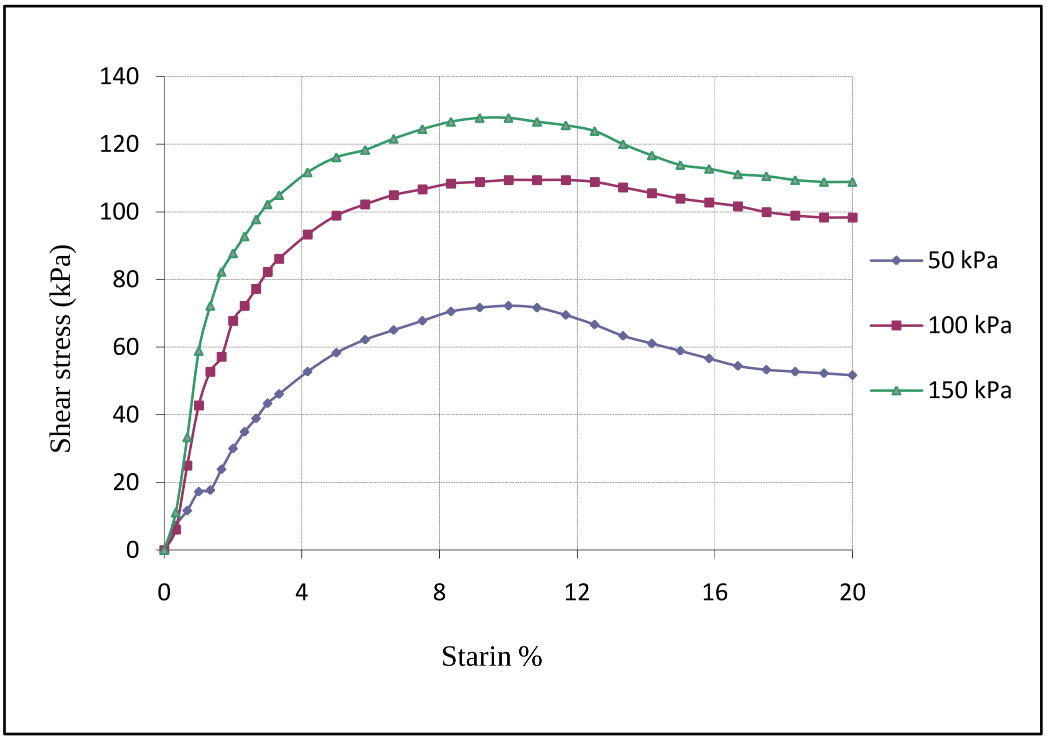

Figure 5.29. Shear stress – strain curve for 16mm sand columns of 33% sand content of sand clay mixture

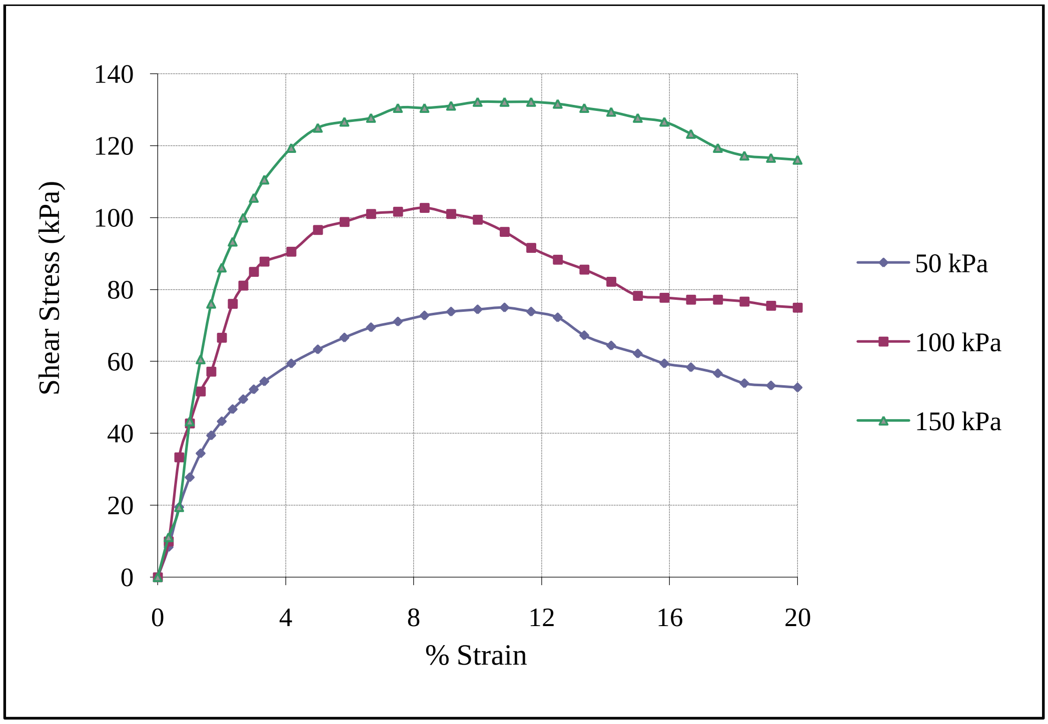

Figure 5.30. Shear stress – strain curve for 27 mm sand columns of 33% sand content of sand clay mixture

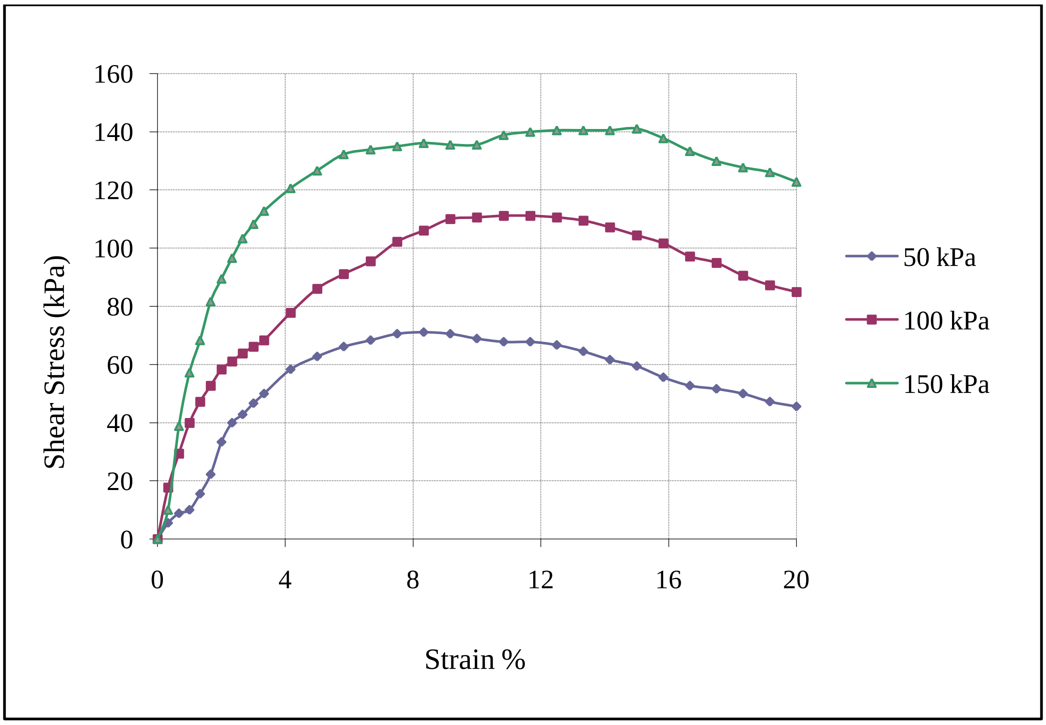

Figure 5.31. Shear stress – strain curve for 37mm sand columns of 33% sand content of sand clay mixture

Shear stress for 16 mm 6 sand columns (33% sand content) in sand-clay mixture increases with increasing strain up to peak point but after that point, the stress slightly decrease, however, shear stress of the direct sheer tests for the sand-clay mixtures increases with increasing normal load so that the stress reached at 128 kPa for the highest loading shown in Figure 5.29.

According to direct shear tests for 27 mm 2 sand columns in sand-clay mixture, shear stress increases with increasing strain until peak stress and then decrease slightly decrease. In addition, the stress increases with increasing normal load and reached up to 136 kPa for highest load shown in Figure 5.30.

As it can be seen in Figure 5.31, Shear stress increases with increasing strain until peak point but after that, the stress slightly decrease, however, the stress increases with increasing normal load so that the stress reached at 141 kPa for the highest loading for the direct shear tests for 37 mm 1 sand column (33% sand content) in sand-clay mixture.

