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SOME ENGINEERING PROPERTIES OF MIXED SOILS WITH VARIOUS PORE FLUIDS

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Rozhgar Abdullah HASAN
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Some Engineering Properties of Mixed Soils with Various Pore Fluids

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Supervisor
Assoc. Prof. Dr. Ali Fırat ÇABALAR

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Rozhgar Abdullah HASAN

ABSTRACT

SOME ENGINEERING PROPERTIES OF MIXED SOILS WITH VARIOUS PORE FLUIDS

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M.Sc. in Civil Engineering
Supervisor: Assoc. Prof. Dr. Ali Fırat ÇABALAR
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This thesis presents an extensive series of experimental works conducted on different shape and size of sand saturated with different viscose pore fluids. It is aimed to investigate the effects of shape, size of sand particles, as well as the effects of different chemical viscose pore fluids on the strength properties of sand. On the other hand sand- clay mixtures were experimentally tested to evaluate the effects of clay content in various mix ratios on shear strength and compression behavior of the soils. In the study, two series of experiments have been performed. Direct shear box tests were performed on various grain sizes of angular and rounded sand in a loose state, saturated with water, petrol or gasoline. It was found that shear strength of the sands decrease as the viscosity of fluids increase. From the experimental investigations on the effects of both shape and size of the sands, it was found that an increase in angularity of sand grains causes an increase in peak friction angle, and that the particle size was found to be effective on the stress–strain relationships. Oedometer testing machine were performed on samples of clean sand, and sand homogeneously mixed with clay at different mix ratios of 5% up to 30%. The samples tested were saturated with water, petrol or gasoline. It was found that the compression behavior of mixed soils was increased with increasing clay content.

Keywords: *sand, sand– clay mixture, fluids, shape, size, shear strength, compression.*

ÖZET

FARKLI BOŞLUK SIVISI İLE HAZIRLANAN KARIŞIM ZEMİNLERİN BAZI MÜHENDİSLİK ÖZELLİKLERİ

HASAN, Rozhgar
Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Doç. Dr. Ali Fırat ÇABALAR
2012, 88 Sayfa

Bu çalışmada, farklı boşluk sıvıları ile doygun halde hazırlanmış farklı şekil ve büyüklükteki temiz kum, ve kum-kil karışımları üzerinde yapılmış bir deneysel çalışma sunulmaktadır. Çalışmada, hem kum tanelerinin şekil ve büyüklükleri, ve hem de boşluk sıvısı olarak kullanılan farklı akmazlığa sahip sıvıların mukavemet ve sıkışabilirlik özelliklerine etkisi araştırılmıştır. Farklı oranlarda hazırlanmış kum- kil karışımlarının kesme mukavemeti ve sıkışabilirlik özellikleri de incelenmiştir. Çalışmada, başlıca iki deneysel çalışma yapılmıştır. Direk kesme kutusu deneylerinde yapılan çalışmalarda, gevşek halde hazırlanan farklı büyüklük ve şekildeki kum örnekleri su, petrol ya da benzin ile doygun hale getirildikten sonra test edilmiştir. Bu deneyler sonunda, kullanılan sıvının akmazlığı arttıkça ulaşılan kesme mukavemeti değerlerinin arttığı gözlemlenmiştir. Kum danelerinin büyüklüğün sonuçları üzerinde etkili olduğu, köşeli olmalarının ise içsel sürtünme açısı değerlerini artırdığı tespit edilmiştir. Temiz kum, ve farklı orandaki kum- kil karışımları üzerinde ödometre deneyleri de yapılmıştır. Kil oranları %5 ile %30 arasında değişirken, karışımlar deneye tabi tutulmadan önce su, petrol ya da benzin ile doygun hale getirilmişlerdir. Bir sıvının akmazlığının sıkışabilirlik özelliğini etkilediği ve kil oranının ise bunu artırdığı gözlemlenmiştir.

Anahtar Kelimeler: *kum, kum-kil karışımı, akışkan, şekil, boyut, kesme mukavemeti, sıkışabilirlik.*

To The Soul of my Father. My loving Mother and those who helped me.

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LIST OF SYMBOLS

AS	Crushed stone sand (angular sand specimen)
RS	Trakya sand (rounded sand specimen)
CL	Clay
e	Void ratio
e_o	Initial void ratio
e_{max}	Maximum void ratio
e_{min}	Minimum void ratio
Dr	Relative density
FC_t	Fine transition content
e_s	Intergranular void ratio
Cc	Compression index
Cc-s	Granular compression index
FC	Fines content
ϕ^o	Internal friction angle
G_s	Specific gravity

CHAPTER 1

INTRODUCTION

Field investigations show that the most common and wide spread materials are granular materials. Sand as a type of granular material is available in nature as various shapes and sizes. It is the geotechnical responsibilities to predict the influence of shape of sand particles on engineering behavior of soils. Unfortunately there are limited researches in establishing the role of shape. It is because most of the studies concerning the role of various sized sand grains instead of their shape. Actually, the effects of shape is not less than the role of particle size on stress- strain relationships, shear strength, and compression behavior of sandy soils. In addition, the role of fines mixed with sand in various mix ratios are taken into account in some studies. Because field investigations showed that granular materials may inhabit considerable amount of fines. Practically established that these fines add some contributions especially to shear strength and compression behavior of mixed soils. The effects of various viscosity pore fluids are also studied in this thesis so as to show the effects of contaminant chemical fluids on the engineering properties of clean sand and sand mixed with fines.

In 1925 Terzaghi, in reviewing differences in soil behavior, noted that in superficial examination sand and clay seem to be different materials but concluded that no essential differences was found to exist between sands and clays other than differences in size and shape. In 1928 Gilboy noted that the shape of grains had an almost significant influence on the compression characteristics of a granular material. Over the past decade, research studied related to particle characteristics have been accelerated (McDowell and Bolton,1998, Nakata et al., 1999; Thornton, 2000; Clayton and Heymann, 2001, Clayton, 2010). The reasons behind these researches is the knowledge about the role of shape in soil response and the relationship between individual particles overall material response still remains poorly understood.

Fines could affect the stress- strain, shear strength, and liquefaction potential (Georgiannou et al., 1990; Fear and McRoberts, 1995; covert and Yamamuro, 1997; Lade and Yamamuro, 1997; Boulanger et al., 1998, Thevanayagam and Mohan, 1998; Yamamuro and Lade, 1999; Perlea, 2000; Salgado et al., 2000; Yamamuro and Covert, 2001; Thevanayagam and Martin, 2002; Xenaki and Athanasopoulos, 2003; Naeini and Baziar, 2004; Cabalar, 2008). Also there are some models in literature about the compression of cohessionless soils (Hardin, 1987; Pestana and Whittle, 1995; Cabalar, 2010). These models assessed the effects of initial void ratio, relative density, particle shape, mineralogy, structure and applied stress conditions. Viscose fluids may adhere some contributions on engineering behavior of granular material. There are some researches on the effects of contaminant soil by chemical viscose fluids on the response of coarse grain granular material and fine grain (i.e., sand, clay). For example, Acar and Olvieri, (1989) Evgin and Das (1992) Alsanad et al., (1995) Alsanad and Ismail, (1997) Shine et al., (1999), and Shine and Das (2001). The goals behind these researches is to understand the role of environmental impact of contaminant harmful substances on the geotechnical properties of soils to be properly used in construction. Recently, however, research studies on the adverse effects of chemical viscose fluids on the shear strength, atterberg limit, damping and compression behavior of soils accelerated markedly (Khamehchianyan et al., 2007; Cabalar and Clayton, 2010; Habib –ur-Rehman, 2010; Zulfahmi et al., 2010; Zulfahmi and Hamzah 2011). Further investigations on the effects of contaminated viscose fluids required to understand better the drastic role of chemical fluids on shear strength, atterberg limits, compression and damping of sand and sand- clay mixtures.

Objective of this research is to investigate the shear strength and compression behavior of various size and shape of sands, and sand – clay mixtures. The effects of saturated chemical viscose fluids on this interaction is also examined and correlated with the water saturated usual case by oedometer and direct shear test performed on sand and sand – clay mixtures.

The outline of the thesis is as follows: Chapter 2 gives a background related with some of the important concepts such as shear strength of various size and shape of sands, settlements of coarse and fine grained soils, discussion the results of various research analyzes about stress-strain properties of sandy soils. Chapter 3 focuses on contaminant soil and its effects on geotechnical behavior of sandy soils. Properties of materials used throughout the tests are given in Chapter 4. The experimental program is also explained in a detailed manner. The results are presented and discussed in Chapter 5. Chapter 6 gives conclusions drawn from the previous chapters.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

2.1 Shear Strength of Soils

The shear strength of soil is defined as "the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it" (Das, 2002). There are two important shear strength parameters for soils, the angle of internal friction (ϕ) and cohesion (c). The ϕ angle indicates the degree of friction and interlocking among the soil particles, and the cohesion represents the ionic attraction and chemical cementation between the soil particles. Both of these parameters can be determined in a geotechnical laboratory by performing shear strength tests. The shearing resistance for soil is embraced of the following main components (Kumari, 2009):

- Structural resistance of soil to displacement due to the interlocking of soil particles.
- Resistance of friction to translocation between the individual soil particles at their contact points.
- Cohesion or adhesion between the surfaces of soil particles.

Shear strength of soil is a function of the normal stress applied, the angle of internal friction, and the cohesion. The angle of internal friction describes the interparticle friction and the degree of the mechanical interlocking of particles. This characteristic depends on soil particle gradation, shape, and the void ratio. Cohesion describes soil particle bonding caused by electrostatic attractions. So, with normal stress, the angle of internal friction, and cohesion. The shear strength of a soil indicates of the strength and stability of the soil under various loading conditions, compaction, and moisture content. Shear strength parameters are crucial for stability analysis against slope failures and landslides. Soils with high shear strength will be able to support structures without failing. Otherwise, the structure will not be stable and side effects will occur, either, in the short term or long term depending on the shear strength (Kojo, 2010). The following equation, known as the Mohr-Coulomb theory, can be used to find the shear strength of soil based on a total normal stress:

$$T = c + \sigma (\tan \phi) \qquad (2.1)$$

where

T = Shear stress

c = Cohesion

σ = normal stress applied

ϕ = angle of internal friction

For saturated soil case, water fills all the void spaces between the soil particles. This leads to the concept of effective and normal stress. When a body of saturated soil is subjected to load, the total stress is effectively carried by the structure of soil particles and the water filling the pore space presents between soil particles. Therefore the total stress is effectively carried by soil particles, and resist offered by the water filled voids. This distribution of stress leads to change the shear strength of soil, because some of the energy are lost which produced by applied load to the soil structure due to the resistance of pore water pressure. The equation has given below describes the saturated soil case:

$$\sigma = \sigma' + u \tag{2.2}$$

where σ = Total stress; σ' = effective stress; and u = pore water pressure.

Effective stress represents the average stress carried by the soil skeleton. According to Terzaghi (1936). All the measurable effects of a change in stress, such as compression, distortion, and a change in shearing resistance, are exclusively due to changes in effective stress. In terms of effective stress σ' expressed by Mohr-Coulomb failure criterion, the effective stress carried by the soil solids, will be in the form:

$$T = c' + \sigma_n'. \tan \phi ' \tag{2.3}$$

where

T = shear stress at failure plane

c' = effective cohesion

σ_n' = effective normal stress at failure plane

ϕ' = effective friction angle

The significance of equation (2.3) can be explained by referring to figure 2.1. The stress on shear plain are resolved in to the effective normal stress σ_n', which acts perpendicular to the surface and the shearing stress, T, which acts along the surface.

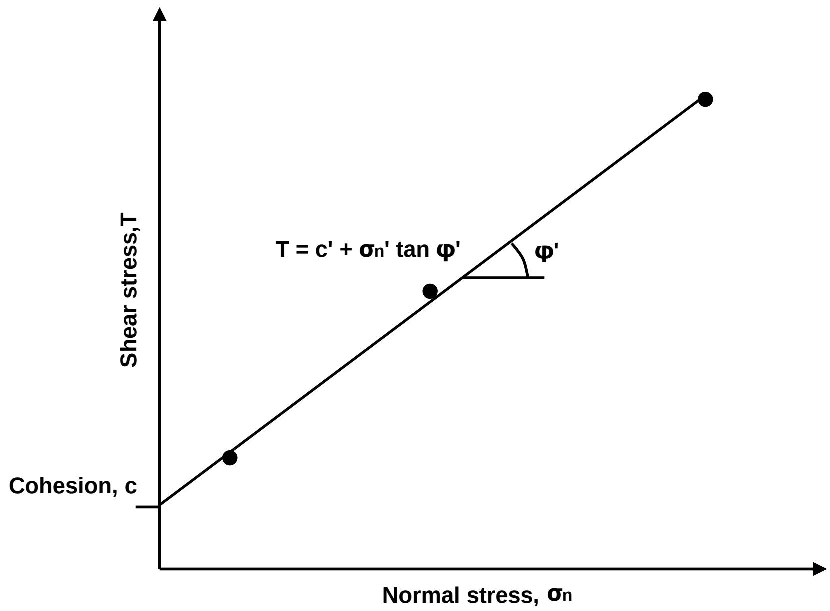


Figure 2.1: Mohr-Coulomb failure envelope

2.1.1 Shear Strength of Sands

The shear strength of sand can be represented by equation 2.4. which is a special case of equation 2.3, where c = 0.

$$T = \sigma_n . \tan \phi \qquad (2.4)$$

In general, the value of friction angle for cohesionless soil are affected by,

- Compaction state and the density of soil. The friction angle increases with increasing the density (decreasing voids ratio), but the increase is not linearly.
- Coarseness, shape of grains. Interlock of angular grains are more effective than rounded ones, therefore emerging a larger friction angle.
- Mineralogical content of soil. Hard particles result in higher friction angles than soft grains, which may crush more easily, thereby reducing the interlocking or bridging effects. The mineralogical content seems to make little difference except for sands if contains mica. In that case usually the void ratio is larger, thereby resulting in loose interlocking sand lower friction angle.
- Effects of grain size distribution. Dense, well - graded sand usually displays higher friction angle than uniform sized sand (Cernica, 1995).

The direct shear test is one of the equipments used to determine the shear strength of soils. The principal of direct shear test apparatus is illustrated in figure 2.2. The soil specimen is placed inside the rigid shear box. Normal force on the specimen is applied from the top of the shear box and the shear force is applied by moving one half of shear box relative to the other to cause failure in soil specimen. The failure occurs along the plain of split of the two halves of shear box. The shear strength of soil is the shear stress T that causes the upper half of soil slip over the lower half on the plain of split. It can be defined by Mohr-Coulomb theory (in equation 2.2).

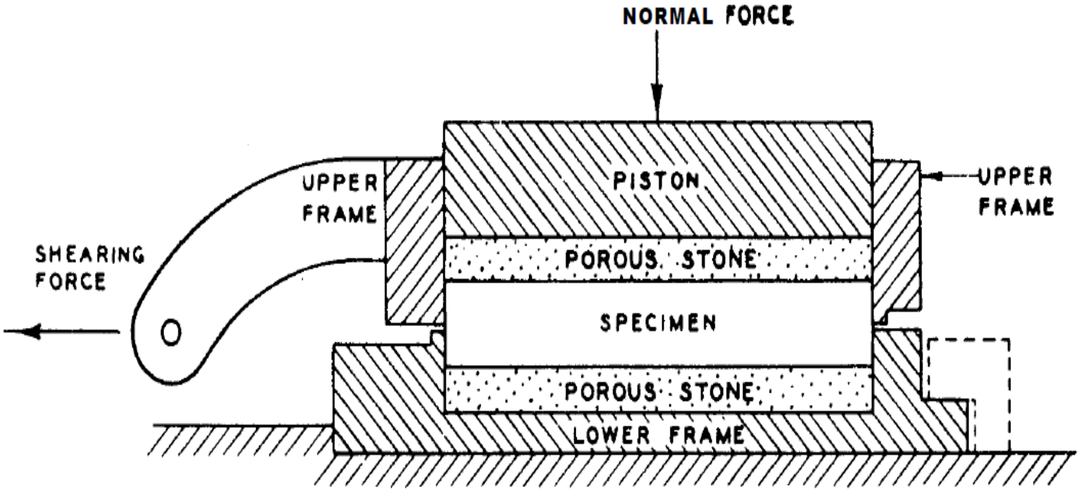


Figure 2.2: Schematic diagram of direct shear box (Maclver, 1970)

When shear stress is applied, the resulting distortion is usually associated by a volume change, which is known as dilatancy. The volume change is the result of the presence of voids in the soil. Void ratio is one of the most important parameter that affects the shear strength of soil. The lower the void ratio (densest packing) the higher the shear strength. The shear-induced volume change occurs as a result of two competing modes of particle movement, namely, slip-down and roll-over (Dafalias, 1993). Figure 2.3, illustrates vertical displacement of soil measured during direct shear test.

Generally, loose sand exhibit contraction. The slip-down movement causes the reduction in volume by repacking the sand into a denser case. This is the characteristics of loose sand, because of the highest void ratio in loose sand.