The tests were repeated for three different loading cases namely for 50, 100 and 150 kPa. As the magnitude of the loading increases the shear stress required for given strain increases accordingly. The peak and residual shear stresses reached can be summarized as given in Table 5.11.

Table 5.11 Peak and Residual Shear Stress values for of 33% sand content of sand clay mixture

Loading Magnitude, kPa	Peak Shear Stress, kPa	Residual Shear Stress, kPa	Sand Content by Sample Area, %
50	72	52	16
100	109	98	16
150	128	109	16
50	75	53	27
100	103	75	27
150	132	116	27
50	71	46	39
100	111	85	39
150	141	123	39

The volumetric change within the soil specimen can be best revealed by plotting vertical displacement versus horizontal displacement recorded during the test. The plot is given in Figure 5.32, Figure 5.33, and Figure 5.34. According to the direct shear tests, volume of the specimen decrease slightly at first and then volume of the sand-clay mixture increase during shearing and decreasing normal load.

As shown in Figure 5.32, for each loading case the soil is first subjected to slight consolidation but after that volume of sand-clay mixture for 16 mm 6 sand columns (33% sand content) increases up to 0.75 mm with increasing horizontal displacement. The increment of volume behaves inverse proportion with normal loading case.

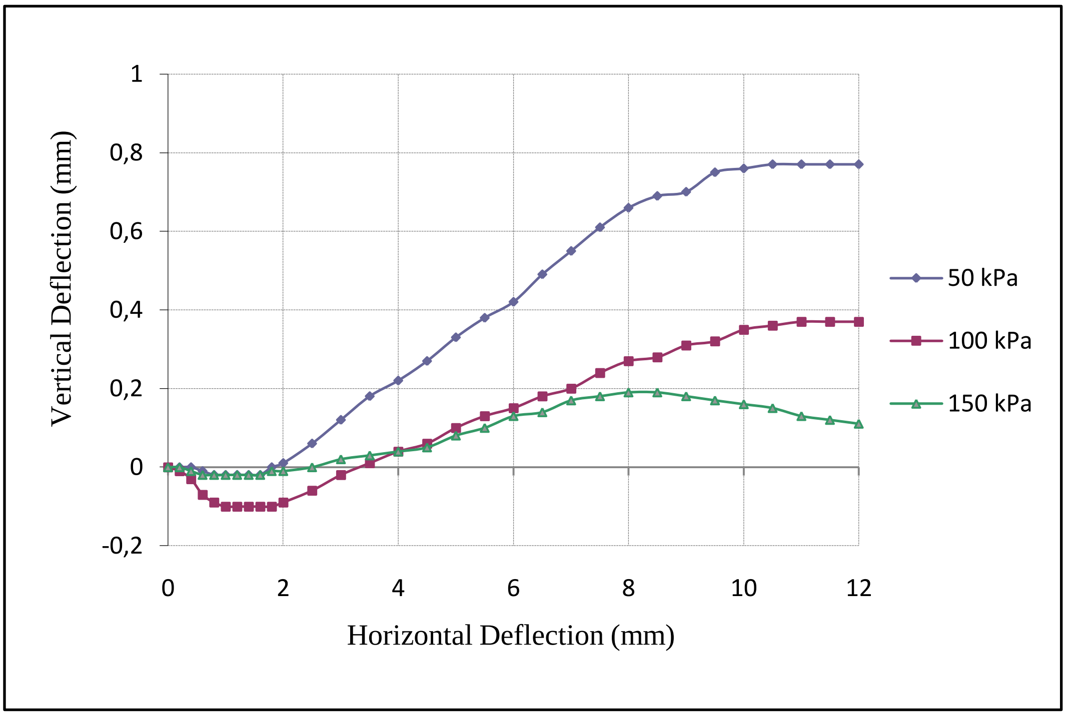

Figure 5.32. Volumetric change obtained for 16mm sand columns of 33% sand content of sand clay mixture

The volumes of 33% sand content of sand-clay mixtures decrease slightly at first subsequently, the volume increase during shear. In addition, volume of sand-clay mixture with 27 mm 2 sand columns increases with decreasing loading which was reached up to 1.05 mm for the lowest loading shown in Figure 5.33.

As it can be seen in Figure 5.34 the volumes of 37 mm 1 sand column (33% sand content) decreases slightly at first after that, the volume increase during shear. In addition, volume of 33% sand content of sand-clay mixtures increases with decreasing loading which was reached up to 1.05 mm for the lowest loading shown in Figure 5.34.

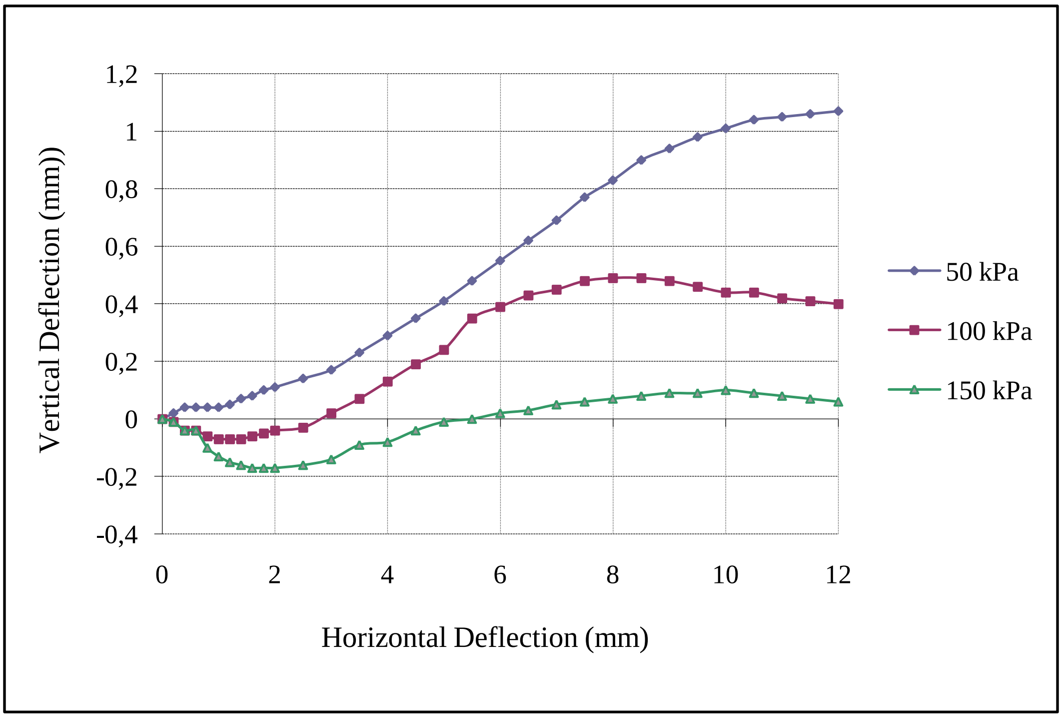

Figure 5.33. Volumetric change obtained for 27 mm sand columns of 33% sand content of sand clay mixture

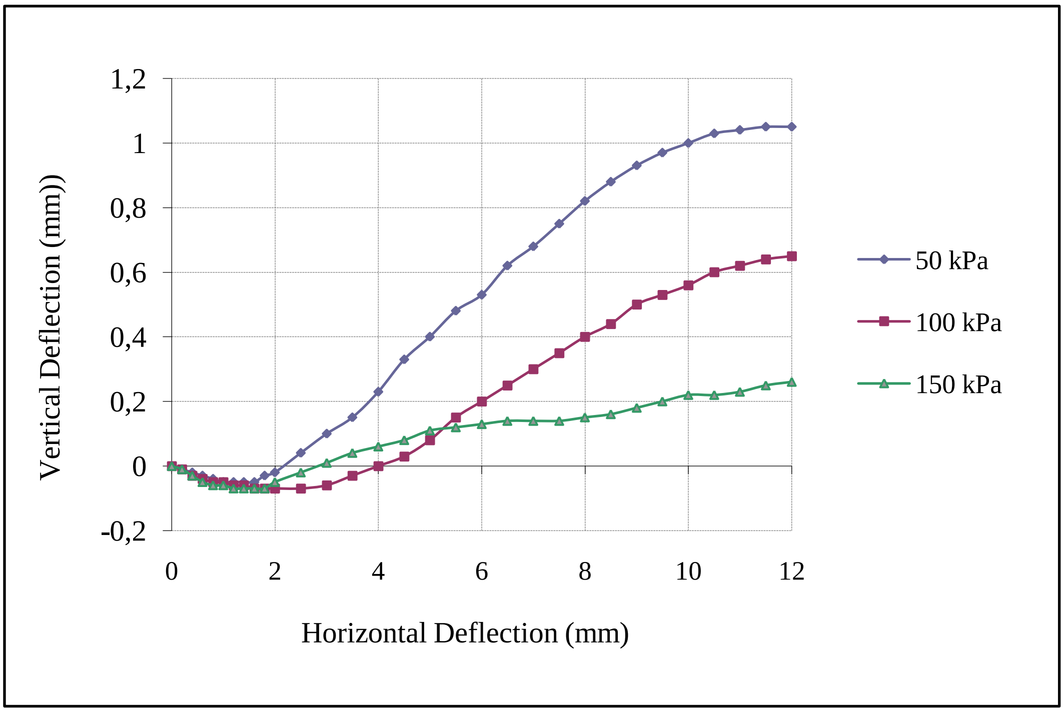

Figure 5.34. Volumetric change obtained for 37mm sand columns of 33% sand content of sand clay mixture

The shear strength parameters, therefore, for this given mixtures specimens can be revealed by plotting shear stress values versus that of normal stress as given in Figure 5.35, Figure 5.36, and Figure 5.37. The results were tabulated as given in Table 5.12. As shown in the figures, shear stress for different diameter of sand columns at 33% sand content in sand-clay mixtures increases with increasing normal stress.

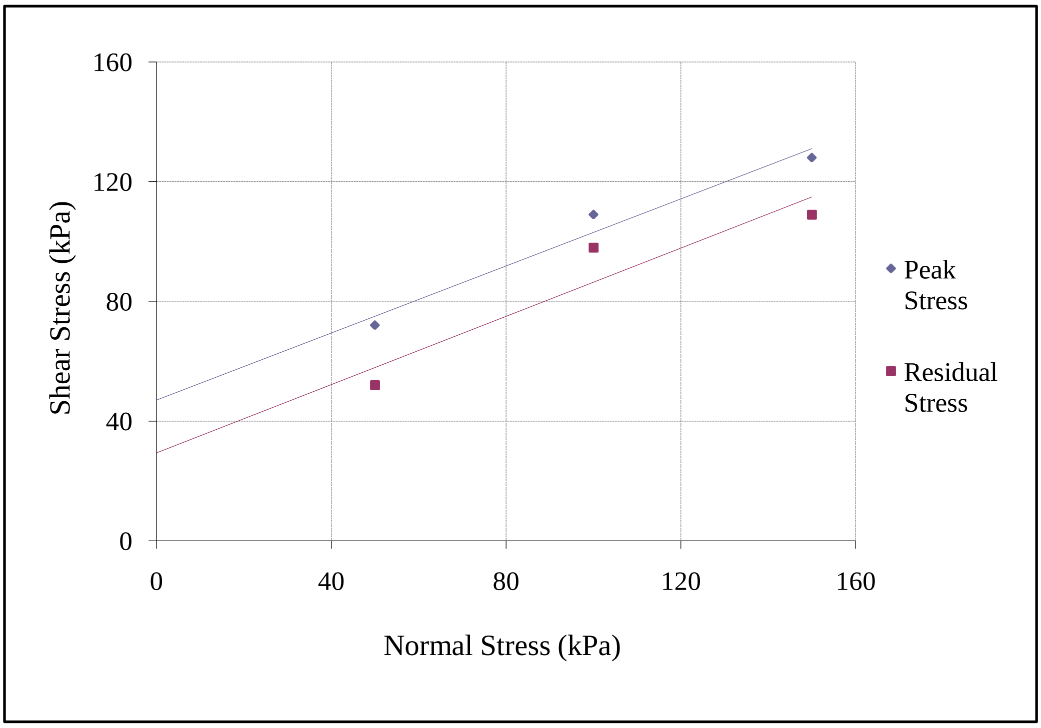

Figure 5.35. Shear strength parameters for 16mm sand columns of 33% sand content of sand clay mixture

As shown in Figure 5.35, peak and residual shear stress for 16 mm 6 sand columns in sand-clay mixtures increase with increasing normal stress. In addition, shear strength parameters internal friction angle and cohesion were calculated respectively 29° and 47 kPa for peak stress.

Shear stresses for 33% sand content of sand-clay mixtures increase with increasing normal stresses shown in Figure 5.36. Shear strength parameters (internal friction angle and cohesion) for 27 mm 2 sand columns in sand-clay mixtures were calculated and given in Table 5.12.

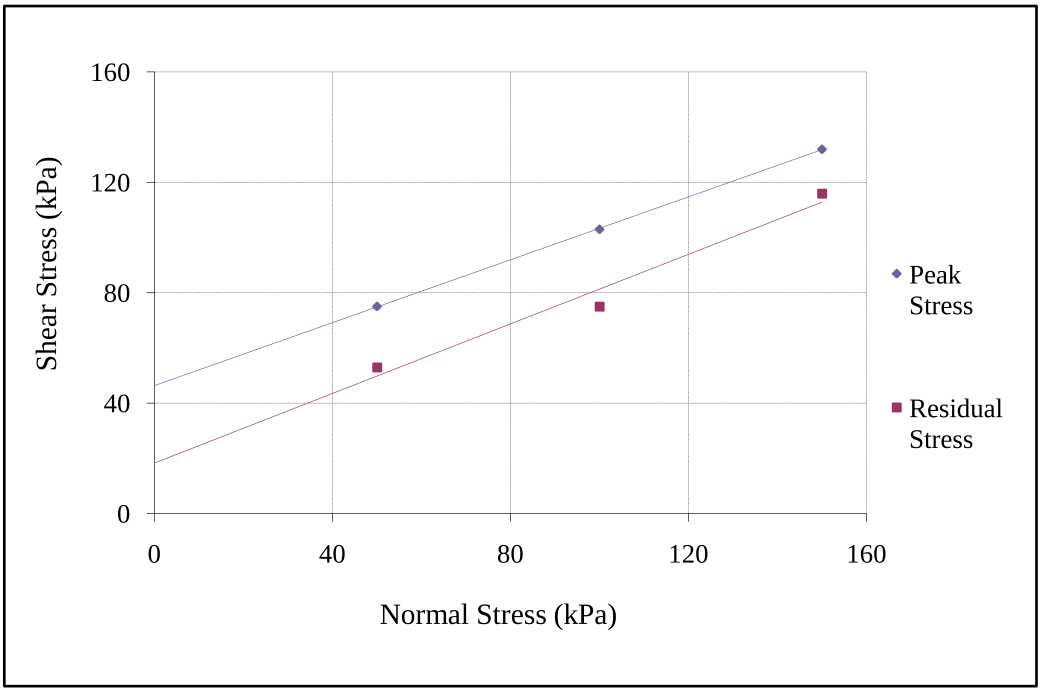

Figure 5.36. Shear strength parameters for 27mm sand columns of 33% sand content of sand clay mixture

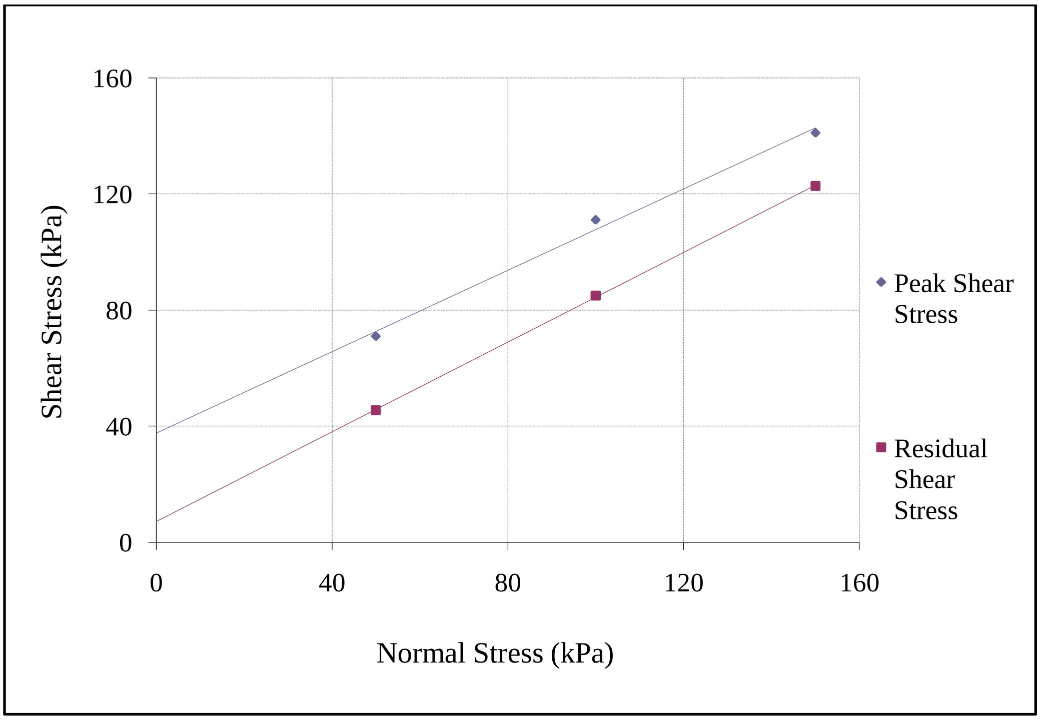

Figure 5.37. Shear strength parameters for 36mm sand columns of 33% sand content of sand clay mixture

According to direct shear tests for 37 mm 1 sand column (33% sand content) in sand-clay mixture, shear stress increases with increasing normal stress shown in Figure 5.37. In this result, internal friction angle and cohesion were calculated 35° and 37.6 kPa respectively.