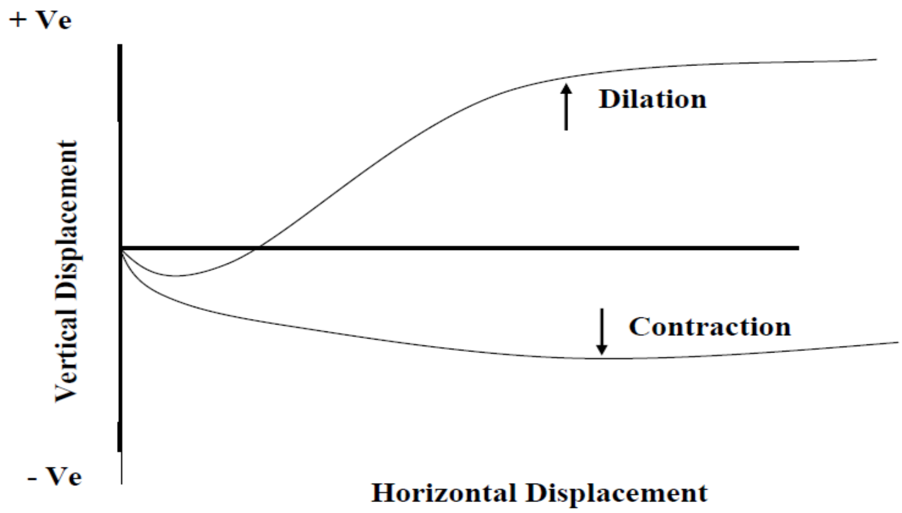


Figure 2.3: Vertical displacement of soil measured during shearing (Dafalias, 1993)

When the slip-down takes place, sand particles are filling gaps in the void due to normal force and there is no chance for sand particles move largely in the direction of shearing. Therefore, the volume reduction or contraction is generally observed at an early stage of loading on sands and the role of slip-down movement is just make the sands to rearrange in the densest state without mobilizing a large amount of shear strain. While dense sand exhibit dilation where the shear stress continuously rises with increasing the shear strain. This is the mechanism of roll-over which tends to increase the volume of sand. A larger movement is always required for particles to roll over neighboring particles and hence the volume increase or dilation is observed. In the later stage of shear stress application, the large deformation is noticed due to the large mobilizing shear strain. Figure 2.4 illustrates the nature of the results of typical direct shear tests in loose, medium, and dense sandy soils (Das, 2008).

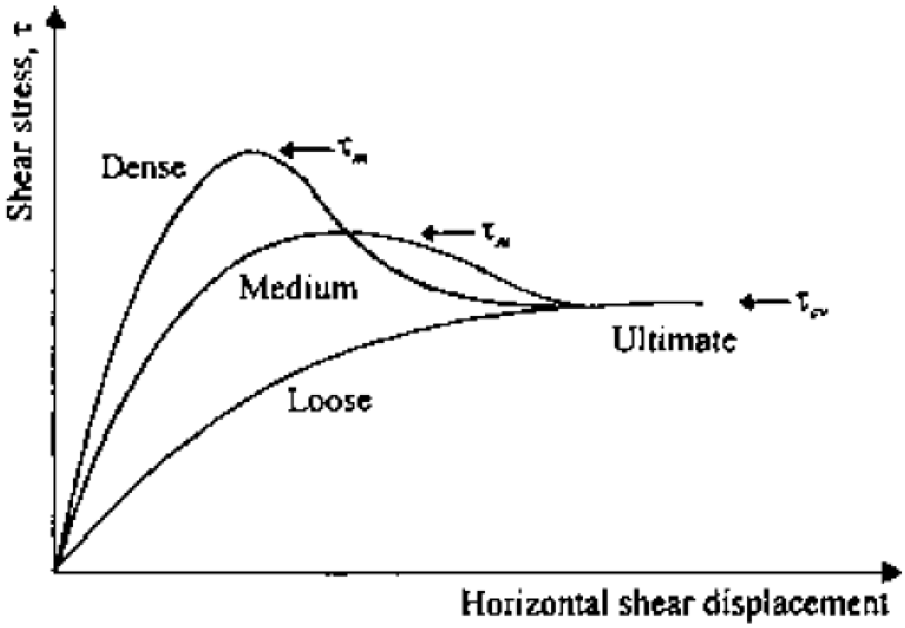


Figure 2.4: Typical direct shear test results of loose, medium, and dense sandy soils. (Das, 2008)

2.1.2 Shear strength of mixed soils

Generally, there are more researches and studies have done on shear strength and stress-strain behavior of granular soils essentially the response of clean sand. Conversely, granular soil may contain appreciable amount of fines (i.e., silt, clay) which puts influences on the engineering behavior of sandy soils. Several characteristics experimentally and theoretically are studied by researchers to observe the behavior of soil mixtures. For example for density, Fragaszy et al (1990), Stovall et al (1996), Thevanayagam, (1998), for strength of soil mixtures Gergiannou et al (1990), Fragaszy et al (1992), Vallejo and Mauby, (2000), for deformability Kumar and Muir, (1996), and for permeability Dunn and Mehuys, (1984).

Georgiannou et al. (1990) observed that the clay content does not significantly decrease the shearing resistance angle of the granular elements when a clay fractions up to 20%. Vallejo and Mawby (2000) reported based on direct shear tests that for fines content less than 25% the shear strength of the sand-clay mixtures would be completely governed by the sand. On the other hand, Olmez (2008) performed direct shear and triaxial tests on consolidated and unconsolidated kaolin- sand mixtures. He founded that the stress-strain relations and shear strength properties of the kaolin-sand mixtures changes significantly for about 20 % kaolin-sand composition.

Thevanayagam (1998) reported, based on undrained triaxial tests, that the shear strength of dense silty sand is approximately the same as that of the host sand at a void ratio equal to es, when the intergranular void ratio es of a dense silty sand specimen less than the emax. of the host sand. While the shear strength of silty sand is low and it depends on the initial confining stress when the es is greater than the emax. of the host sand.

Salgado et al. (2000) performed consolidated drained triaxial compression tests on various fine contents (0, 5, 10, 15, and 20%) of Ottawa sand - silt mixtures. They stated that for both loose and dense specimens, an increase in dilatancy of specimens associated with an increase in silt contents of the mixtures and explained for low silt contents the fabric is mostly accompanied to the contacts between sand grains. Additionally, the silt particle fills the spaces between the neighboring sand particles, this leads to more interlocking between the particles and causing to more dilatancy of soil specimen.

Lade and Yamamuro (1997) performed undrained triaxial compression tests, they found that the presence of non-plastic fines can greatly increase the potential for static liquefaction of clean Nevada and Ottawa sands. That style of behavior directs researchers to enhance the potential for liquefaction by explain the increasing compressibility of the body structures due to fines. On the other hand, in 1998 they stated from drained and undrained triaxial compression tests on Nevada silty sands, that the specimens of greater density was liquefy at lower pressure than looser specimen at higher pressure, thus according to this finding they explained that the void ratio is not enough to describe the behavior of loose silty sand.

Antonio et al. (2009) conducted static, isotropically consolidated drained triaxial compression tests to evaluate the stress-strain and volumetric response of Ottawa sand mixed with non- plastic silt and kaolin clay. They observed increases both the peak and critical-state friction angles of the soil when the non-plastic silt mixes with the Ottawa sand and makes the compression response of specimen to more dilative than of clean sand, while more contractive noticed in a very loose state of soils. On the other hand, both the peak and critical-state friction angles of soil decreases as a result of the addition of kaolin clay, less dilation and more contractive is observed for all states of soil.

2.1.3 Effects of particles shape on shear strength

In general, the particle shape and size reflect the origin and type of material, grain development, transporting agent and depositional environments. One of the major characterization of sand as an individual particle is the shape. Shape characteristics of sand is generally influenced by size and mineralogy of the parent material, but particularly the mechanical and chemical actions play great role in the creation of sand shape over an extended period of time. Waddle (1932) is the first how studied the shape of particles throughout 20th century. He developed procedures for the characterization and to gaining control the shape on manifold scales which are used constantly until today.

Barrett (1980) pointed out the shape of particles through three independent relative scales which are:

- Form; which depict changes in the dimensions of particle.
- Roundness; which depict changes of corners that imposed over the particles.
- Roughness; which depict characterization that imposed over the corners and surfaces of particles.

Figure 2.5 shows the independency of these descriptions mean that one may vary noticeably without acting on the others. Generally as particles angularity increase, the increase will appears in internal friction angle of particles, because of having sharp edges and corners which lead more interlocking during shearing.

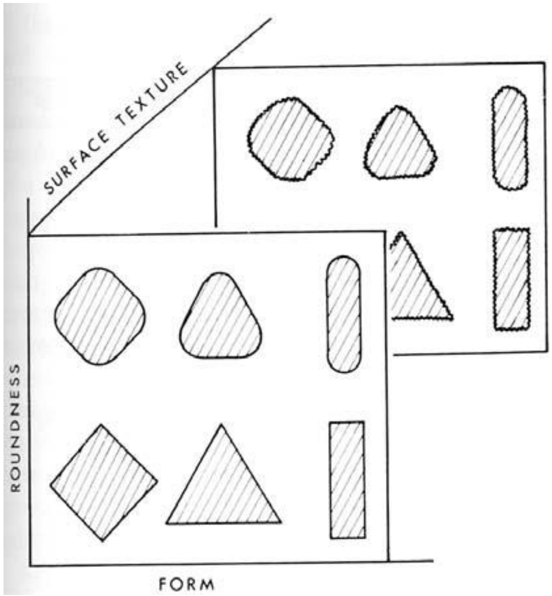


Figure 2.5: Illustrative diagram of independency of form, roundness and roughness (Barrett 1980)

The particles start to filling the voids due to rotation and sliding over each others. In angular particles because of rotational frustration, the localization is less likely to be occur. Therefore a higher strains level with less pronounced drop in shear strength would be occur. Also It is the particle rotation that makes the specimen to more consolidate and the shear stress of the specimen increases more (Dodds, 2003).

Mair et al. (2002) shows that the shear resistance leads to increase, because as the angularity increases the ratio of sliding to rolling contacts increases. Santamarina and Cascante (1998) observed that the rotational frustration is more noticeable in denser packing's. During shear, all the particles are not rotates in complementary trends. Therefore as the total number of particles contact increases, the contact points in a definite direction for resisting rotation would increases. As a result the rotational frustration leads to increase. Figure 2.6 shows the effects of surface roughness of particles and the manner of rotation of particles with each others during shear. Surface roughness of particles leads the particles to roll without gliding, because with the presence of asperities on the surface of particles, this hinder the rolling between the particles and producing greater force which increases the shear resistance. However for smooth surfaces of particle, there is no needs for the force to increase, because the particles roll smoothly , therefore the shear resistance would not increases.

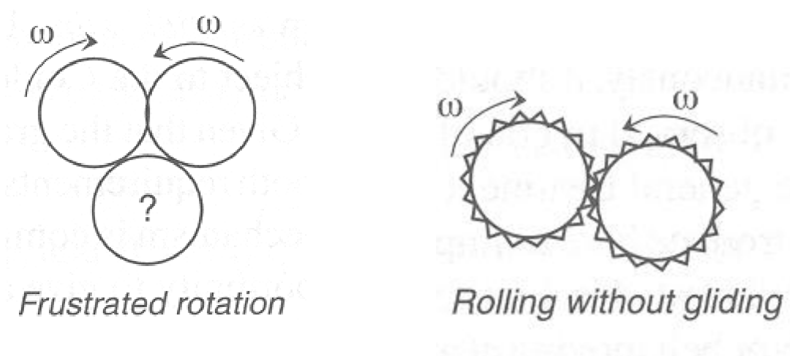


Figure 2.6: Surface roughness and rotational frustration (Duran 2000)

2.1.4 Effects of sand particle size on shear strength

In general, particle size and shape are the products of different natural or artificial physical processes in soil formation. Therefore, the soil grain size distributions are conducted to separate the different particle sizes by performing the process of sieve analysis. The particle size is useful in description, definition and also in selecting the suitable type of soils for geotechnical design and projects such as construction materials roads and backfill materials. The shear strength is one of the most important soil characterizations in many geotechnical designs. The shear strength of granular soil is affected by several various factors like particles shape, water existence, deformation and particle size of soil. The particle size is one of the main parameter which have the great influence on the shear strength and stress- strain behavior of soils.

Soil is classified as coarse grained (gravel and sand) and fined grained (silt and clay). Mostly, the properties of coarse grains soils are controlled by grain size besides the grain size distribution, and occasionally shape of particles. The strength behaviors of soils are effects by both coarseness and fineness of soils. Shear strength and stress- strain of soil are affected depending on the particle size. The coarse particle sizes have shear strength higher than the shear strength of fine grain size, and as the coarseness of particle increases, the internal friction angle will also increase.

Depending on particle size, some classes of soil consist of a wide range of sizes that known as well graded, while others known as gap graded, but soils which consist of a narrow range of particle sizes termed uniform graded. In accordance to these terminologies, the particles may be arrange itself as loose, medium or dense regarding to the manner of packing of materials. Investigations showed that well graded soils would creates a denser packing hence have had a high shear strength in comparison to gap graded soils because the finer particles of well graded soils fills the voids produced by coarse particles, and giving more interlocking inside the material. The particle size in sandy soil is important in the prediction of the angle of internal friction. The angle of internal friction increases as the particle size of sand increases from fine to medium particle sizes, and from medium to coarse particle sizes. The particle sizes also affect the dry density of sandy soils, in the loose state of sandy soils (low dry density) the internal friction angle increases with increasing the percentage of coarse sand. While for the case of soils with high dry densities (medium dense and dense states) by increasing the total percentages of medium and fine particles of sands, the angle of internal friction increases due to insertion of fine and medium particles between coarse particles of sand and makes more occurrence in contact shear surfaces between particles.

2.2 Soil settlements

The soils under structures that build by civil engineers are under loads exerted by overlying structures. The soils beneath these structures are subject to increase stresses producing in strain that leads to settlement of stratum. In general, most settlements in soil is a result of a decrease in the volume of soil mass. In a completely saturated soil system, when the water in the voids are expelled by applied stress then the reduction in the volume takes place due to removal of pore water fluid. Therefore, rearrangement of soil particles in the voids that produced by the outflow of water from the voids.

The rate of the change in volume depends on permeability of soil which is related with the rate at which pore water moves out, and how much the rearrangement and compression which occurs on the soil body. Holtz and Kovacs (1981) states that the rigidity of soil skeleton decides how much the rearrangements and compression takes place on the soil particles.

The total settlements of soil consist of three components:

- Immediate settlement
- Primary consolidation settlement

- Secondary consolidation settlement

Immediate settlement of soil is also called as elastic settlement. Actually soils are not an elastic material but the elastic theory is applied for the calculations in this component of a settlement. Immediate settlement occurs directly after the load is applied on the soil skeleton. There is a constant volume but the occurrence of distortion (change in shape). In low permeability soils, the rates of flow of water are negligible because of the less pervious characteristics. While in high permeability soils, the rate of flow of water is quick at constant volume due to more pervious characteristics.

Primary consolidation settlement occurs as a result of expelled the pore water from the voids of low permeability soil type saturated with water. Here the deformation is due to squeezing of water from the pores leading to rearrangement of soil particles. Therefore it is the time dependent process because the mechanism of expulsion of the water takes a more time due to less pervious of fine grained soils. The pressure dissipation of pore water related with the passage of time for decreasing the rate of water flow. There is no gradual dissipation of pore water pressure during this process but simultaneously an increase occurs in effective stress. So the consolidation settlement occurs at the time at which the water starts to flow out from the pores to the time of ceasing flows from the soil voids. This is also the time at which the excess pore water begins to reduced (increasing effective stress). Therefore, the compression of soil is the time dependent process for equalization the total stress with effective stress.

Secondary consolidation settlement is also called creep settlement. It is the change in volume for a fine grained characteristically soil owing to rearrangement of particles under constant effective stress. The secondary consolidation settlement starts after the completion primary consolidation settlements. In other words after the dissipation of the excess pore water pressure. Secondary compression settlement is a time dependent process, as in a primary consolidation settlement. It occurs in a very slow rate as compared to primary consolidation settlement. In highly organic soils and peats, the secondary consolidation settlement forms the main component part of the total settlement.

2.2.1 Influences of particle shape on consolidation behavior of soils

Consolidation of granular soils is a more complex operation process because of the contribution of multiple mechanisms including particle rearrangement, elastic and plastic distortion, fracturing and crushing. These mechanisms directly linked with the effects of particles shape. Therefore the shape controls the packing structure of particles which could contribute to the difference in stiffness between the particles. The soil structure composed of platy particles commonly the particles orientation and the contact manner of arrangement (face-to-face or edge-to -face) may be give weaker resistance than the structures of rotund particles at the same density due to structure flexibility , and bending and fracturing of platy particles.

The rearrangement of particles is a process of voids filling and producing denser state soils. As angularity of particles increase, the more interlocking between particles would occurs due to owing more open spaces produced by structure arrangement of particles. Therefore, the compressibility of angular particles higher than the rounded particles at the same density. Angular sand have a higher void than rounded sand in the same state of density, because the orientation and contact manner of arrangement in angular sand (edge- to-edge) occur. Therefore, bigger and different shape of voids produced in the packing of angular particles, while smaller and closer shape of voids for rounded shape would occur.