Table 5.12 Shear strength parameters for of 33% sand content of sand clay mixture

Shear Strength Parameters	Peak Values	Residual Values	Diameter of Sand columns
c, kPa	47	29	16
ϕ, °	29.3	29.7	16
c, kPa	46	18	27
ϕ, °	29.8	32.3	27
c, kPa	37.6	7	39
ϕ, °	35	37.7	39

In this chapter, only direct shear tests results were given about pure clay, and pure sand, and sand – clay mixture. However, next chapter, the results of discussion and comments will be given.

CHAPTER 6

DISCUSSION OF THE TEST RESULTS

6.1. Introduction

This chapter presents results of various series of tests on sand-clay mixtures. Unconsolidated undrained direct shear box tests were performed on all specimens. Four series of tests were performed on sand-clay mixtures. In the first series 10mm diameter sand columns were used. In the second series 16mm diameter sand columns were used. In the two series tests had three different sand-clay ratios by area. These rates were 11%, 22%, and 33% sand content in sample area. In the third and fourth series tests 27mm and 39mm sand columns were used with 33% sand content in sample area. These tests were performed under 50 kPa, 100 kPa, and 15 kPa vertical pressures. The tests results were given 5.Chapter. However, in this chapter result will be explained and interpreted.

This chapter has three main parts. First one is effect of sand content on shear strength parameters of sand-clay mixtures. In the second part effect of different diameters of sand column at on shear strength parameters of sand-clay mixtures was investigated. The shear strength parameters are internal friction angle and cohesion. The third part is volume chance in all tested sand-clay mixture.

6.2. Effects of Sand Content on Sand Clay Mixtures

The tests were performed on 10 mm and 16 mm diameters of sand columns. 5, 10, and 15numbers of columns were used for 10 mm diameters of sand columns to achieve 11%, 22%, and 33% sand content on sample area respectively. For 16 mm sand columns 2, 4, and 6 numbers of columns were used to obtain 11%, 22%, and 33% sand content on sample area respectively. The shear strength parameters of internal friction angle and cohesion will be interpreted for peak and residual state of samples in this part.

6.2.1. Effects of 10 mm Diameter of Sand Columns

Pure dense sand, pure compacted clay and sand–clay mixtures for 10 mm diameters sand columns were tested and its results were given in Chapter 5. The changing of the internal friction angle with increasing sand content in sample is given in Figure 6.1.

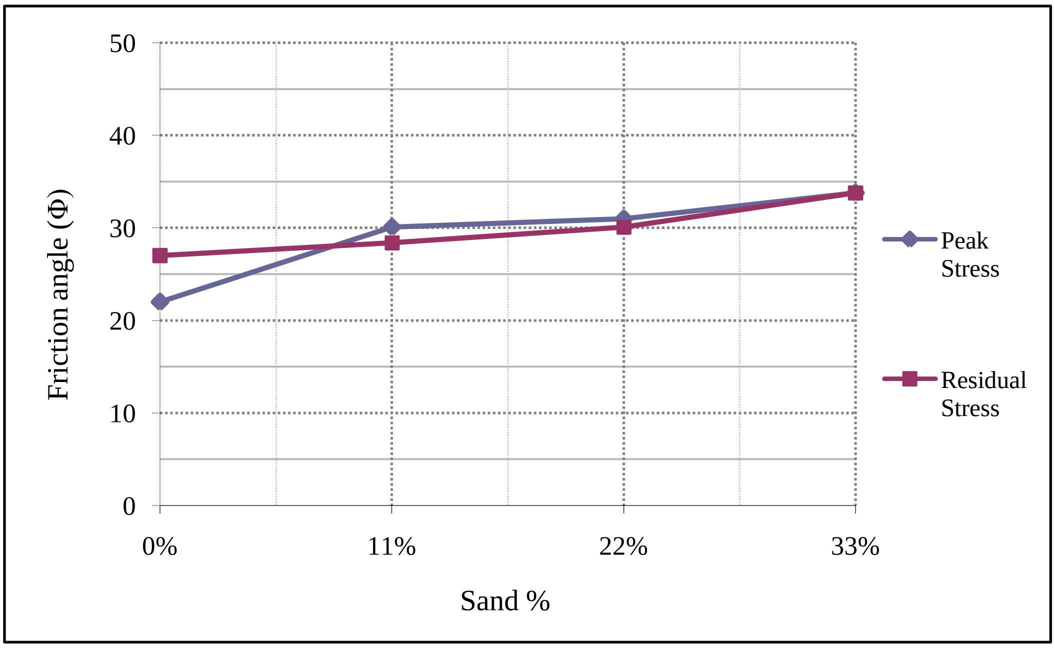

Figure 6.1. Internal friction angle for increasing sand content of 10 mm sand columns

When the sand columns were placed in clay the clays shear strength parameters started to change. The internal friction angle increases as the sand content is increased at the peak stresses. Rate of increases is very high up to 11% sand content and between 11% and 33% sand content rate of increase decreases. The compacted clays peak internal frictional angle was 22°. The compacted sand internal friction angel was 48°. This difference of sand and clay caused affection on sand-clay mixtures. Therefore, because of the increment of internal friction angle of sample was the effecting of inclusion of sand column of clay. Because of the increment of friction is relation with dilatancy between especially dense sand particles which was explained Chapter 3 shown in Figure 3.9. This effect increased the internal friction angle to 34° on peak stress a 33% sand content of sand-clay mixtures. The reasonable agreement of the increment of internal friction angles against percent clay fractions was reported by Bayoğlu at all (1995) can also be observed in Figure 2.2

The residual internal friction angle was 27° and 32.4° for pure clay and pure sand respectively. The residual internal friction angle of clay and sand were no more difference between them. Therefore, the test results were not much affected the sand content of sand-clay mixture. Because of this, affection of interlocking between sand particles is not much like dense sand particles explained Chapter 3.

Cohesion for the compacted pure clay for residual and peak stresses were respectively 33 kPa and 55 kPa. Cohesion for compacted pure sand was 0 kPa.

This difference of sand and clay caused affection on sand-clay mixtures. Therefore, because of the decrement of cohesion of sample was the effect of increasing sand content in mixture. This change is given in Figure 6.2.

Cohesion decreases remarkably as the sand content is increased at 11% for peak stresses. When the sand contents are 22% and 33% there is little increase in cohesion on the mixtures. This increase is not remarkable which may be neglect. The cohesion of pure clay decreased until 47 kPa on peak stress with increasing sand content in mixtures.

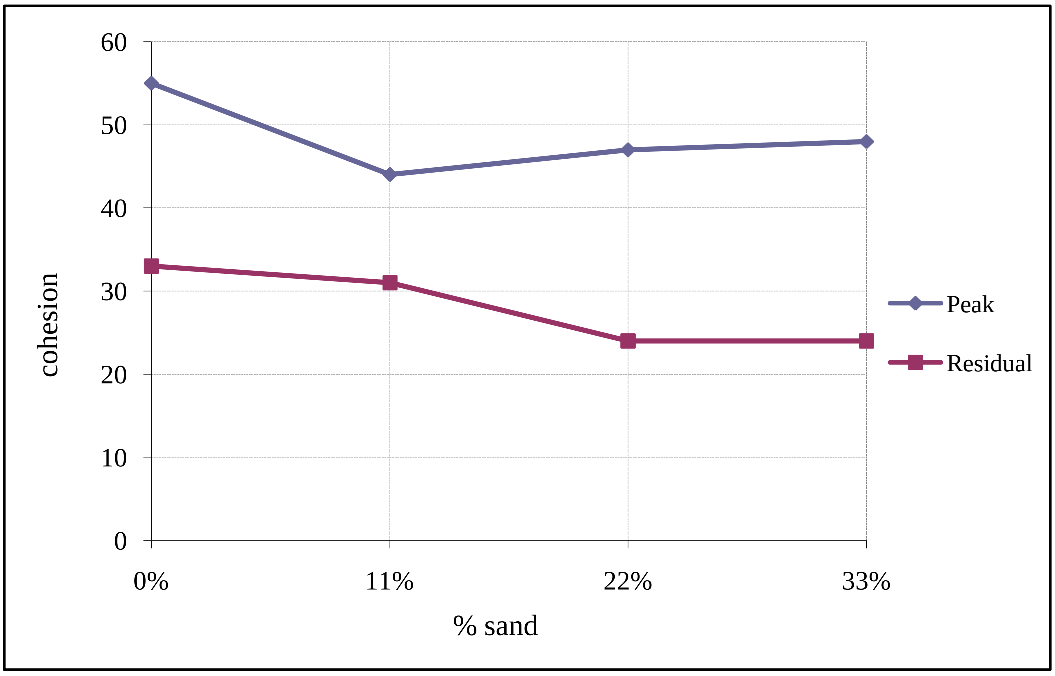

Figure 6.2. Cohesion for increasing sand content of 10 mm sand columns

Cohesion decreases as the sand content is increased at the residual stresses. There is remarkable decrease continues at 11% and 22% sand content and then the decrement decelerates at between 33% sand content. Therefore, the decrement of cohesion is the highest at 22% sand content. Because of the effect of sand content, cohesion decreased until 24 kPa on residual stress of 33% sand content of sand-clay mixtures. The reasonable agreement of the increment of cohesion against % kaolin % invented by Ölmez at all (2008) can also be observed in Figure 2.9.

6.2.2. Effects of 16 mm Diameter of Sand Columns

Pure dense sand, pure compacted clay and sand–clay mixtures for 16 mm diameters sand columns were tested and its results were given in Chapter 5. The changing of the internal friction angle with increasing sand content in sample is given in Figure 6.3 as a graphically.

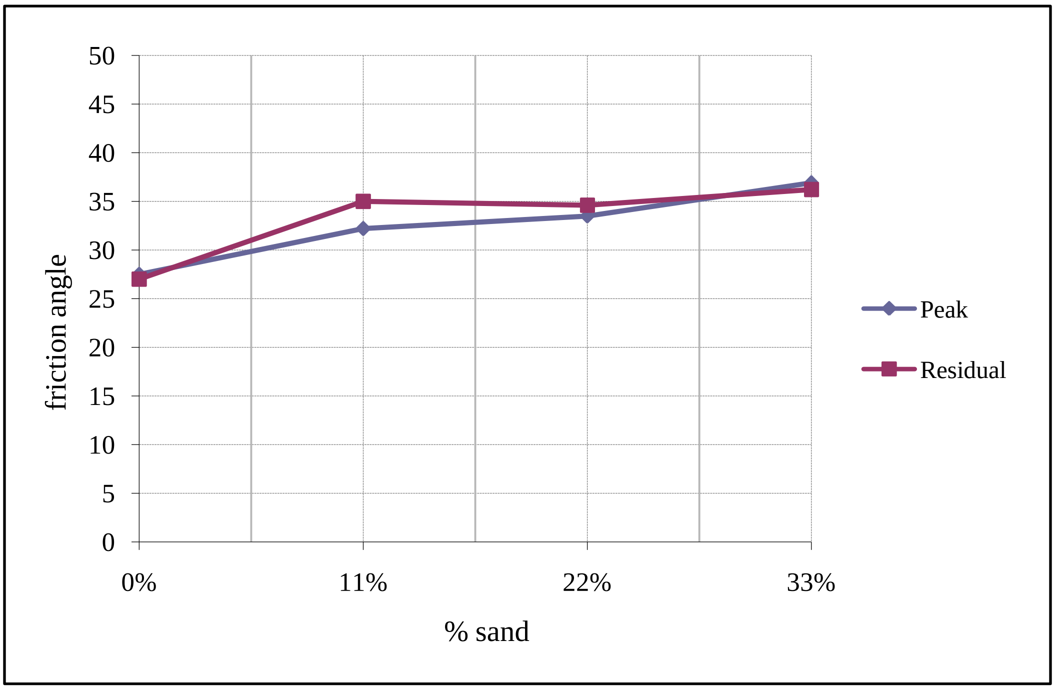

Figure 6.3. Internal friction angle for increasing sand content of 16 mm sand columns

When the direct shear tests were performed on clay mixtures with sand columns the clays shear strength parameters were observed to change. The tests for 16 mm columns were found nearly same results for peak and residual stresses. The internal friction angle increases as the sand content is increased for both peak and residual stresses. There is remarkable increase in internal friction angle at 11% sand content and between 11% and 22% sand content the increment decelerates and the friction started to decrease for residual stress. However, the internal friction angle for peak and residual stress increase again at 33% sand content. The compacted clays peak internal frictional angle was 27.5°. The compacted sand internal friction angel was 48°. This difference of sand and clay caused affection on sand-clay mixtures. It was seen the same behavior in this results like 10mm sand column tests. Therefore, of the increment of internal friction angle of sample was the effecting of increasing sand content of clay fraction. This effect increased the internal friction angle until 37° and 36° respectively for peak stress and residual stresses of 33% sand content of sand-clay mixtures. Because of the sand affect in mixture was explained in shear strength of cohesive soil and shear strength of sand in Chapter 3. Dilatancy effect between sand particles causes increasing internal friction. In addition, the reasonable agreement of the increment of internal friction angles against % clay fractions % invented by Bayoğlu at all (1995) can also be observed in figure 2.2. Another reasonable agreement, higher than 15% sand content, internal friction increases increasing sand content invented by Canakci and Gullu (2007).

Cohesion decreases remarkably as the sand content is increased at 11% for peak stresses. The decrement decelerates at 22% sand content and then the decrement of 33% sand content is as more as decrement of 11% sand content. Cohesion for the compacted pure clay and compacted pure sand for peak stress were respectively 59 kPa and 0 kPa. This difference of sand and clay caused affection on sand-clay mixtures. Therefore, because of the decrement of cohesion of sample was the affecting of increasing sand content of clay fraction shown in Figure 6.4. Consequently, the cohesion of the mixture on peak stress decreased until 43kPa.

There is no change in cohesion as the sand content is increased at the residual stresses until 33% sand content. There is remarkable decrease in cohesion at 33% sand content. Cohesion for the compacted pure clay and compacted pure sand for residual stress were respectively 26 kPa and 0 kPa. This difference of sand and clay caused affection on sand-clay mixtures. Because of this, the cohesion of the mixture on residual stress decreased until 13kPa (figure 6.4). The reasonable agreement of the increment of cohesion against % kaolin % invented by Ölmez at all (2008) can also be observed in Figure 2.9.