There are some reviews of the literature on the effects of particle shape on the behavior of soils. For example on maximum and minimum void ratios of soil. Edil et al. (1975) reported that maximum and minimum void ratio decrease with increasing roundness. Also the particles shape have a great significance on the change in initial void ratio which leads the soils to more contraction or less during applying loads. For this effect, Nouguier- Lehon et al. (2003) studied the influence of grain shape and angularity in two-dimensional numerical modeling on the behavior of granular material. They stated that the critical state concept for elongated grains is meaningless, as the behavior of samples generated with such particles is highly dependent on the direction of loading with respect to the initial fabric. On the other hand, Clayton and Abireddy (2006) carried out a two- dimensional study to investigate the effects of particle shape on both depositional void ratio and dilation during compressive shear. They observed that particle shape had a controlling effect on initial void ratio, and the platy particles assembles did not dilate.

Lambe and Whitman (1979) showed major differences between the packing and structural arrangements between sand and clay particles. They reported that there is a limit for the maximum volume in a given mass of sand can occupy, as their grains are roughly spherical. However, clays can exist at very high void ratios owing to their plate-like particle shape and structure. While the arrangement of plate-like clay particles in flocculent structure accommodates large voids. On the other hand, they stated that the swell and compression indices of sands are much less than that of clay.

2.2.2 Influences of particle size on consolidation behavior of soils

Consolidation is a time- related process. Since it associated with a water drainage out of the soil voids and producing a densest state of soils. It is generally relates with fine- grained soils such as silty and clayey soils. These types of soils undergo slow rates of consolidation because of their low permeability characteristics. While the coarse grained soils such as sandy and gravelly soils undergo faster rate of consolidation due to the presence of large voids and more open spaces between grains (high permeability). In one dimensional consolidation the soil is confined laterally. Therefore, inevitable case for flowing of water out of the soil vertically (i.e., no consolidation in horizontal directions). As a result, the soil undergoes one dimensional consolidation settlement with dependence on the presence of open voids in the soil skeleton. This mechanism is applied on the size of openings which produced by particle sizes of soil skeleton. The size of openings in soil skeleton is associated with the permeability of the soil. The smaller the openings, the longer time spans for the water to flow out and dissipate its pressure.

The stiffness of soils also affects the amount of its consolidation settlement. The stiffness of soils is related to the formation and mineralogical composition of parent materials. In sandy soils because of the high stiffness allows less compression than silty soils which have lesser stiffness and silty soils undergoes more compression than clayey soils. Therefore, the coarse size of sand particles have lesser compressibility than medium sized sands, and fine grained sand undergoes less compression than silty soils. The elastic deformation occurs especially in granular materials which does create only a small part of the settlements. As a result, there is much less quantity of recoverable settlement when the applied stresses were removed. While, plastic deformations are more prevalent in soft soils, and very much amount of consolidation settlement would be recovered after releasing the applied stress. Therefore, the grain size is one of the much more important characterizations of granular soils which affect the compression behavior. Besides, the mineralogical composition of grains, grain shapes, initial state of relative density and levels of applied stress are the other factors which influencing the compression behavior of sands (Yamamuro et al. 1996; Chuhan et al., 2003).

CHAPTER 3

CONTAMINATED SOILS

3.1 General

Oil, gasoline, petrol, lubricants and diesel fuels are all common compounds in our lives. These chemical substances are the main sources of fuels that run today's complex society. Commonly used properly, sometimes they pose few problems or un- purposely may cause large difficulties like contamination of earth's surface. There are different sources for leakage of these substances such as leakage of oil from damaged pipelines and storage tankers, tanker accidents, discharge of coastal facilities, petroleum production facilities in offshore, spills of underground storage and natural seepage. The spillage of these fluids on the ground surface may cause massive problems to surface and subsurface soils. Generally, contacts of soil with chemical fluids not cause only environmental and ecological problems, it may also have harmful effects and drastic changes on the engineering behavior of soils such as strength, hydraulic conductivity, shear strength, and stress-strain properties.

3.1.1 Influence of pore fluid viscosity on engineering properties of soils

Fluid viscosity is one of the most important physical properties in many fluid products, and can be described as a fluids resistance to flow. Fluids resist the relative motion of immersed objects in them as well as to the layers motion with different velocities within them. Therefore, this property of fluids would alters the pore fluid viscosity which causes a change in soil behaviors.

There are limited number of studies on the effects of different viscosity pore fluids on the behavior of soils, and due to the scanty in identifying some important characteristics of the behavior of geomaterials. However, some authors put in to consideration that studies must be through different ways such as centrifuge tests (Zeng et al, 1998) and soil dynamics (Ellis et al, 2000) as well as fundamental soil behavior (Ratnaweera & Meegoda, 2006).

Zeng et al. (1998) presented a series of permeability test results on two types of sands. They found that using a glycerine-water mixture as the pore fluid has no significant effect on the strength and stress-strain relationship of Ottawa sand No. 40. Coefficients of permeability are inversely proportional to the viscosity. On the other hand, at small hydraulic gradients, it was observed that the highly viscous fluids can lead the clogging of flow through the pores. In fact, the soils may be contaminating by viscous fluids through various sources (e.g., leakage of oil from damaged pipelines). Ratnaweera and Meegoda (2006) studied on unconfined compression tests on fine-grained soils contaminated with varying amounts of chemicals. They showed that the reductions in shear strength and stress-strain behaviors will occur because of the presence of chemical additives, as a result of changes in dielectric constant and pore fluid viscosity caused by the additives. Results of consolidated triaxial test to study the granular soil displayed a similar behavior. This is because of the mechanical interactions at particle contacts, caused by the enhanced lubrication offered by the more viscous, compared to water, pore fluid. While, the reductions in shear strength for fine-grained soil is attributed to physicochemical effects caused by a reduction in dielectric constant, and mechanical interactions caused by higher pore fluid viscosity.

3.1.2 Viscous fluid effects on the behavior of clean sand

Pore fluid viscosity is possibly to make some contribution to the dynamic behavior of sands, such as stiffness, damping, and liquefaction characteristics. Pore fluid viscosity is possibly to make some contribution to the dynamic behavior of sands, such as stiffness, damping, and liquefaction characteristics. During the movement of particles with respect to each other, local viscous loss is likely to occur mainly close to the particle contacts (Ellis et al., 2000). Wilson (1988) studied the effect of pore fluid viscosity on damping in sand, and showed that the change in damping between oil and water-saturated samples increased with applied shear strain. It was interpreted that pore fluid would be forced to flow around the moving soil particles by shear deformation. Evgin and Das (1992) carried out triaxial tests on clean and oil contaminated quartz sand. They found that for both loose and dense sands, full saturation with motor oil causes a significant decrease in the friction angle and also drastic increase in volumetric strain. They also showed by finite element analysis that settlement of footing increased due to oil-contamination. Shine and Das (2001) studied the bearing capacity of unsaturated oil- contaminated sand. They observed that oil contamination drastically reduces the bearing capacity. Cabalar and Clayton (2010) carried out a series of triaxial tests on Leighton Buzzard quartz sand. They used silicon oil, syrup which prepared from the solubility of sugar in water resulting different viscosity syrup. The results shows a sharp decrease followed by a gradual increase in the deviatoric stress, stiffness, and a sharp increase followed by a gradual decrease in corresponding pore pressure.

CHAPTER 4

EXPERIMENTAL STUDY

4.1 Materials and material properties

4.1.1 Sands

The materials used throughout all the research program was sand and sand-clay mixtures. Sands were used during the research programs are of two types. Trakya sand obtained from thrace region in North-west of Turkey, and a commercially produced crushed stone sand obtained from Gaziantep City in Southern-central of Turkey. Trakya Sand, which is commonly used in the experimental works, was supplied by Set/Italcementi Group, Turkey, confirming to TS EN 196- 1. It was obtained from the Thrace Region in North-west of Turkey. Crushed stone sand used is already available in Soil Mechanics Laboratory of Gaziantep University which is widely consuming in civil engineering works, in particular earthworks, in Gaziantep City and its vicinity. It is seen that Trakya sand particles have relatively rounded shape, whereas the crushed stone sand particles have angular shape appearance. Figure 4.1 shows Trakya sand and Crushed stone sands.

Figure 4.1. Pictures of (a) Crushed stone sand, and (b) Trakya sand used during the experimental study

4.1.2 Clays

Clay used in the experimental study represent the finer material, and it is a red color clay quarried in Karatash/Gaziantep. Particle sizes less than 0.002 mm, and it has a plastic limit (PL) value of 23.27, liquid limit (LL) value of 34.75 and specific gravity (G_s) of 2.61.

4.1.3 Fluids

During the conducted tests different viscosity fluids were used. Water, petrol and gasoline, and it was specified that the gasoline is a higher fluids viscosity were used. However, water used in the testing program was taped water, which have had a lower viscosity among fluids were used. Petrol have an intermediate viscosity between gasoline and water.

4.2 Testing program and experimental data

Particle size analysis was determined for sands by performing sieve analysis according to BS 410/ ISO 3310. Both dry and wet sieving were performed on sands so to remove fine particles of clay and other dusty materials and clean sands was obtained, and then sands were dried by oven under temperature of 115 °C. Different particle sizes of sand was selected for this study, there are 1.0 - 2.0 mm, 0.6 - 1.0 mm and 0.3 - 0.6 mm diameters. Clay were prepared by sieving over 0.002mm sieve diameter size.

Specific gravity (G_s) of sands and clay were obtained according to ASTM D 854. Plastic limit (PL) and liquid limit (LL) for clay were found in accordance to ASTM D 4318. Throughout the testing program for determination of specific gravity and index properties for both sands and clay, distilled water was used. Maximum and minimum void ratio for dry clean sands were determined by pluviation of dry sands into a mould that has a prespecified volume. Summary of the results of index and some other properties of materials were presented in Table 4.1.

Table 4.1 Some properties of sands and clay used in experimental works

Materials / Properties	Sands		Clay
	Crushed stone sand (AS: angular sand)	Trakya sand (RS: rounded sand)	
Particle size (mm)	1.0- 2.0	1.0- 2.0	<0.002
	0.3- 0.6	0.3- 0.6	
Maximum void ratio, e_{max}	1.1	1.0	-
Specific gravity, G_s	2.68	2.65	2.61
Liquid limit, LL (%)	-	-	34.75
Plastic limit, PL (%)	-	-	23.27

4.3 Testing program and sample preparation

4.3.1 Testing program and sample preparation for direct shear tests

Clean Trakya sand (RS: rounded sand) and Crushed stone sand (AS: angular sand) specimens were prepared in the experimental work to investigate the effects of various shape and size of sand particles, as well as to inspect the effects of various viscosity pore fluids on engineering behavior of sands saturated with water, petrol and gasoline. All the specimens of sands were tested with different viscosity pore fluids, and in loose states of densities. It was also specified that the initial relative density of all specimens fell between 20 and 35%, with most specimens having a relative density in the region of 25- 35%. The tests was performed at an occupied room temperature ranges between 18 °C and 21 °C, to eliminate the effects of temperature change on the viscosity of fluids. Initial void ratio (e_0), maximum and minimum void ratios (e_{max}, e_{min}), and relative densities (Dr %) of the clean sand specimens tested in direct shear were presented in Table 4.2.

Table 4.2 Some index properties of sands prepared for direct shear testing

Sand Type	Particle size (mm)	e_o	e_{max}	e_{min}	Dr (%)
Crushed stone sand (AS: Angular Sand)	1.0 – 2.0	0.929	1.1	0.568	25.5
	0.3 – 0.6	0.864	1.0	0.646	24.3
Trakya Sand (RS: Rounded Sand)	1.0 – 2.0	0.870	0.99	0.586	30.0
	0.3 – 0.6	0.935	0.97	0.676	31.1

Homogeneously mixed soils were prepared from sands mixed with various percentages of clay in order to investigate the effects of fines clay on engineering behavior of sand – clay mixtures. Mixtures were obtained from various percentages of 5% up to 30% of fines, and percentages of fines during testing program refers to the ratio of dry clay weight to total dry weight of mixture. All the prepared mixtures of specimens were tested in loose states and the relative densities were calculated by obtaining the maximum and minimum void ratios for different mixtures in accordance with ASTM D 1557. Maximum and minimum void ratios were obtained for mixtures by pluviation of dry mixture in to a mould that has a prespecified volume. Clay content, resulted fines contents and relative densities for particular mixtures were summarized in Table 4.3 and Table 4.4.

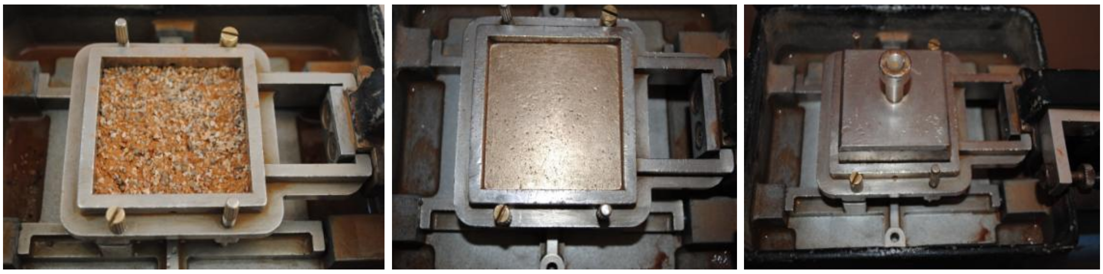
Table 4.3 Initial condition for Crushed stone sands with fines content, AS specimens

Clay content CL (%)	Fines content (g)	Dr (%) of mixture 1.0 – 2.0 mm	Fines content (g)	Dr (%) of mixture 0.3 – 0.6 mm
0	0	26.3	0	33.6
5	6.5	22.6	2.65	16.8
10	13	15.5	5.3	12.54
15	19.5	0.26	7.95	-
20	23	-	10.6	-
25	26	-	13.25	-
30	32.5	-	15.9	-

Table 4.4 Initial condition for Trakya sands with fines content, RS specimens

Clay content CL (%)	Fines content (g)	Dr (%) of mixture 1.0 – 2.0 mm	Fines content (g)	Dr (%) of mixture 0.3 – 0.6 mm
0	0	23.3	0	29
5	6.5	22.0	2.65	14.2
10	13	12.4	5.3	11.8
15	19.5	-	7.95	-
20	23	-	10.6	-
25	26	-	13.25	-
30	32.5	-	15.9	-

‘Direct shear’ (TS 1900 - 22) testing apparatus were employed during the investigation. During the tests performed in direct shear machine, dry sands were gently spooned in thin layers into box, and the box assembled with instruments (i.e., screws, top porous stone). Specimen height inside the box was measured, area correction was made, and the two vertical screws were removed after loading frame employed. Then, the shear box mould was completely filled with the required fluid and kept for a period of time for saturation. Similarly for sand – clay mixtures, the mixed soils gently spooned into the box to have a loose specimen. All mixed soil specimens in direct shear was tested with only water as a pore fluid. Figure 4.2 shows sample preparation for direct shear tests.



Step≠1 Step≠2 Step≠3

Figurer 4.2 Steps of sample preparation for direct shear tests

After the sample preparation process is completed, and the vertical load was applied, rate of shearing was commenced with a 0.1 mm/min. Shear force, horizontal and vertical displacements were recorded at every 50 seconds, until the specimen fails. In the direct shear tests, all specimens of sands and sand- clay mixtures were sheared under different vertical loads (27.24, 54.48 and 108.96 kN/m^2). Figure 4.3 shows the equipment used during the study.

Figure 4.3 Direct shear test machine used during the experimental works

4.3.2 Testing program and sample preparation for oedometer tests

Oedometer (ASTM D3080) testing apparatus were employed during the testing investigation of sand and sand – clay mixtures. During the tests performed in oedometer testing machine the samples were tested in 7.5 cm ring diameter, and 2 cm height for those specimens of sands sized (1.0 – 2.0 mm) particle diameters, and specimens of sands sized (1.0 – 2.0 mm) particle diameters were tested by oedometer mould of 5 cm ring diameter and 2 cm height (Figure 4.4). During the tests performed in oedometer testing machine, dry sands or dry sand – clay mixtures were gently spooned in oedometer cell to have a loose specimens.

Figure 4.4 A view of loose mixed soil prepared for oedometer test

After assembling the instruments (e.g., screws, porous stone, loading frame), and making the measurements of dimensions, specimens were left for a while for fully saturation with the desired fluid (Figure 4.5). Then, loadings were commenced with the effective stress increments of 22.195, 44.354, 88.78, 177.56, 355.12, and 710.24 kPa.