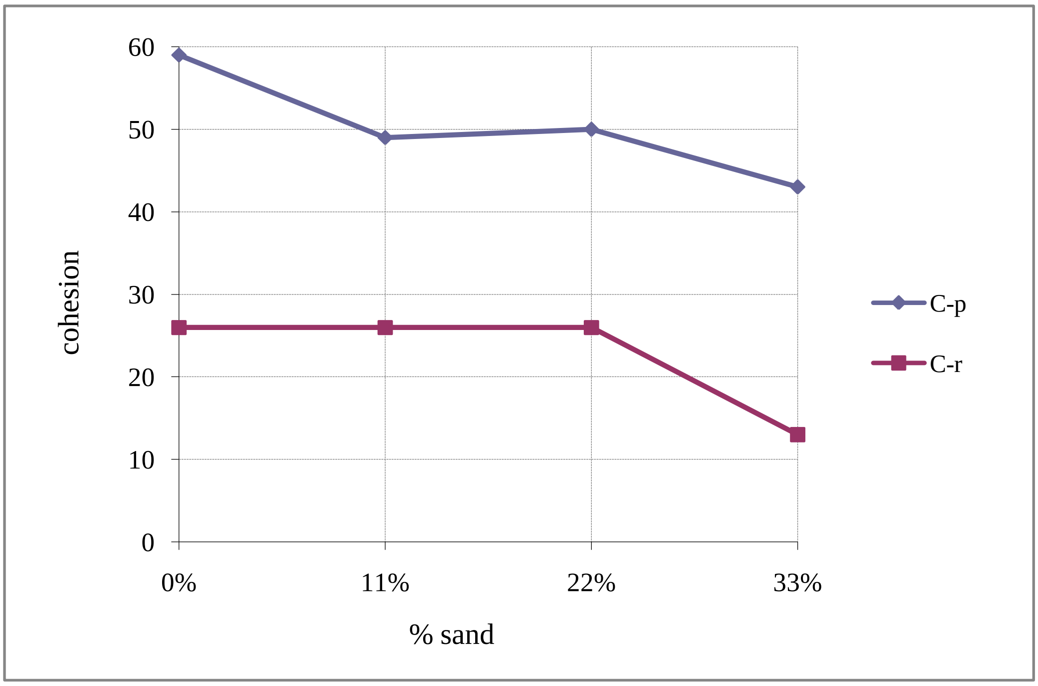

Figure 6.4. Cohesion for increasing sand content of 10 mm sand columns

6.3. Effect of Different Diameters of Columns on Sand-Clay Mixtures

The tests were performed on different diameters of sand column as the same sand content in sand - clay mixture and its results were given Chapter 5. It was explained effect of sand content of sand-clay mixtures in last part. The sand content in clay layer by area is always equal of 33% in this part. The used sand columns diameters are 10 mm, 16 mm, 27 mm and 39 mm. In this section increasing sand column diameter with same sand content in sand – clay mixtures will be explained from obtained tests results. The test results are commented with column diameter and column section area. The shear strength parameters of internal friction angle and cohesion will be interpreted for peak and residual state of samples in this part from the test results. These are shown in Figure 6.5, Figure 6.6, Figure 6.7, Figure 6.8.

The tests were performed on the mixtures for peak and residual states. All sand content of the mixtures were 33% in the tests. The angle of friction internal friction firstly decreases between 10mm and 16mm diameter of sand columns and then the decrement decelerates and stops. Afterwards, internal friction of sand-clay mixtures increases remarkably with increasing diameter of sand column for both residual and peak state shown in Figure 6.5 and Figure 6.6.

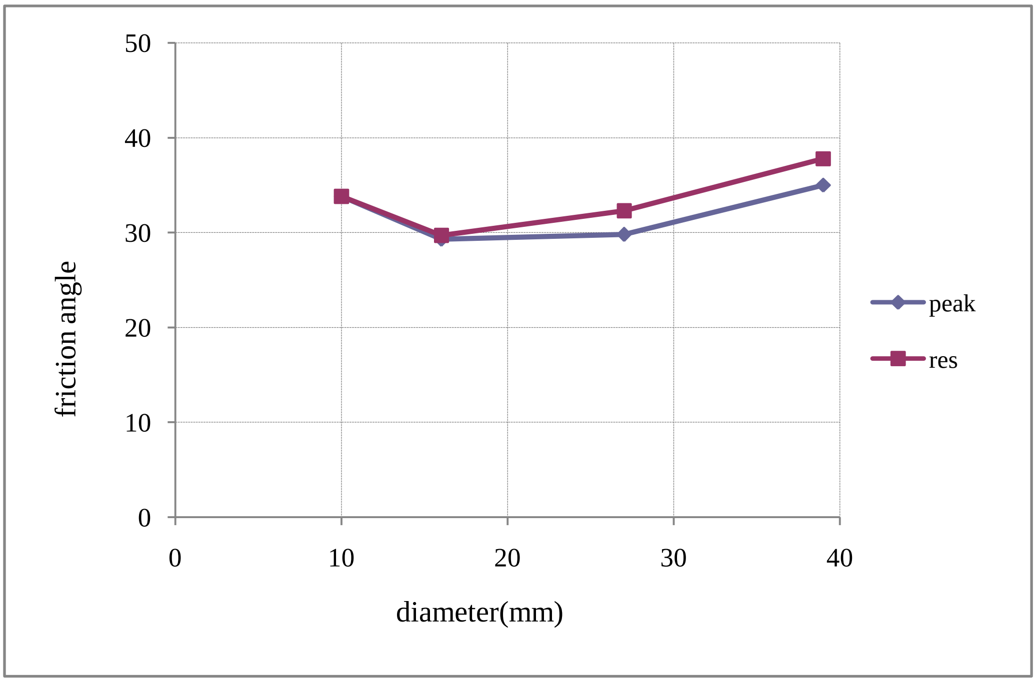

Figure 6.5. Internal friction angle with increasing diameter of sand columns

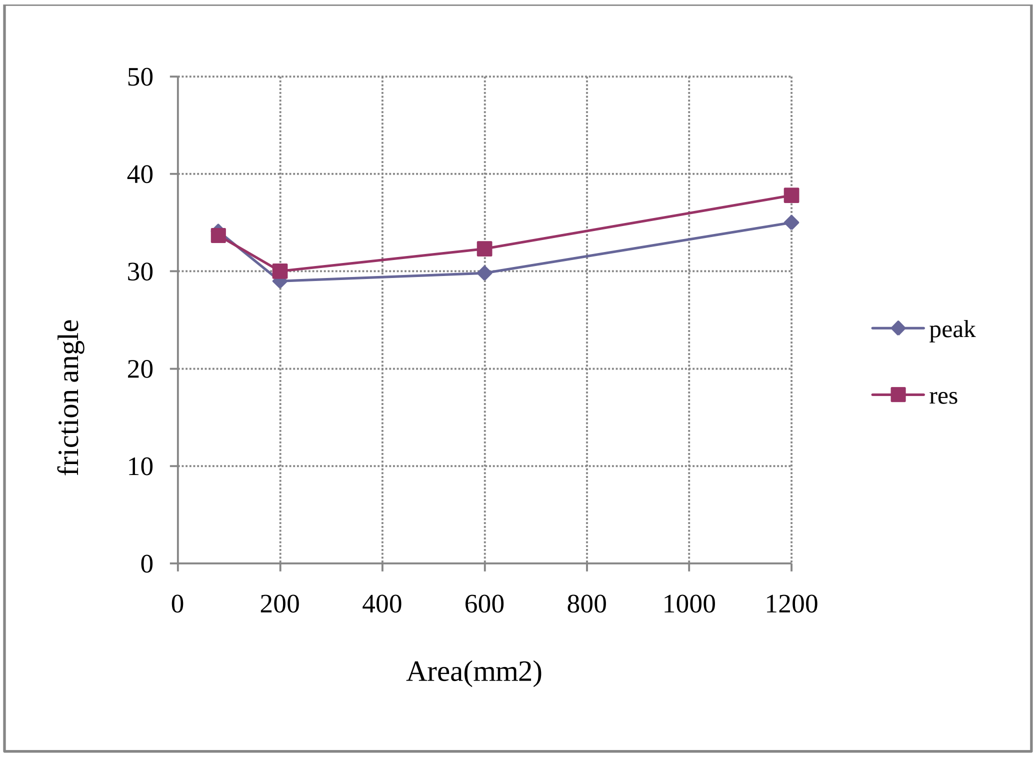

Figure 6.6. Internal friction angle with increasing section area of sand columns

There is no more change in cohesion at first up to 27 mm diameter of sand column for peak state shown in Figure 6.7 and Figure 6.8. Subsequently, cohesion decreases remarkably between 27mm and 39mm diameter of sand column on peak state.

On residual state, cohesion of sand-clay mixture increases firstly between 10mm and 16mm diameter of sand columns. The increment decelerates and stops. Eventually, cohesion decreases with increasing diameter of sand column of sand-clay mixtures shown in Figure 6.7 and Figure 6.8.

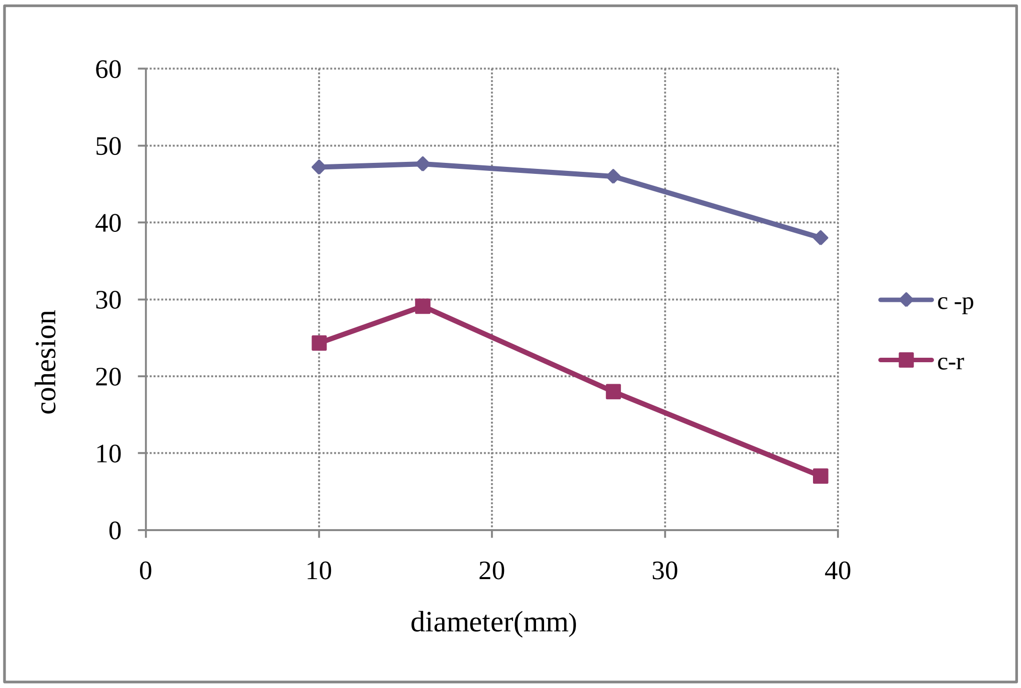

Figure 6.7. Cohesion with increasing section diameter of sand columns

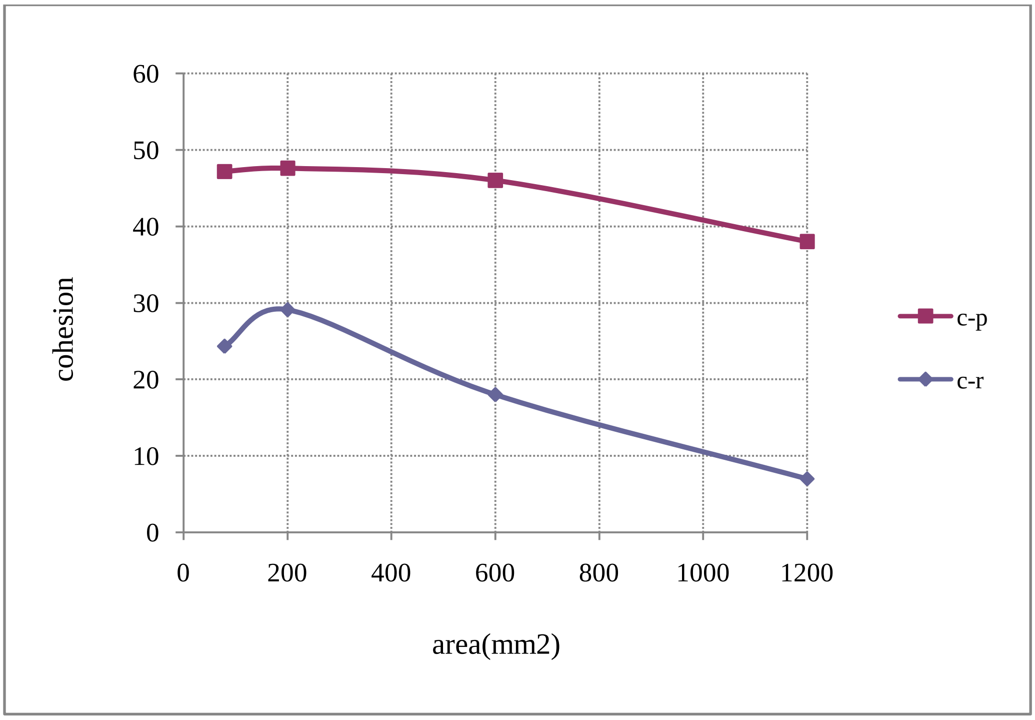

Figure 6.8. Cohesion with increasing section area of sand columns

Finally, internal friction angle generally increases with increasing diameter of sand column and cohesion generally decreases with increasing diameter of sand column in sand clay mixtures of undrained direct shear tests. Dilatancy effect between sand particles increases with increasing section area of the sand columns. The reasonable agreement of friction and cohesion against sand column area invented by Black at all (2007).

6.4. Volume Changes on Sand-Clay Mixtures

Undrained direct shear tests were performed on sand-clay mixtures with different content and dimensions of sand columns. Shear strength parameters (ϕ, c) were explained in the last part.

Volume change of the sand–clay mixtures were calculated on one direction that is vertically. A gauge reading was placed on the top of the test specimen and volume chances of the test results were obtained which were given chapter 5.

Volume changes of sand-clay mixtures were explained in two parts. They are volume changes with increasing sand content and increasing dimension of sand column of vertical load of 50 kPa, 100 kPa and 150 kPa. All values of volume changes were taken from all direct shear tests results at peak point in this part. The values were given Table 6.1.

Volume of sand clay-mixtures increases with decreasing of vertical load in all tests.

Volume increases with increasing sand content shown in Figure 6.9. Because of this increment of the volume, especially volume of the dense sand increases remarkably under shear force. This increment of volume of sand is named dilatancy effect. Reasonable agreement of the increment of the volume was explained in Chapter 3 on Figure 3.8 and Figure 3.9.

Table 6.1 Volume values of the peak point of different diameter of sand column

Loading (kPa)	Volume of 10mm sand column (mm)	Volume of 16mm sand column (mm)	Volume of 27mm sand column (mm)	Volume of 39mm san column (mm)
50	1.09	1.22	1.07	1.05
100	0.48	0.55	0.49	0.65
150	0.19	0.28	0.1	0.26

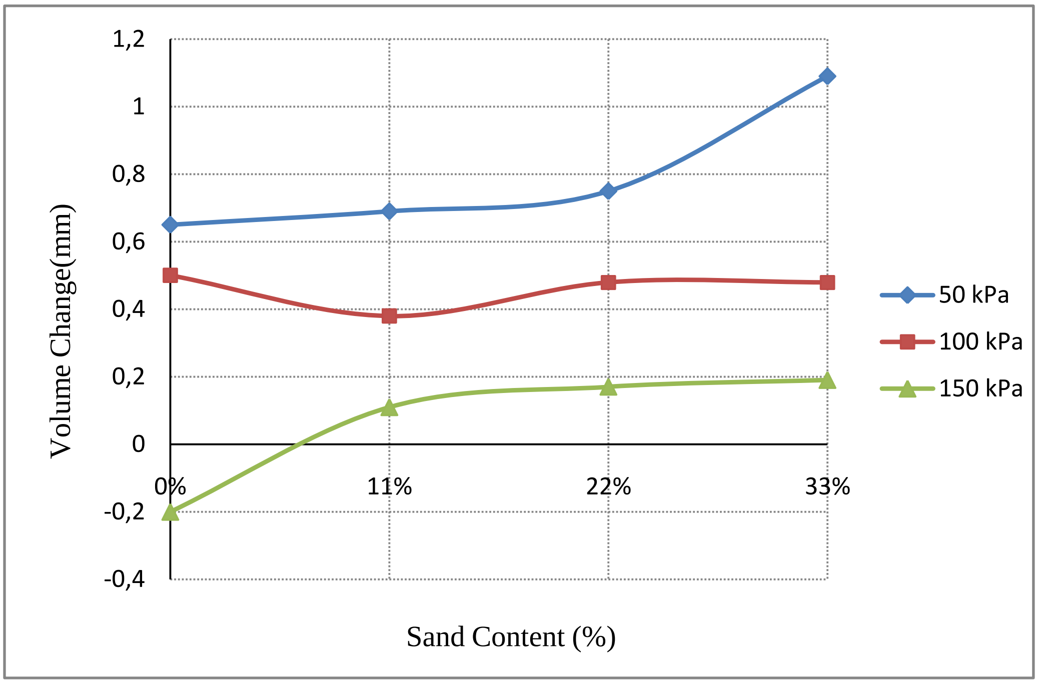

Figure 6.9. Volume changes with increasing sand content of 10mm sand columns

Volume changes of sand clay mixture is nearly same with increasing sand content for both 10mm and 16mm diameter of sand columns shown in Figure 6.9 and Figure 6.10. Volume changes in clay are slower than granular soil which was explained in shear strength of cohesive soil in Chapter 3.