Figure 4.5 Oedometer testing machines used during the experimental works

CHAPTER 5

RESULTS AND DISCUSSION

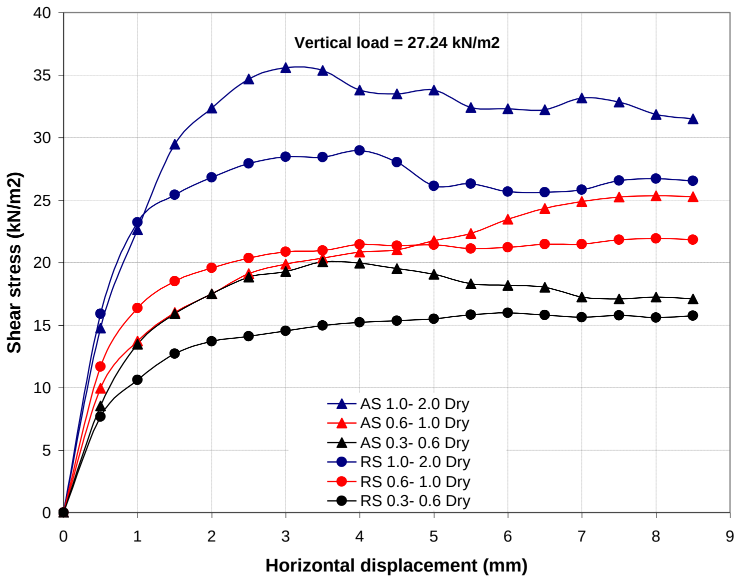
5.1 Direct Shear Tests

Variation in maximum shear strength and angle of internal friction of dry sands were measured at 27.24, 54.58, and 108.96 kN/m^2 vertically normal stresses and a constant shear rate 0.1 mm/min. in a direct shear box to inspect the role of different size and shape of sandy soils. The role of shape can be expressed by means of angularity, roundness and surface roughness of sand particles in conjugation with particle size diameters. The effect of initial condition of specimens was also expected to affect the soil behaviors.

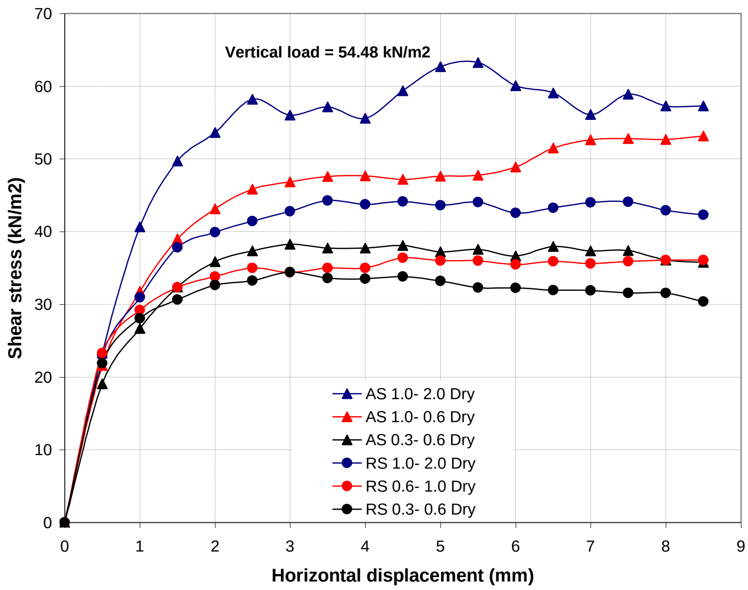
5.1.1 Variation of shear strength and angle of internal friction in dry sands

Variation in maximum shear strength and angle of internal friction values under different normal loads for loose state of sand specimens is presented in Figure 5.1. a, b, and c, respectively.

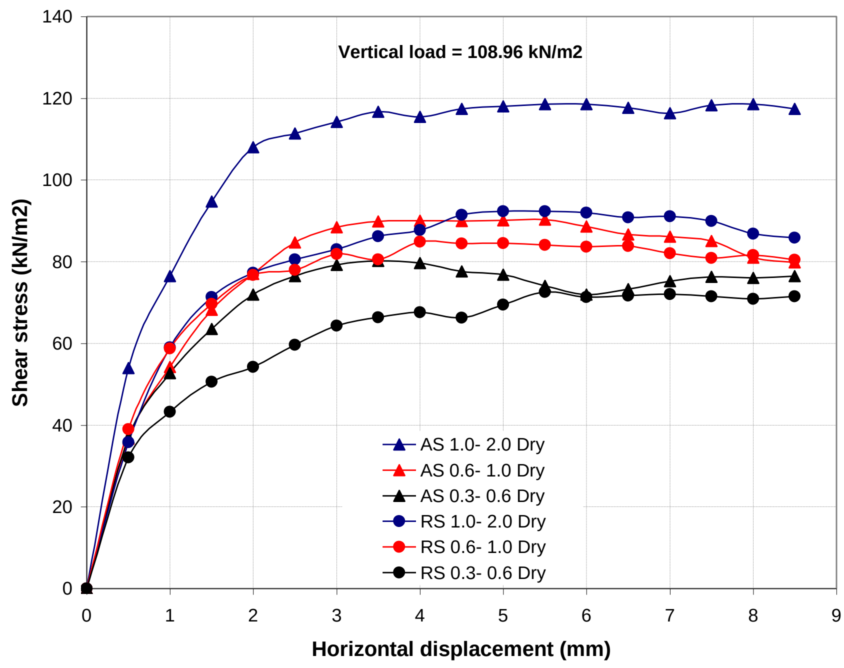
As can be seen in Table 4.2 that the initial void ratios scattered between 0.864 and 0.935 because of particle size and initial conditions, and also relative densities scattered between 24% and 31% which indicate that the specimens are in a loose density state. Figure 5.1 shows the influence of particle size and shape on maximum shear strength of sand specimens. It was observed that as size of sand particle increases, the maximum shear strength and angle of internal frictions increases. Also, angular sand specimens, gives higher shear strength and internal friction than rounded sand specimens, for all particle sizes of both angular and rounded sandy soils.



(a)



(b)



(c)

Figure 5.1 Shear strength variation under different normal stresses a) 27.24 b) 54.48 and c) 108.96 kN/m^2

As can be seen from Table 5.1 and Figure 5.2 that the internal friction angle for angular sand specimens (i.e., Crushed stone sand AS) decrease when the particle sizes decrease because of the decreasing the surface area of friction between the particles during shearing as a result of decreasing particles angularity. Also smaller particle size have a lesser stiffness than greater particle sizes (i.e., soft shearing prevalence between sand particles). However, rounded sands (i.e., Trakya Sand RS) gives lower angle of internal friction than crushed stone sand, because of no sharp edges and corners, and the rolling mechanism of particles movement over each others during shearing. It is also observed that decreases in particle sizes decreased the internal friction of soil but the ratio of decrease not reached to that occurred in angular sands. Table 5.2 and Figure 5.3 shows the alteration of internal friction angle for rounded sands (RS).

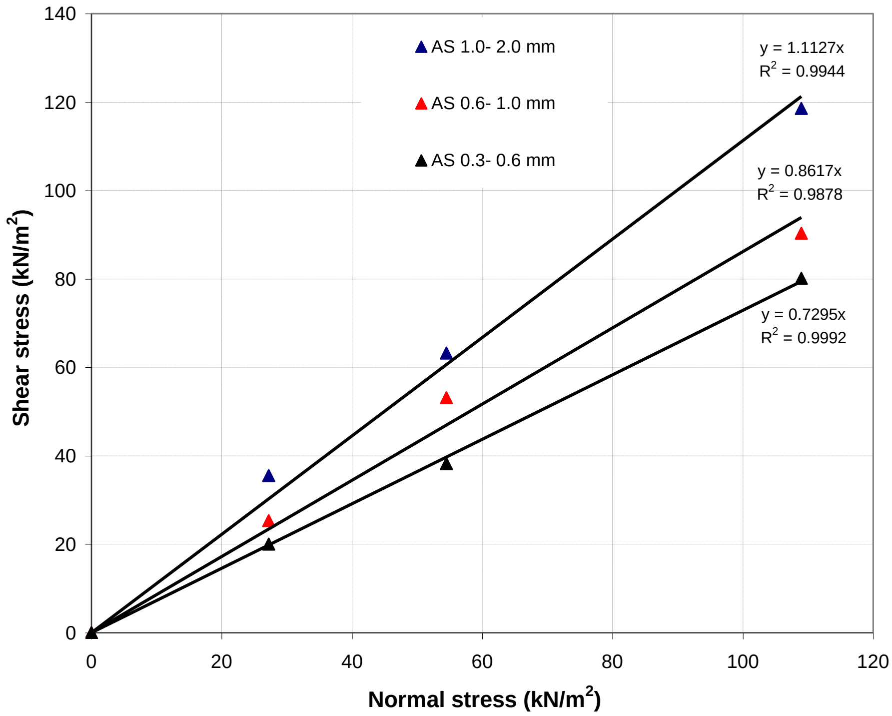


Figure 5.2 Variation of internal friction angles for different particle sizes of crushed stone sands

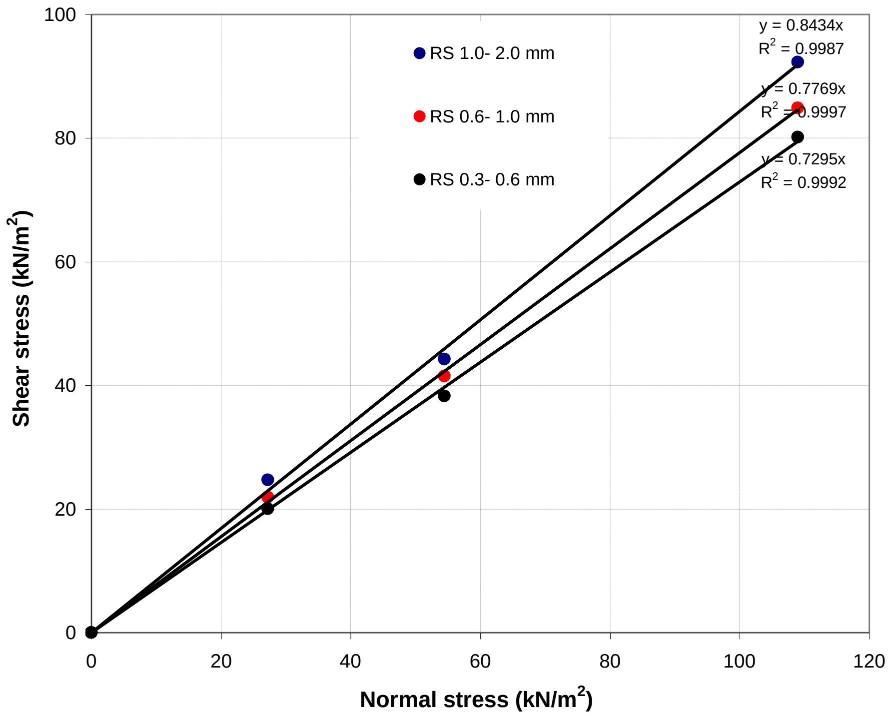


Figure 5.3 Variation of internal friction angles for different particle sizes of rounded sands (Trakya Sand)

Table 5.1 Summary of AS specimens in dry case at various normal loads

Maximum shear strength (kN/m^2)			Friction (ϕ^o)		
Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm	Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm
35.58	25.23	20.06	47.98^o	40.7^o	36^o
63.26	53.16	38.28			
118.6	90.31	80.17			

Table 5.2 Summary of RS specimens in dry case at various normal loads

Maximum shear strength (kN/m^2)			Friction (ϕ^o)		
Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm	Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm
24.7	21.92	15.96	40.1^o	37.6^o	36^o
44.27	41.5	34.42			
92.3	84.87	72.56			

5.1.2 Shear strength variation of sands saturated with different viscosity pore fluids.

For the inspections of shear strength of sandy soils, both AS and RS specimens of sand were tested, and specimens were loose. Figure 5.4 shows the variation of shear stress and horizontal displacement. The size of sand and type of pore fluids in AS specimens were tested under a normal stress of 27.24 kN/m^2. Similarly, Figure 5.5 presents the relationships between the stress and horizontal displacement for different particle size of RS specimens with different pore fluids. It can be seen that the shear stress values decrease as the viscosity of pore fluid increases for all sizes of both AS and RS specimens tested. Amount of decrease in shear stress values in AS specimens seems to be higher than those in RS specimens. It is interpreted that the difference in behavior between the AS and RS specimens may be attributed to the slipping of the rounded particles over each others, and interlocking of the angular particles. It can be observed that the amount of decrease in shear stress value in RS specimens are not reached to the amount of reduction noticed in AS specimens because of less interlocking, and also smooth mechanical movements of rounded particle over each others during shearing for all fluids used.

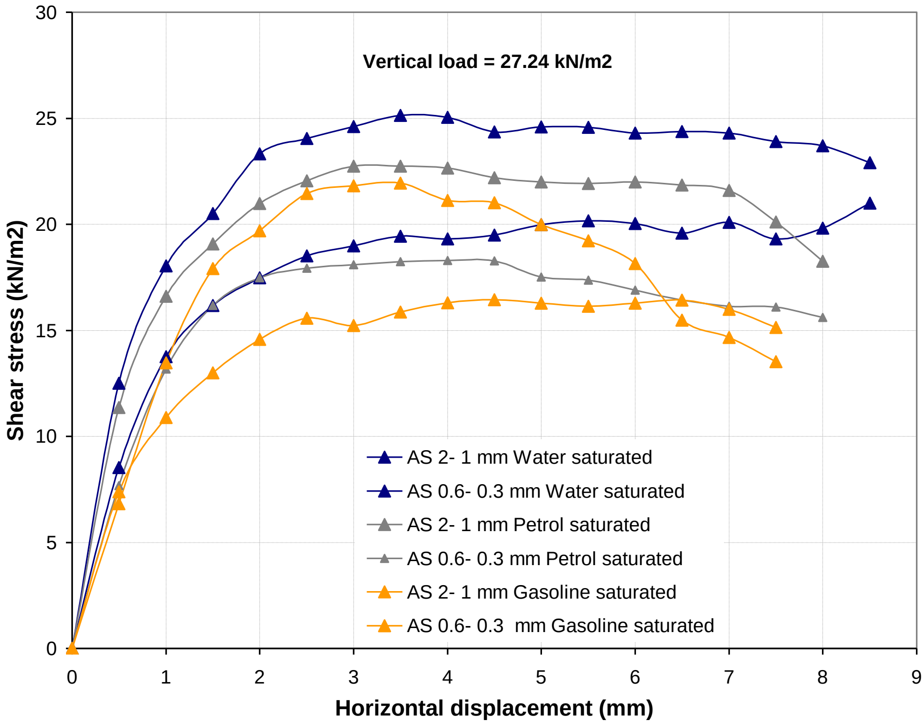


Figure 5.4 Shear stress versus horizontal displacement for different particle size of AS specimens with different pore fluids

Table 5.3 and 5.4 give the summary for different sizes of Crushed stone sands (AS specimens) and Trakya sands (RS specimens) saturated with different viscosity pore fluids. As can be seen that the shear strength parameter ϕ of the sands noticeably decreases as the pore fluid viscosity increases because of the viscosity of medium, and the chemically constituents of fluids which give lubrication to particle surfaces. As a result it simplifies the slippage of particles and a reduction in internal friction angle occurs. A more decrease in the internal friction angle was observed for sands having rounded shaped particles especially sands saturated with petrol and gasoline, because these fluids despite of coating surfaces , also it clogs to the reduction of voids ratio that occurs due to particles interlocking within the specimens. It means, not allowed more interlocking of particles and then decreasing friction angle.

Table 5.3 Summary of AS specimens data

Pore fluid	Maximum shear strength (kN/m^2)			Friction (ϕ^o)		
	Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm	Particle size 1.0- 2.0 mm	Particle size 0.6- 1.0 mm	Particle size 0.3- 0.6 mm
Water	27.54 49.95 104.89	23.22 48.05 98.61	21.63 42.11 86.79	43.71^o	41.9^o	38.37^o
Petrol	24.27 54.58 103.8	22.3 46.12 94.21	18.29 41.73 83.16	43.8^o	40.66^o	37.2^o
Gasoline	21.82 47.95 96.14	15.78 31.56 62.21	16.44 35.97 82.69	41.2^o	37.07^o	36.2^o

Table 5.4 Summary of RS specimens data

Pore fluid	Maximum shear strength (kN/m^2)			Friction (ϕ^o)		
	particle size 1.0- 2.0 mm	particle size 0.6- 1.0 mm	particle size 0.3- 0.6 mm	Particle Size 1.0- 2.0 mm	particle Size 0.6- 1.0 mm	Particle Size 0.3- 0.6 mm
Water	21.16 48.65 86.88	19.51 41.22 78.46	16.9 35.75 75.35	39.79^o	36^o	34.27^o
Petrol	19.15 41.45 73.64	16.64 34.85 66.57	16.01 37.03 54.67	34.73^o	31.64^o	28.2^o
Gasoline	17.05 33.19 67.6	15.78 31.56 62.21	10.58 28.78 58.30	31.73^o	29.8^o	27.4^o

Figure 5.5 presents the relationship between shear stress and horizontal displacement for different particle sizes of RS specimens with different viscosity pore fluids under a normal stress of 27.24 kN/m^2.

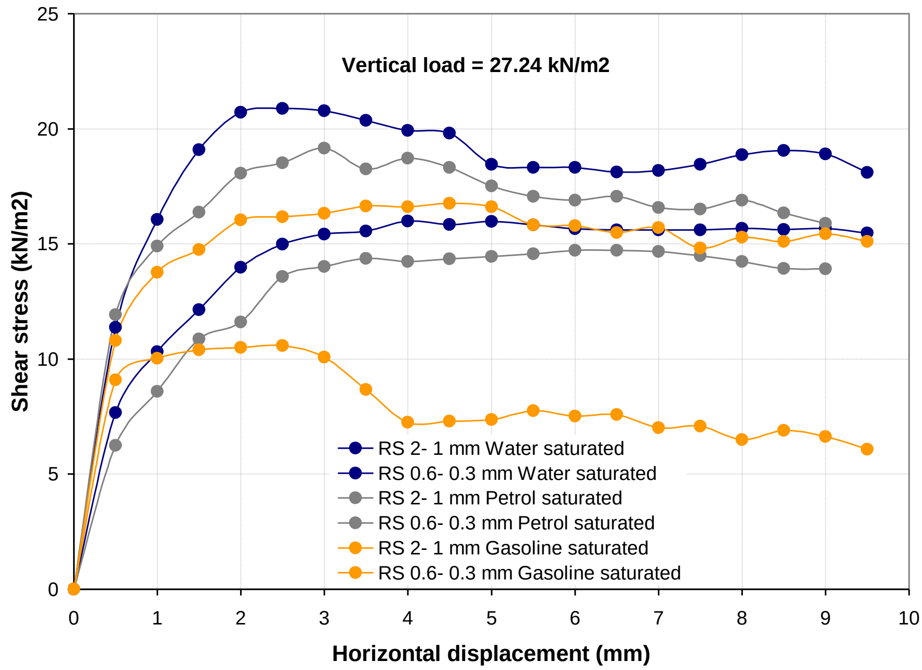


Figure 5.5 Shear stress versus horizontal displacement for different particle size of RS specimens with different pore fluids

5.1.3 Variation of shear strength for different size and shape of sands mixed with clay in water saturated case

During this research, mixed soils were studied to investigate the effects of fines content on strength properties of sandy soils. Interaction between finer and coarser grain matrices, and influence of this interaction on shear strength behaviors of sand-clay mixtures were also investigated. For this investigation different size and shapes of sands with various fine contents were prepared, and a series of direct shear tests was performed. Fines content (clay: CL) does not significantly decrease the shear resistance and angle of internal friction of granular elements when fines content fractions up to 20% (

Georgiannou et al. 1990) The shear strength of sand- clay mixtures would be completely governed by the sands when fines content less than 25% (Vallejo and Mawby, 2000). Shear strength properties for different shape and size of sands mixed homogeneously with clay at a specified fractions which are 5%, 10%, 15%, 20%, 25% and 30% were tested in a fully saturated with water and under different normal stress of (27.24, 54.48 and 108.96 kN/m^2). Shear stress variations and particle sizes of both AS specimens (Crushed stone sands) and RS specimens (Trakya sands) from Figure 5.6 to 5.6.6 representing shear stress and horizontal displacement relations of angular and rounded sands sized (1.0- 2.0 mm) diameters, with different clay contents.

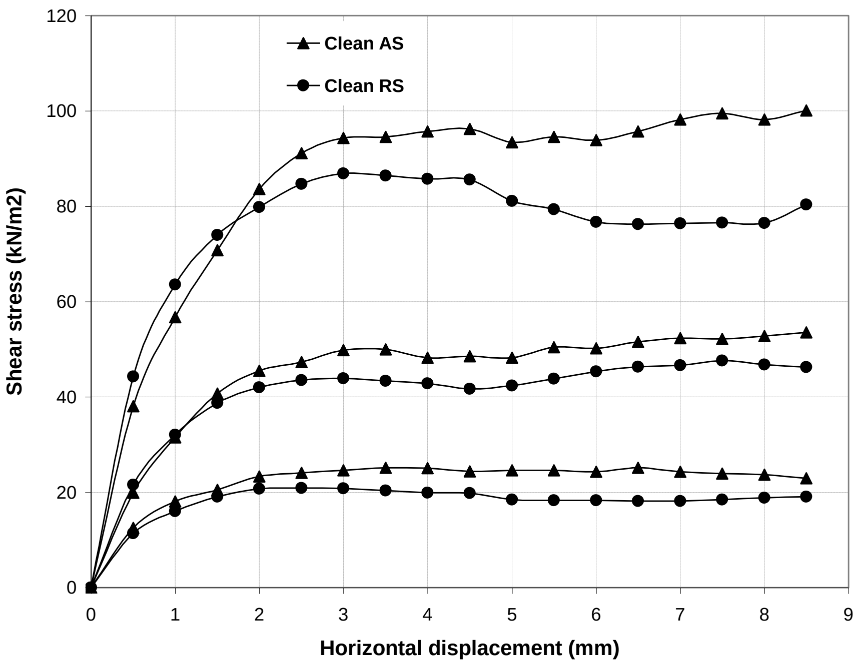


Figure 5.6 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

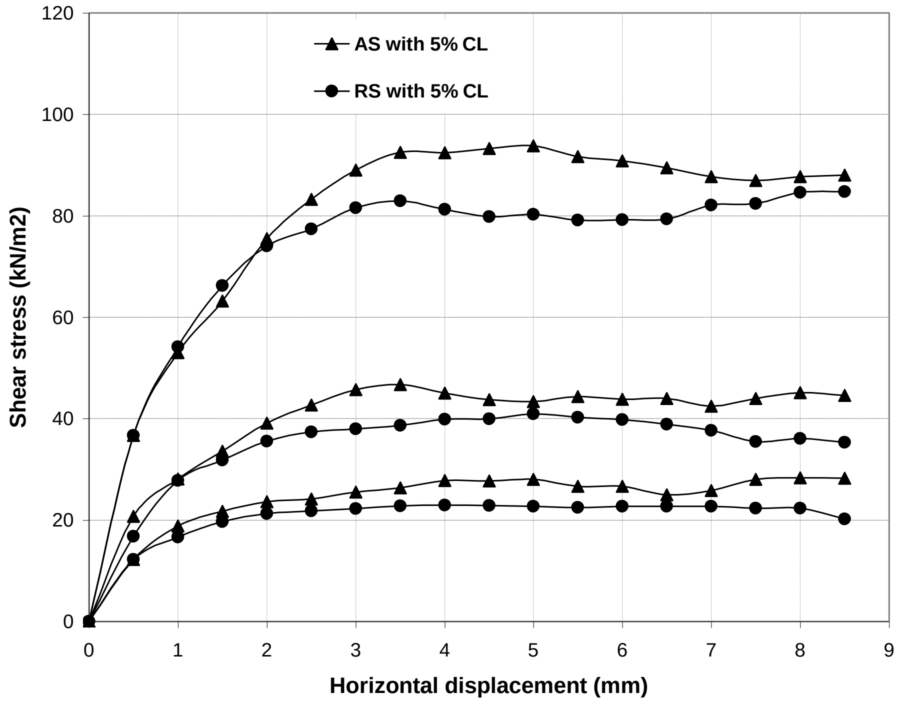


Figure 5.6.1 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 5% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

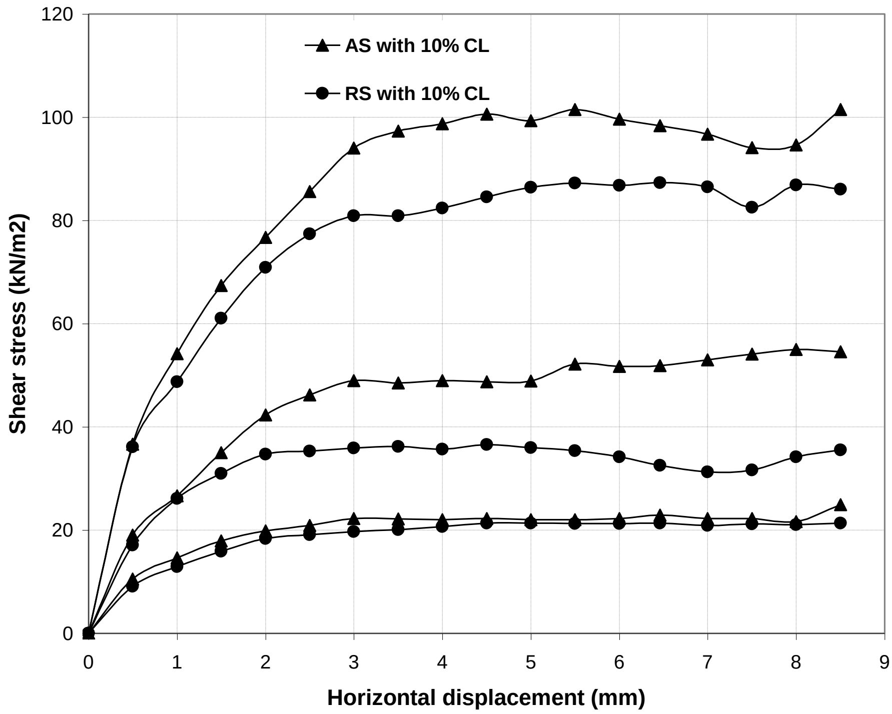


Figure 5.6.2 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 10% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

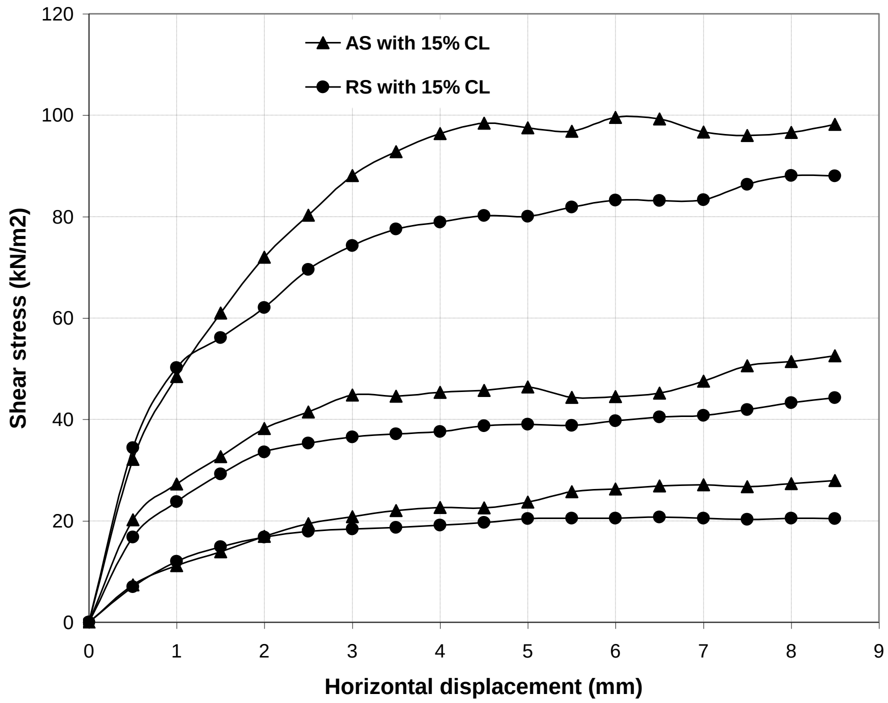


Figure 5.6.3 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 15% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

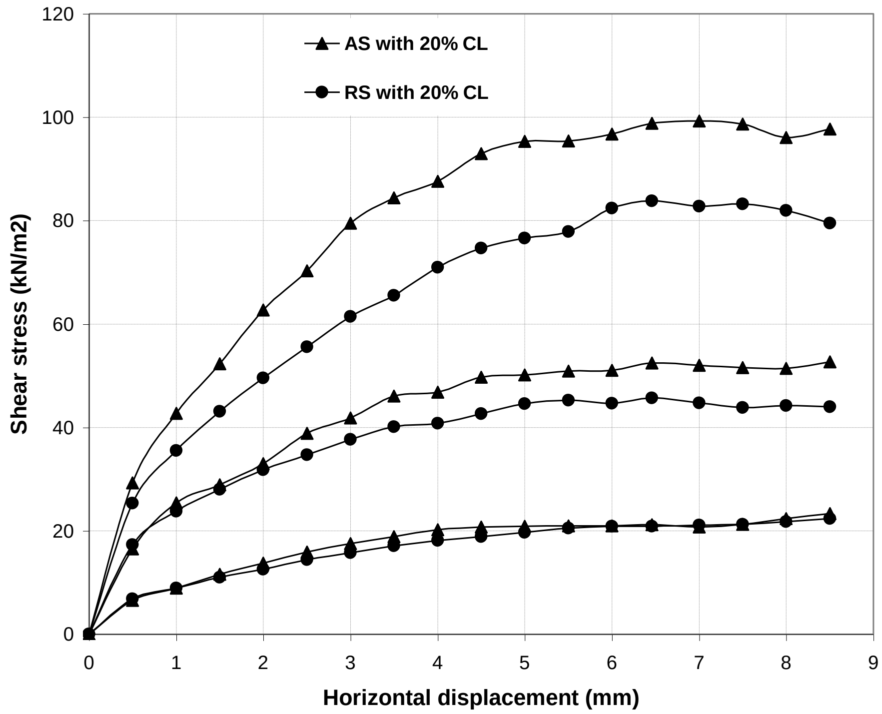


Figure 5.6.4 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 20% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

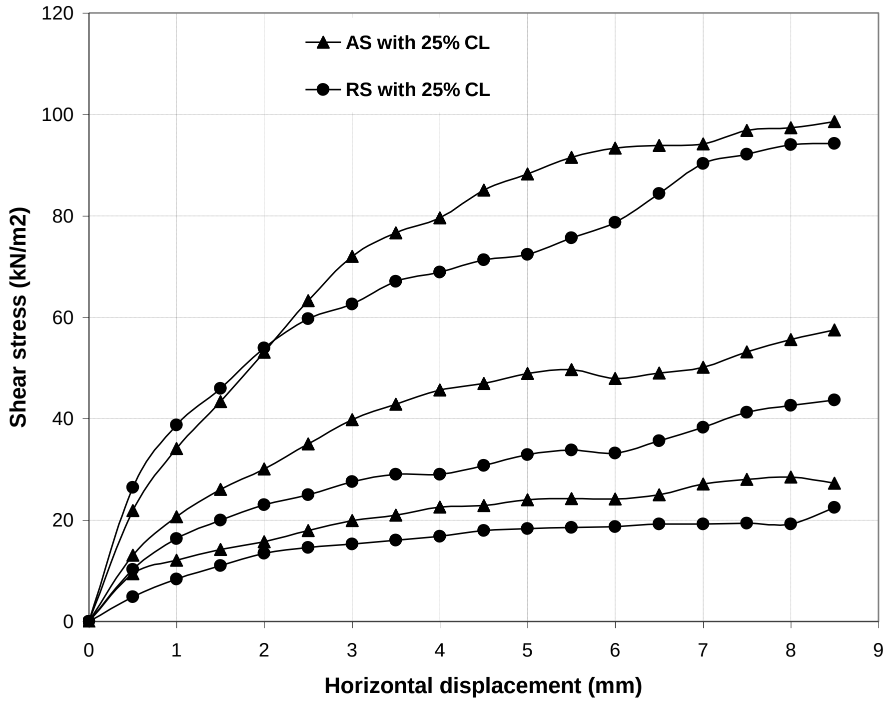


Figure 5.6.5 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 25% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

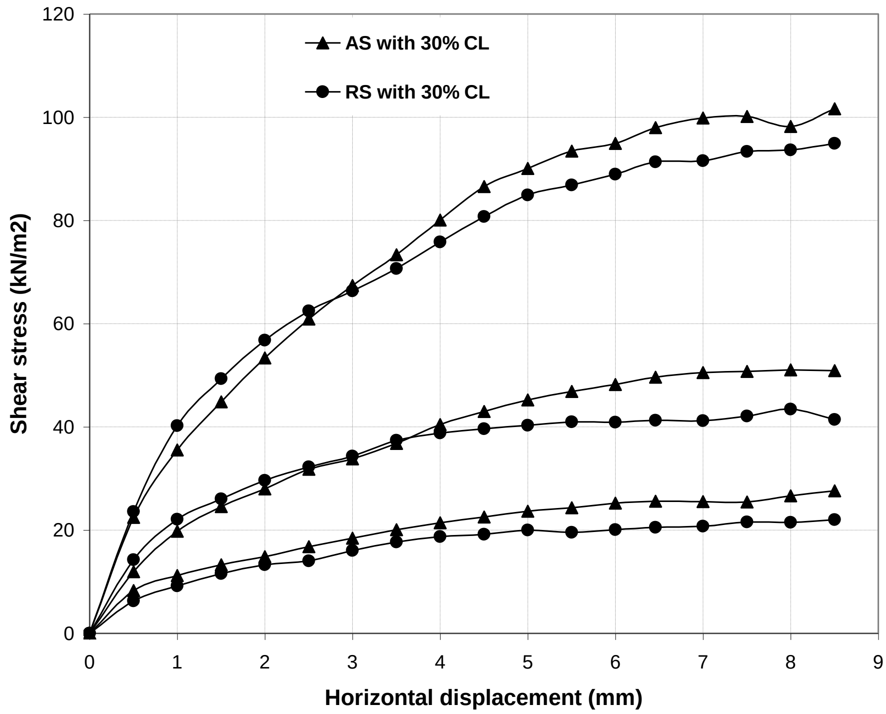


Figure 5.6.6 Shear stress versus horizontal displacement for sands (1.0- 2.0 mm) with 30% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

Figure 5.6 shows the applied shear load against horizontal displacement for both angular and rounded clean sands. All the specimens of AS and RS were prepared in a loose case in water, and tested under different applied shear loads. It was observed that by adding more clay more ductile behavior appeared in specimens, therefore the shear stress values in the specimens of high clay contents decreases much gentler than those with less clay contents. Figures 5.6.1 to 5.6.6 are representing the effects of clay content on shear stress and horizontal displacement relations for different percentages of 5% up to 30% clay contents. It is clear that for angular sand specimens and angular sands mixed with different percentages of clay the shear stress values are greater than the shear stress values of rounded sand specimens and sands mixed with different percentages of clay. It means that the effects of clay content on the shear stress and angle of internal friction is same for both AS (Crushed stone sands) and RS (Trakya sands). Therefore, the main cause which makes the shear stress of AS specimens greater than RS specimens is the role of sand particle shapes.