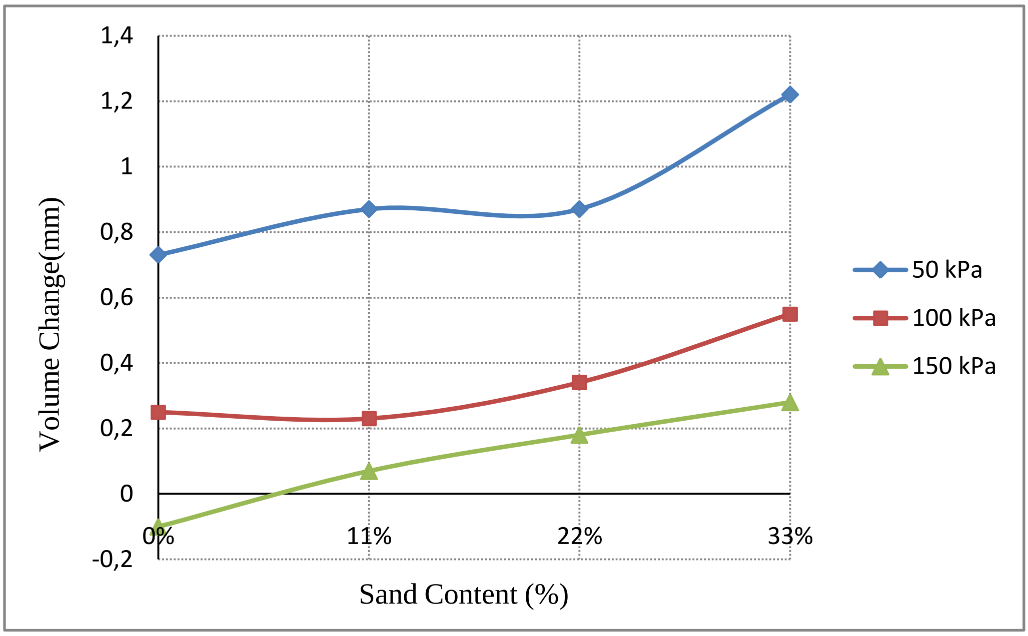

Figure 6.10. Volume changes with increasing sand content of 16mm sand columns

Volume of the sand-clay mixtures specimen increases firstly with increasing diameter of sand column but the increment decelerates and stops. After wards, volume decreases slightly with increasing diameter of sand column shown in Figure 6.11. Nevertheless, the volume change of different diameter and area of sand-clay mixture can be neglect because there is not more change of volume and there is not more affect of diameter and area shown in Figure 6.11 and Figure 6.12

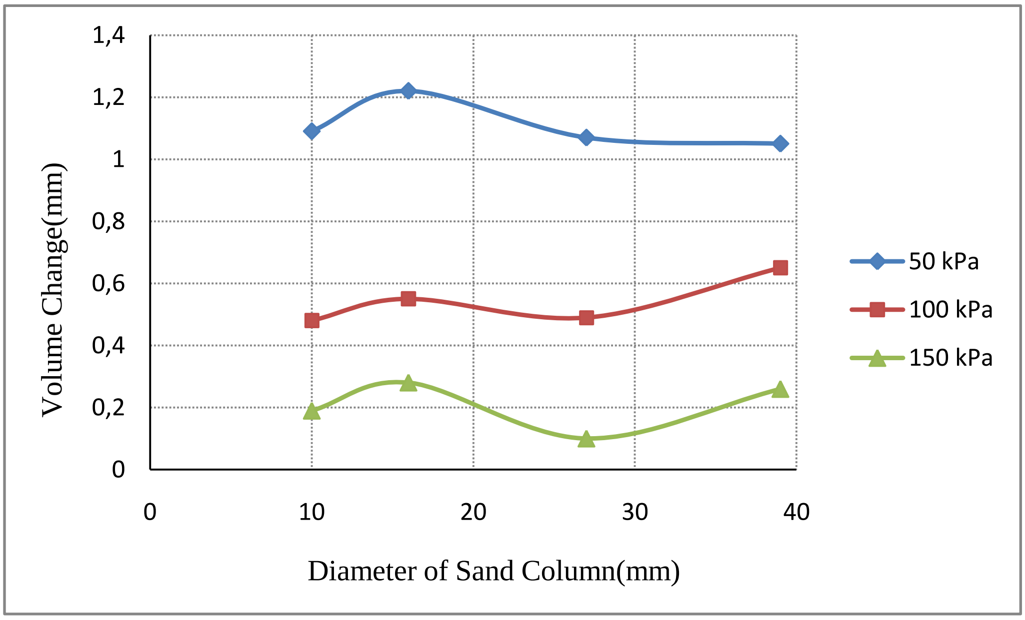

Figure 6.11. Volume changes with increasing diameter of sand columns

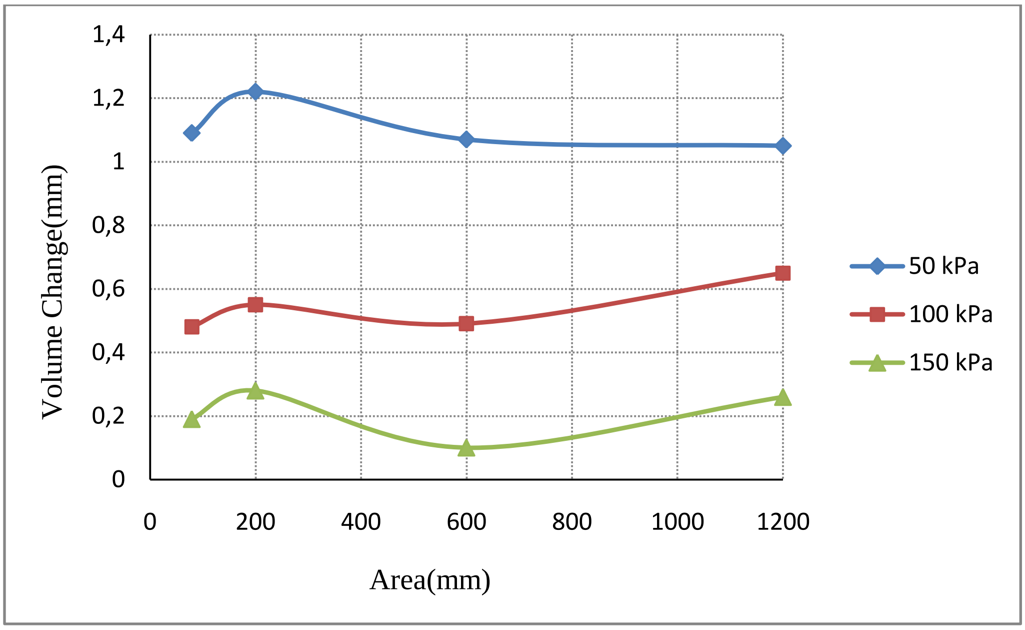

Figure 6.12. Volume changes with increasing area of sand columns

Finally, volume of the mixture increases with sand content but volume does not change much with increasing diameter of sand column in sand-clay mixtures.

In this section, discussions of the test results were explained. Next section is the final section on these studies. The conclusions and future work of the studies will be given.

CHAPTER 7

CONCLUSIONS

7.1. Conclusions

In this study, effect of number of sand column and effect of sand column size on shear strength of low plastic clay were investigated. Undrained shear strength parameters internal friction angle (ϕ) and cohesion (c) were determined. In addition, volume changes during the tests were observed. The following conclusions can be obtained from the test results of the experimental study.

1. Internal friction angle of sand-clay mixtures increases with increasing number of sand column and sand content. The increment of internal friction angle was observed significantly at 11% sand content. After 11% content, the increment decreases.

2. The internal friction angle of sand-clay mixtures increases similarly on both 10 mm and 16 mm diameter of sand columns. The increment of friction angle is nearly 10°.

3. Cohesion decreases remarkably at 11% sand content of the sand-clay mixtures on both 10 mm and 16 mm sand columns. After 11% sand content, decrement in cohesion diminished and nearly stops.

4. Shear strength parameters (ϕ, c) are considerably effected from 11% sand content of sand-clay mixtures. 11% sand content can be defined as effective content of the sand-clay mixtures.

5. Internal friction angle of different diameters sand columns at same sand content (%33) decreases up to 16 mm sand column. Afterwards, internal friction angle increases with increasing column diameter.

6. Cohesion increases firstly between 10 mm and 16 mm sand columns after that it decreases with increasing column diameter of same sand content.

7. Volume of the sand-clay mixtures increases with increase in sand content and decreases with increasing vertical load.

8. Volume of the sand-clay mixtures increases slightly with increasing diameter of sand column during shearing process.

7.2. Future Works

Following future work can be performed on sand-clay mixture. Shear strength parameters of sand-clay mixture with compacted clay columns incorporated in sand can be investigated. Effect of clay plasticity, water content, and stiffness on shear strength parameters can also be studied.

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