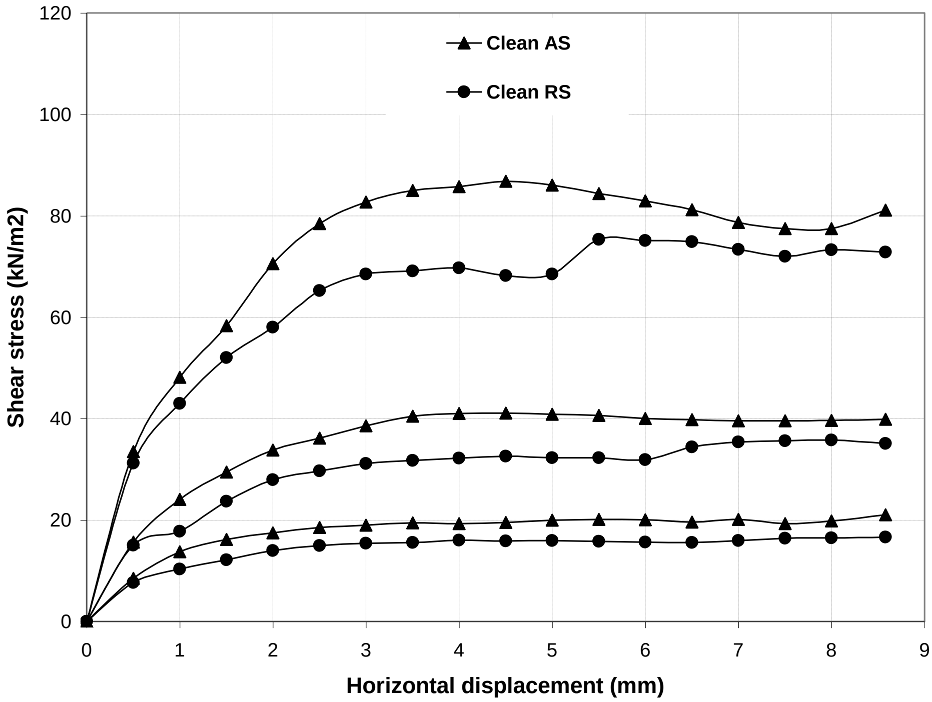


Figure 5.7 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

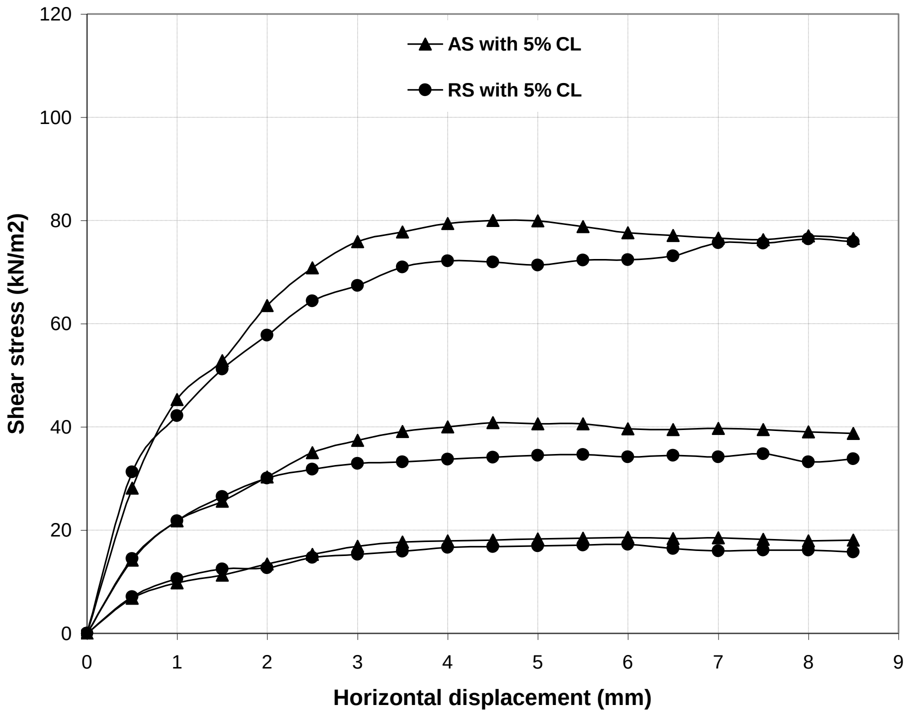


Figure 5.7.1 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 5% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

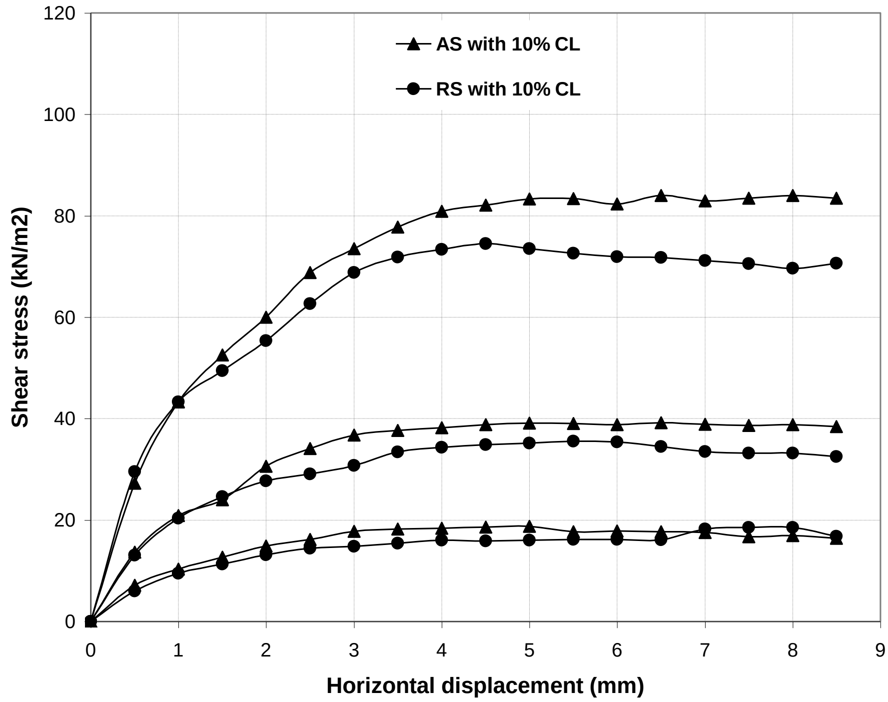


Figure 5.7.2 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 10% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

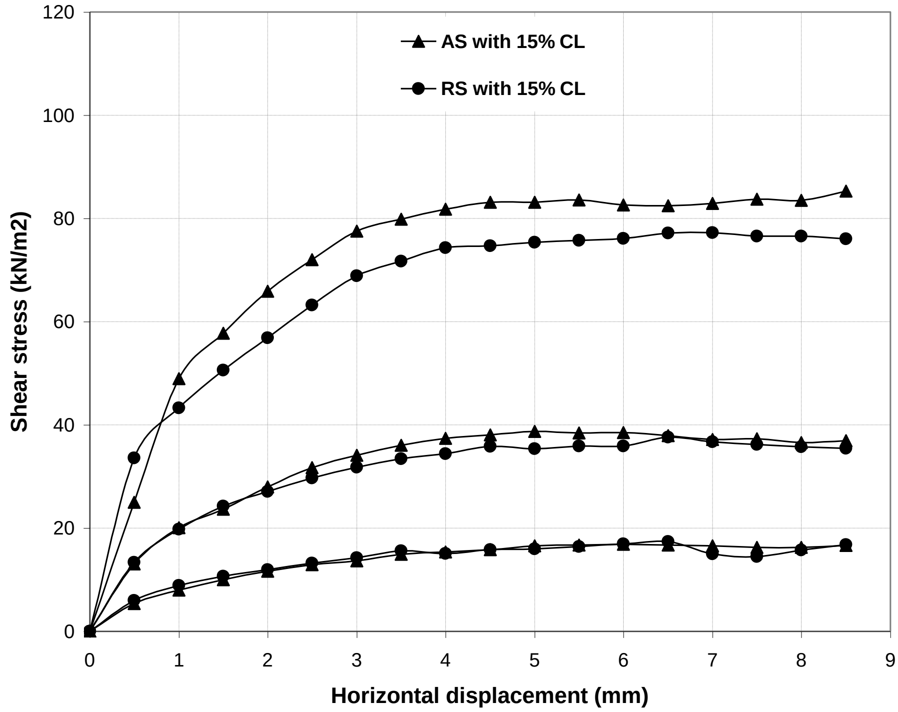


Figure 5.7.3 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 15% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

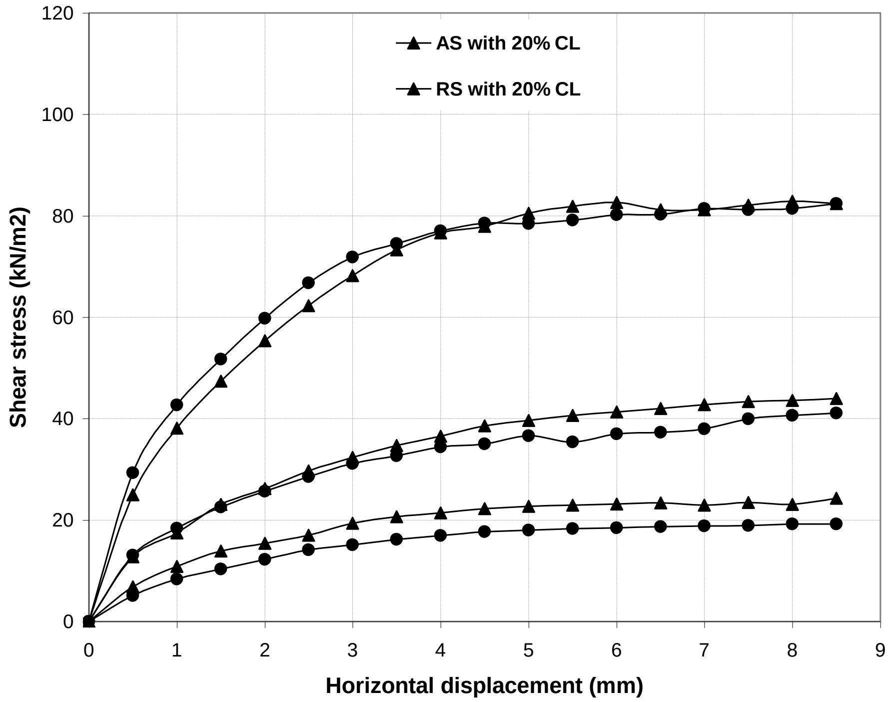


Figure 5.7.4 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 20% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

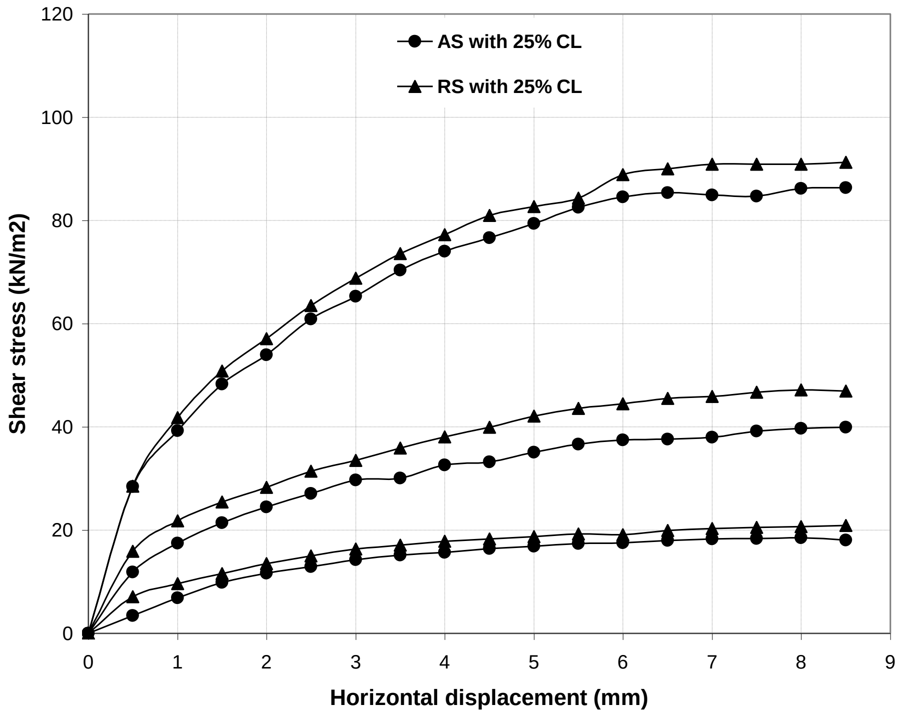


Figure 5.7.5 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 25% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

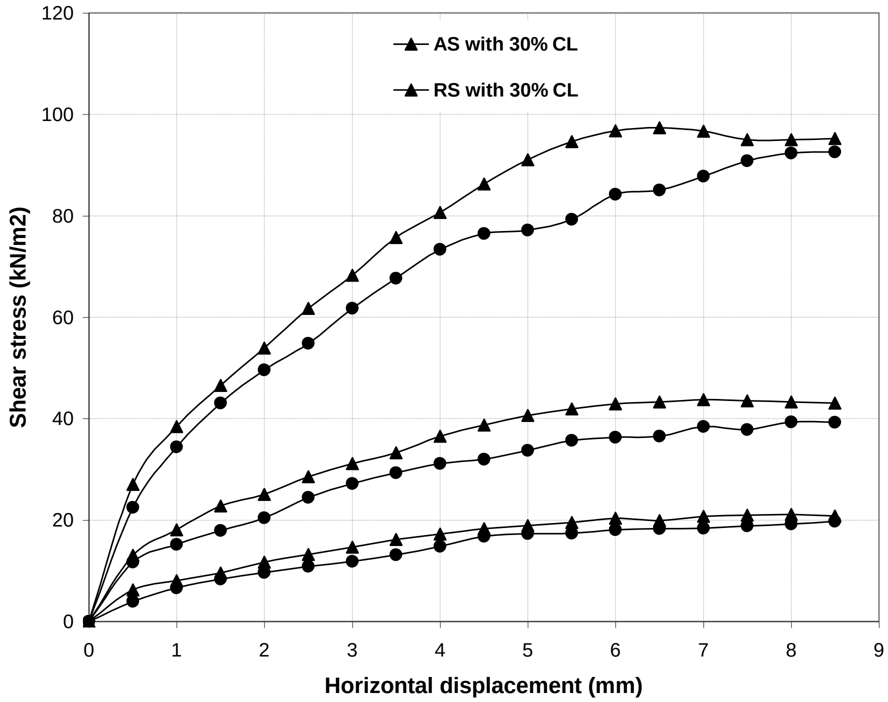


Figure 5.7.6 Shear stress versus horizontal displacement for sands (0.3 – 0.6 mm) with 30% clay content saturated with water under different normal stresses (27.24, 54.48 and 108.96 kN/m^2)

An idealized curve were made for illustrating the effects of increasing clay content on decreasing the shear stress of sand- clay mixtures in Figure 5.8.

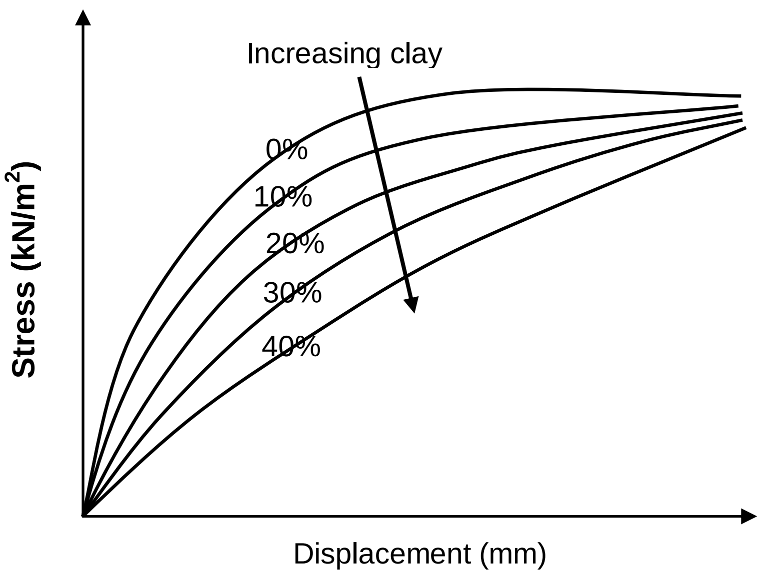


Figure 5.8 Idealized curve illustrating effects of increasing clay content

5.2 Oedometer tests

In this study, one dimensional compression tests were performed on sands and sands mixed with different fines content. Sands were tested in a loose state saturated with different viscous fluids, and under different effective stresses applied. Effects of pore fluid viscosity on compression behavior of sands were inspected. In addition the effects of various size and shapes of sand particles were investigated to understand the role of interaction of particle size and particle shape of sands with various fluids on compressibility behaviors of clean sand. Similarly, mixed soils of sands with different proportions of clay were prepared in a loose state and tested under the same effective stresses that applied on clean sands. The mixed soils were tested with water and gasoline as pore fluids to understand the effects of the changing viscosity of the medium on consolidation behaviors of sands with clay. Also to investigate the effects of viscous fluids on the interaction between coarse and fine grain matrixes during compression, and the effects of various sizes and shapes of coarse particles were investigated on this interaction.

5.2.1 Variation of global void ratio for sands with various viscosity pore fluids.

Variation of global void ratio for different particle sizes of crushed stone sands (AS) and Trakya sands (RS) with various pore fluids viscosity are presented in Figure 5.9 and 5.10 respectively.

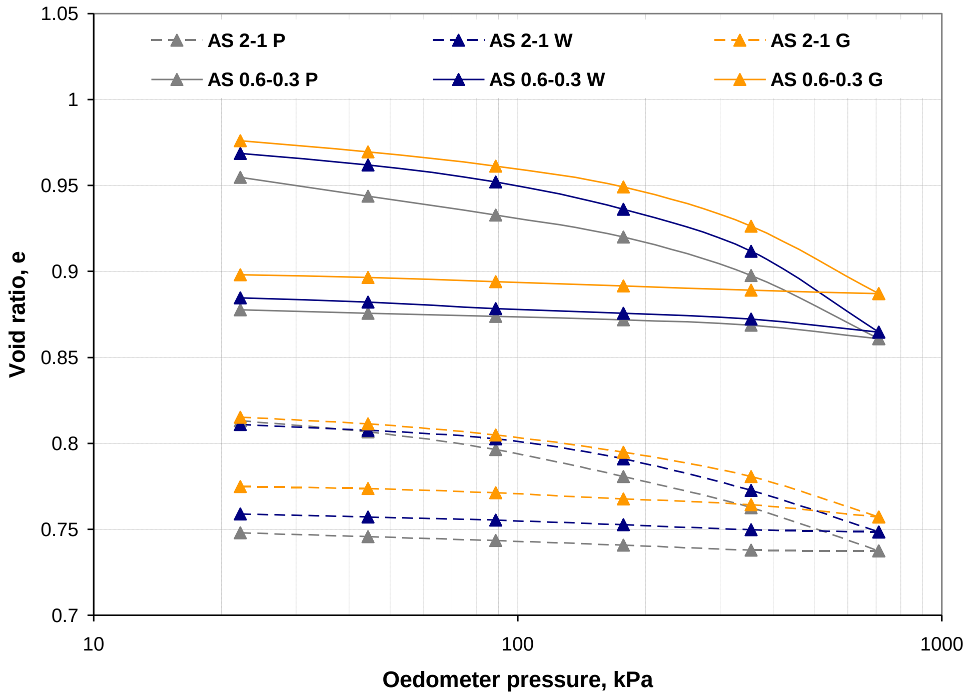


Figure 5.9 Variation of global void ratio for different particle size of angular sand specimens saturated with different viscosity pore fluids

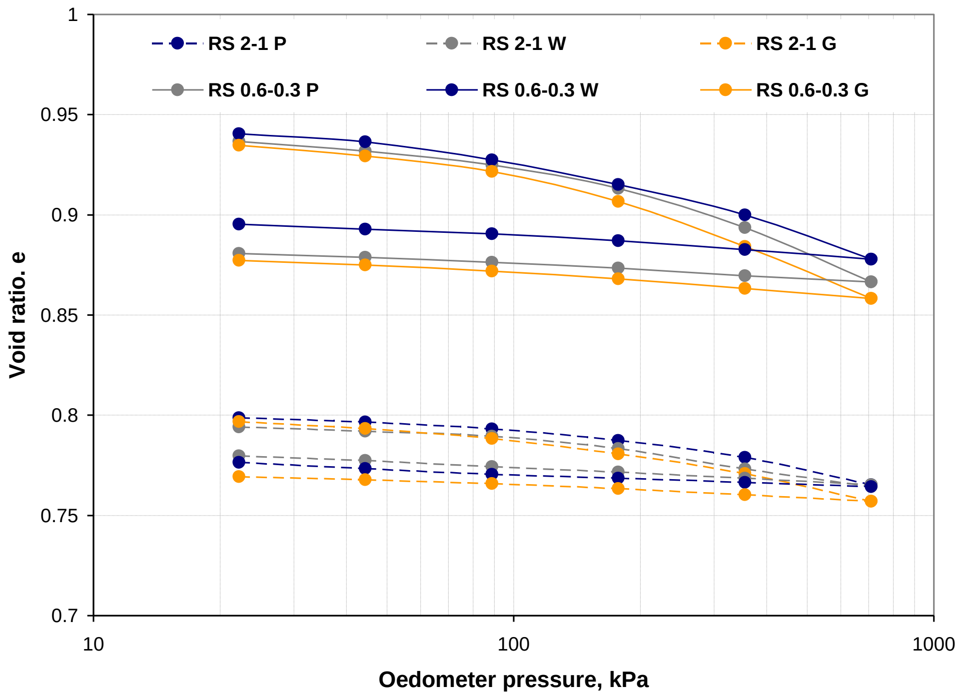


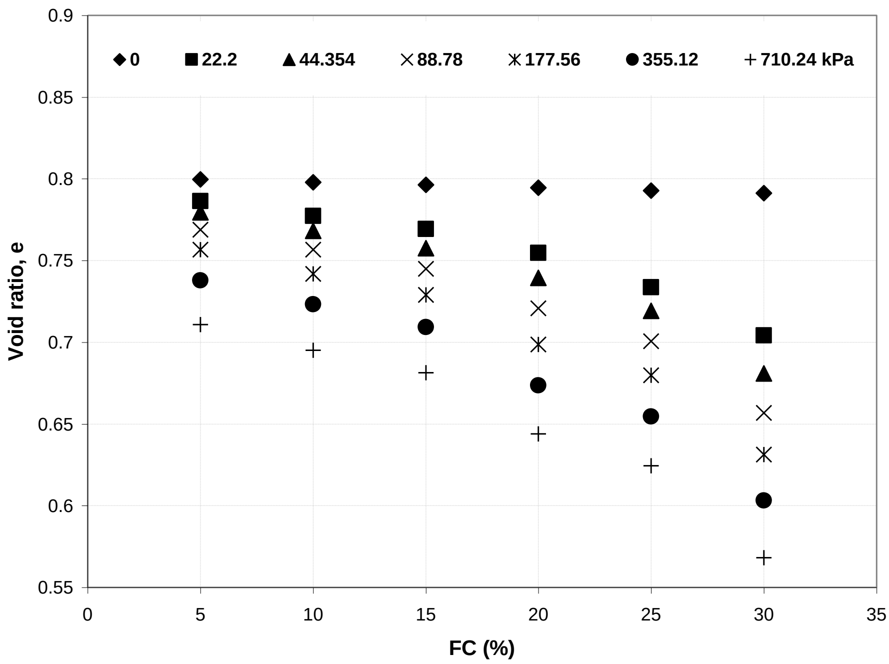
Figure 5.10 Variation of global void ratio for different particle size of rounded sand specimens saturated with different viscosity pore fluids

Figure 5.9 shows one of the consolidation testing results conducted on different specimens of crushed stone sands (AS) to investigate the change in void ratios of different specimens with different viscosity pore fluids. The tests were carried out under the vertical stress increments of 22.195, 44.354, 88.78, 177.56, 355.12, and 710.24 kPa. As can be seen that there is a significant effects of pore fluid viscosity on compressibility of specimens. It is also observed that the initial void ratio (e_o) of different particle sizes (0.3- 0.6 mm, 1.0- 2.0 mm) of specimens saturated with gasoline is the highest, as those saturated with petrol is the lowest one, and the particles having a larger size have less void ratio values than those having smaller size. In Figure 5.9 it is noted that the (e) values in specimens are high because of the open arrangement taken place in matrix of the angular particles.

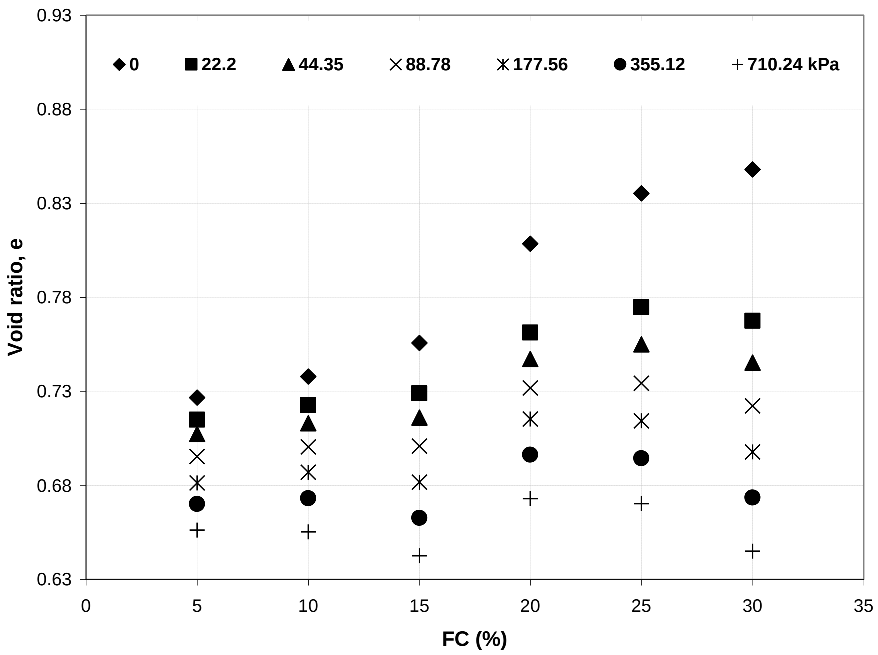
Figure 5.10 represent the change in e values for the different particle sizes (0.3- 0.6 mm, 1.0- 2.0 mm) of RS specimens saturated with gasoline, petrol and water. As can be observed that the higher e values were obtained in the RS specimens with water, and lower e values were obtained in gasoline used as pore fluid. It is interpreted that it could be because of the presence of some carbonate grains within the RS specimens which cause chemical reactions with the surfaces of those grains and changing the dimensions of particles. Actually, it points out the importance of the mineralogy of geomaterials tested in various pore fluids. In general, comparing the results obtained from Figures 2.9 and 2.10 , it is observed that e values in AS specimens are more than the e values determined in RS specimens. It is interpreted that the matrix of angular sand particles denotes to an open arrangement because of having sharp edges and corners, while lesser open arrangements occur in matrix of rounded particles. The change in void ratio also gives an indication for differences in compressibility for different shapes and sizes of angular and rounded sands. It is noticed that the compressibility of angular sand specimens is more than the compressibility of rounded sand specimens, because of the more interlocking between angular particles comparing to the rounded particles during one dimensional consolidation.

5.2.2 Variation of global void ratio with fines content for different particle sizes of sand.

Variation of global void ratio with fines content are presented in Figure 5.11 for sands (1.0- 2.0 mm) particle size mixed with different percentages of clay under different stresses.



(a)

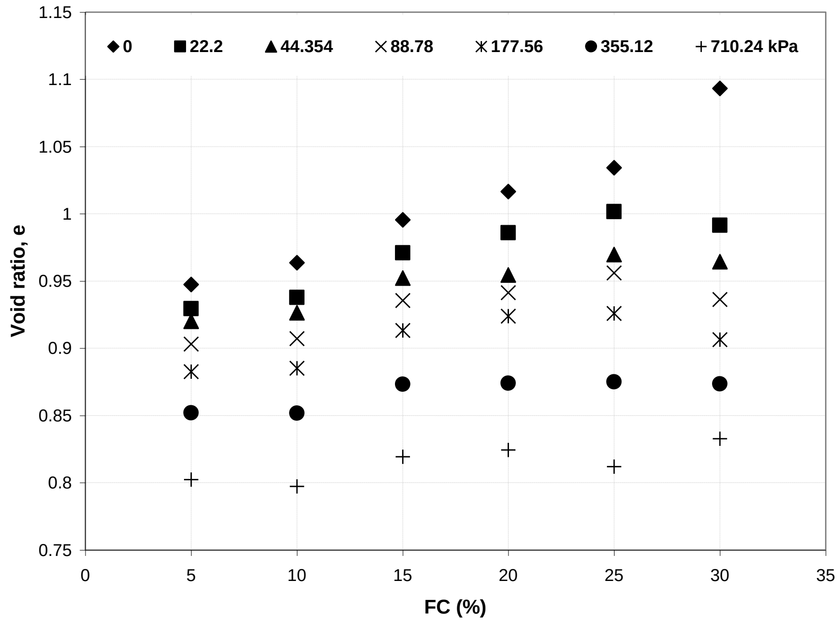


(b)

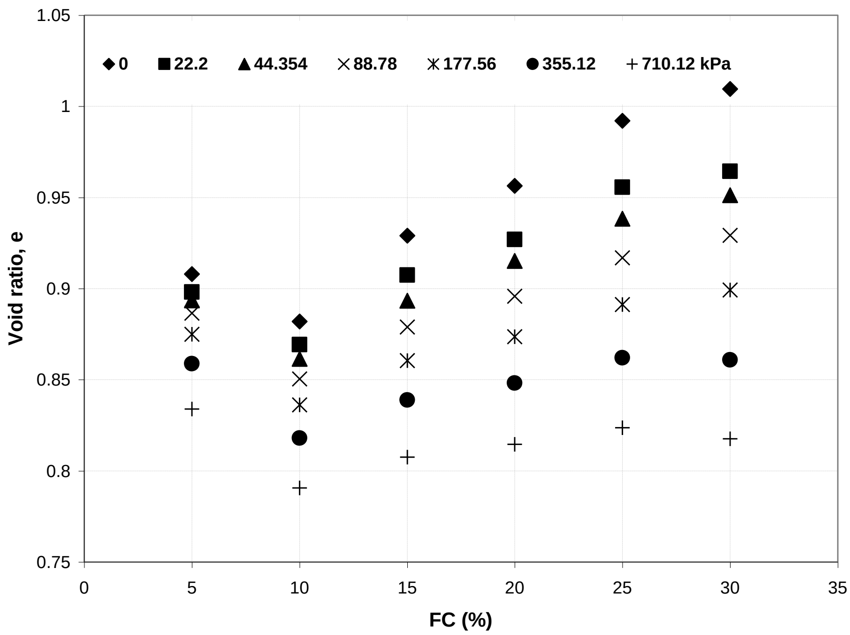
Figure 5.11 Void ratio- fine content under different stresses a) Crushed stone sand (1.0- 2.0 mm), b) Trakya sand (1.0- 2.0 mm)

As can be seen in Figure 5.11 (a) that the initial void ratio for Crushed stone sand specimens are scattered between 0.71 and 0.79 because of initial conditions. However, the initial void ratios for Trakya sand specimens are scattered between 0.65 and 0.84. These differences in initial void ratio may be due to the manner of arrangement of sand particles and the ratio of fine contents. Also Figure 5.11 (a) shows that the amount fines content is not sufficient for making separation between sand particles due to more angularity of particles, which makes an open fabric. However, the presence of small intergranular voids in specimens of Trakya sands which make the particles far away from each others by the addition of clay. Therefore, it is observed that there is a continuous decrease in void ratio for all percentages of fines content in Figure 5.11(a), while a continuous increase noticed in void ratio for all percentages of fines content, except for 5%, in Figure 5.11(b).

Figure 5.12 shows the variation of global void ratio for sands between 0.3 and 0.6 mm particle sizes mixed with various percentages of clay under different stresses.



(a)



(b)

Figure 5.12 Void ratio- fine content under different stresses a) Crushed stone sand (0.3- 0.6 mm), b) Trakya sand (0.3- 0.6 mm)

As can be observed in Figure 5.12 that the initial void ratios for Crushed stone sands and Trakya sands mixed with clays are scattered between 0.80 - 1.09 and 0.84 - 1.01 respectively. Clearly, the initial void ratios are very close to each others because of particle size, and much closest of particles in shape for both types of sand. As a result, there are steep decreases in initial void ratio of Trakya sand with fines content until it reaches 10%. But after that, the initial void ratio increases noticeably in both types of sandy soil.

5.2.3 Transition fines content

The transition fines content "FC_t" would occur when a direct contact of the coarse grains occur, at that time it is presumed that the intergranular void ratio of the mixture becomes equal to the maximum void ratio of the coarse granular material alone (i.e., $e_s = e_{max-c}$) if there is no any fines are located among the coarse grains contact. When the equality between intergranular void ratio and maximum void ratio of the coarse granular material occurs, the fines content is named as "transition fines content". The concept of intergranular void ratio assumed that the fines would be locate in a continuous matrix which formed by the coarse grains matrices.

Figure 5.13 and 5.14 depict the changes in intergranular void ratio (e_s) of the sand- clay mixtures under different oedometer pressures (Thevanayagam, 1998; Monkul and Ozden, 2007). In both Figures 5.13 and 5.14, It can be seen that the e_s increases as the clay content increases for both specimens. Amount of increase in e_s values for the AS specimens seems to be higher (i.e., open fabric) than those for the RS specimens over all the percentages of clay contents. It is interpreted that the differences in shapes of the sands lead to change the initial void spaces, and therefore affects the arrangement (e.g., open fabric, interlocking) of the sand- clay mixtures as well as the clean sands grains.

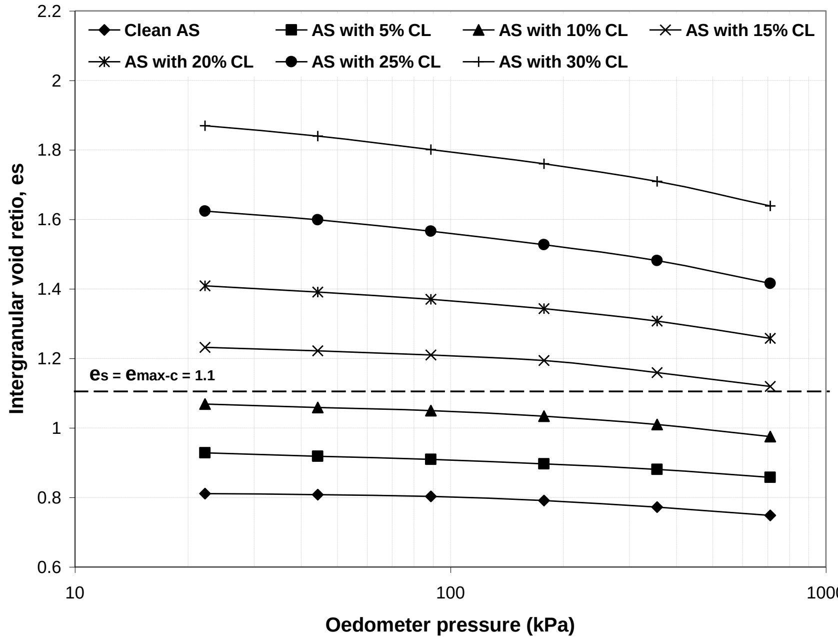


Figure 5.13 Variation of intergranular void ratio with different oedometer stresses for AS specimens (1.0- 2.0 mm)

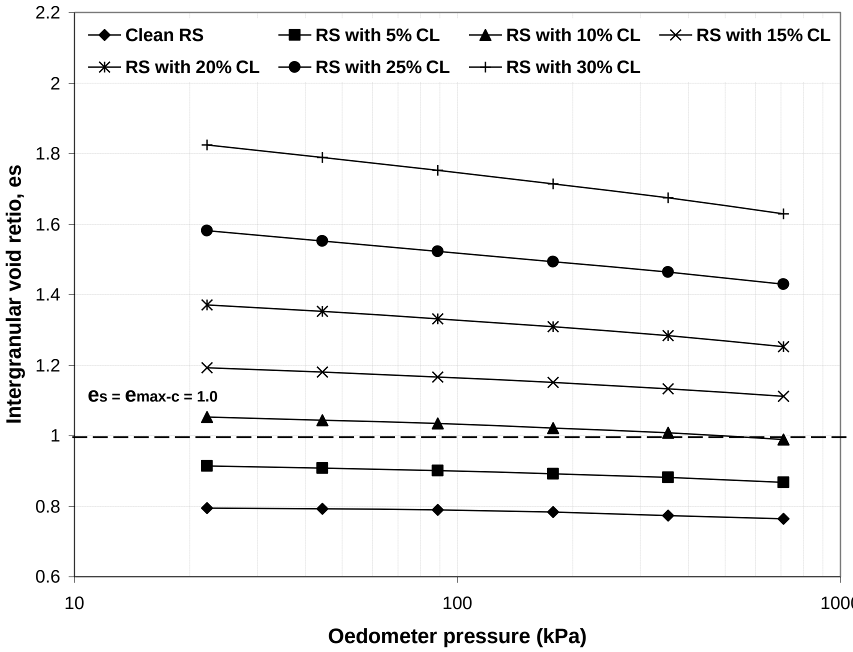


Figure 5.14 Variation of intergranular void ratio with different oedometer stresses for RS specimens (1.0- 2.0 mm)

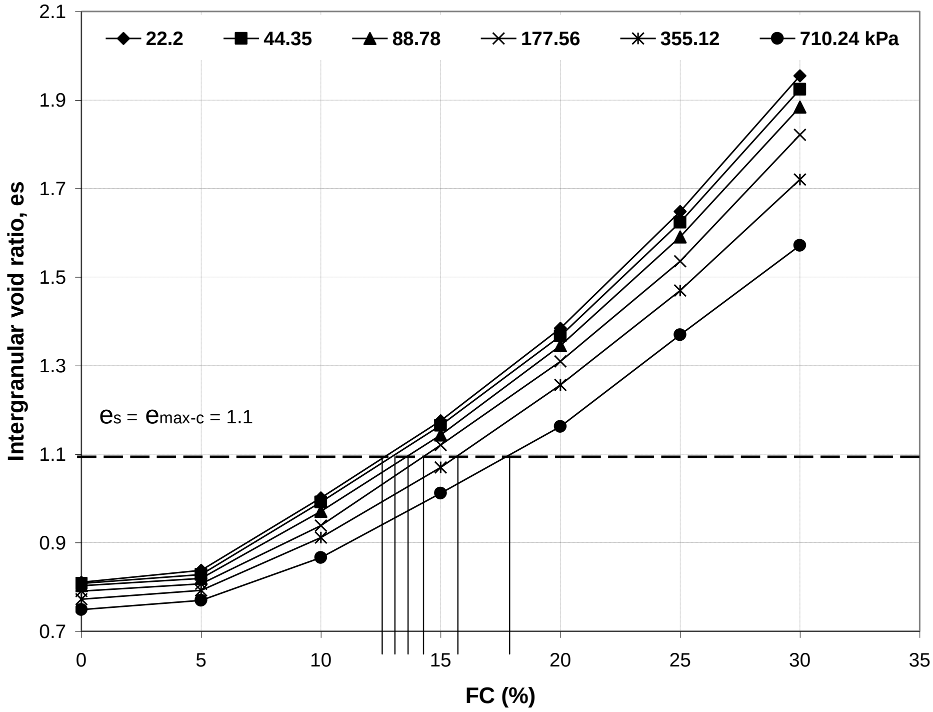


Figure 5.15 Intergranular void ratio versus fines content under different stresses (AS specimens) saturated with water

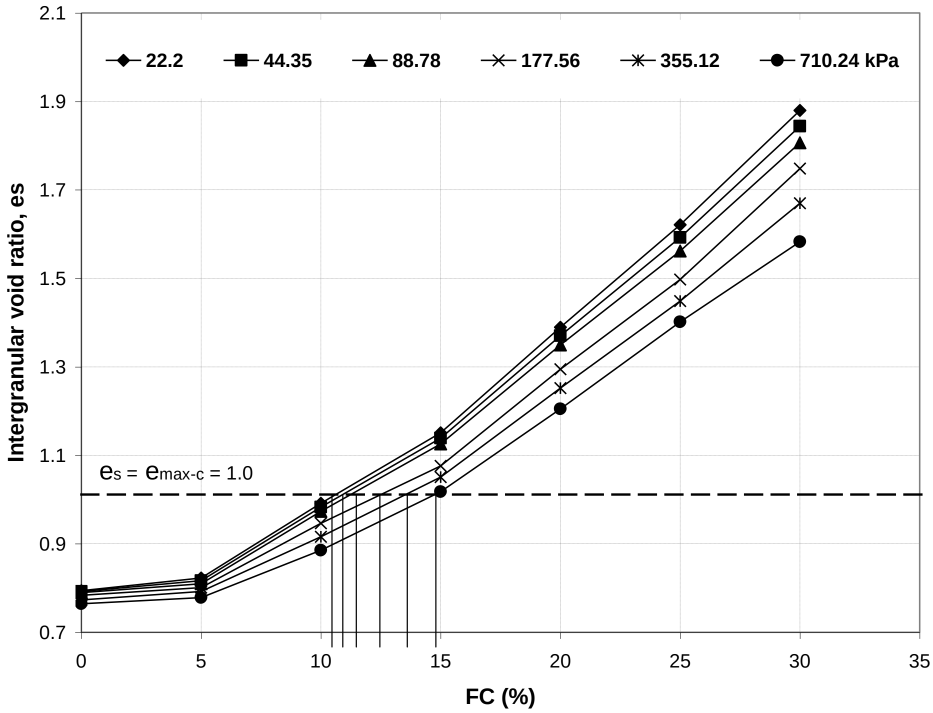


Figure 5.16 Intergranular void ratio versus fines content under different stresses (RS specimens) saturated with water

Figure 5.15 and 5.16 shows the variation of intergranular void ratio with fines content. As can be seen that the dashed lines intersect all of the curves. The corresponding fines content to the intersection points with the curves are the fine transition contents FC_t for different fines content at different oedometer stresses. Effects of water on FC_t values are tabulated in Table 5.5 for the different applied stresses.

Table 5.5 Transition fines content for different oedometer stresses

Oedometer pressure (kPa)	FC_t (%) for AS specimens	FC_t (%) for RS specimens
22.2	12.61	10.45
44.35	13.2	11.03
88.78	13.75	11.6
177.56	14.45	12.5
355.12	15.8	13.63
710.25	17.85	14.8

As can be seen in Table 5.5, transition fines content values in the range of 12%-18% for AS specimens and 10%- 15% for the RS specimens. For both specimens FC_t values increase with an increase in effective stress. In other word, the increase in transition arrangement of coarser grain matrix would occur for higher values of fines content and in incremental effective stresses. Therefore, when fines content increased, the coarser grain needs a higher stress to rearrange them and to form contacts between the particles. Because with an increase in fines content, the coarser grain matrix orients itself in a looser state due to decreasing of the amount of coarser grains. In Table 5.5 and Figures 5.15, 5.16. It is observed that the amount of increase of FC_t values for AS specimens are higher than those for RS specimens over all percentages of clay content. It is explained that the shape of sand particles make the void spaces between the particles to change, because it is observed that the e_s values for AS specimens are higher (i.e., open fabric), as a result the more interlocking of sand – clay mixtures occurs. Figure 5.17 shows variation of fines transition content with oedometer pressure for water saturated specimens.

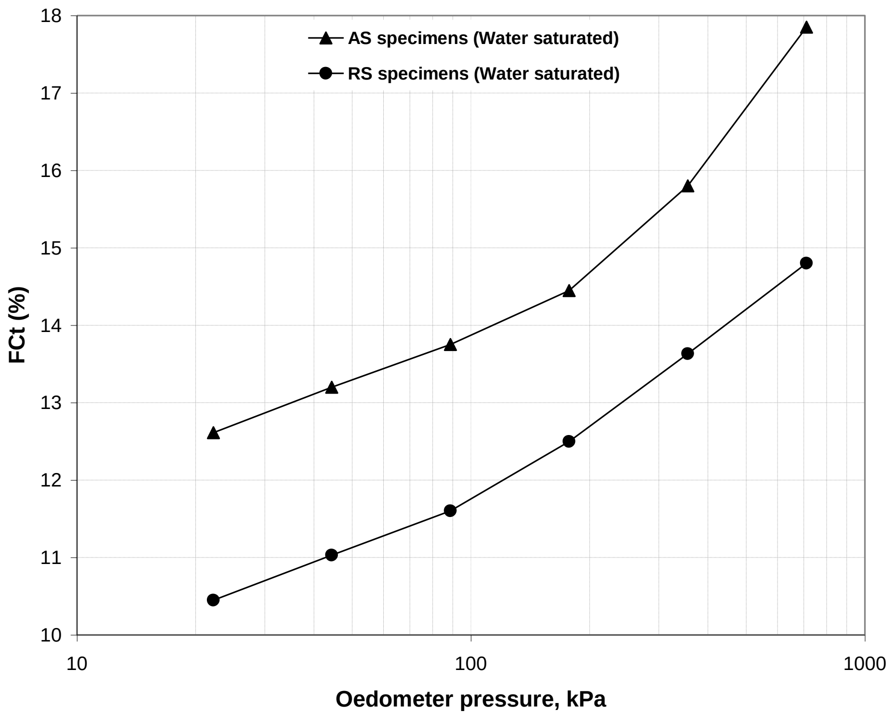


Figure 5.17 Variation of transition fines content with oedometer pressure

The transition fines content FC_t decreases when the pore fluid viscosities increase, because of the viscosity of the medium. Figure 5.20 shows specimens saturated with gasoline as pore fluid. It is observed that the FC_t valeus are decreased in comparison to the FC_t values that illustrated in Figure 5.17. Because the viscosity of the medium changed which leads to decrease the compressibility of the specimens as a result of the resistance of fluids to the relative motion of immersed objects (i.e., sand, clay). Similarly, for RS specimens the FC_t values also decreased, but the amount of decrease is not in an amount that noticed in AS specimens, it could be due to the shape of particles, as angular sand particles have higher compressibility (i.e., interlocking, open fabric) than the rounded sand particles. Figure 5.18 and 5.19 shows intergranular void ratio with fines content for both AS and RS specimens.

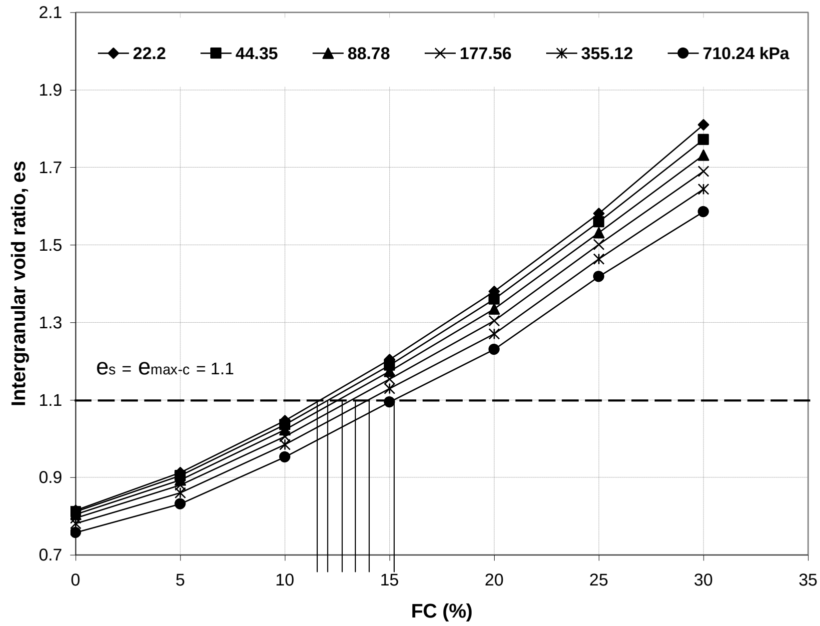


Figure 5.18 Intergranular void ratio versus fines content under different stresses (AS specimens) saturated with gasoline

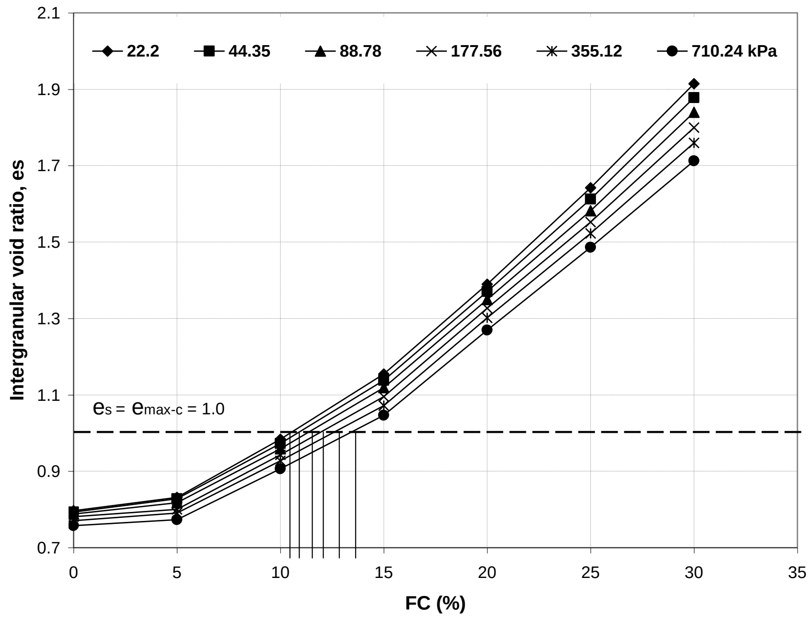


Figure 5.19 Intergranular void ratio versus fines content under different stresses (RS specimens) saturated with gasoline

Table 5.6 and Figure 5.20 show alteration of transition fines content for both AS and RS specimens saturated with gasoline.

Table 5.6 Transition fines content for different oedometer stresses

Oedometer pressure (kPa)	FC_t (%) for AS specimens	FC_t (%) for RS specimens
22.2	11.6	10.5
44.35	12.15	10.9
88.78	12.7	11.6
177.56	13.4	12.05
355.12	14.1	12.95
710.25	15.23	13.6

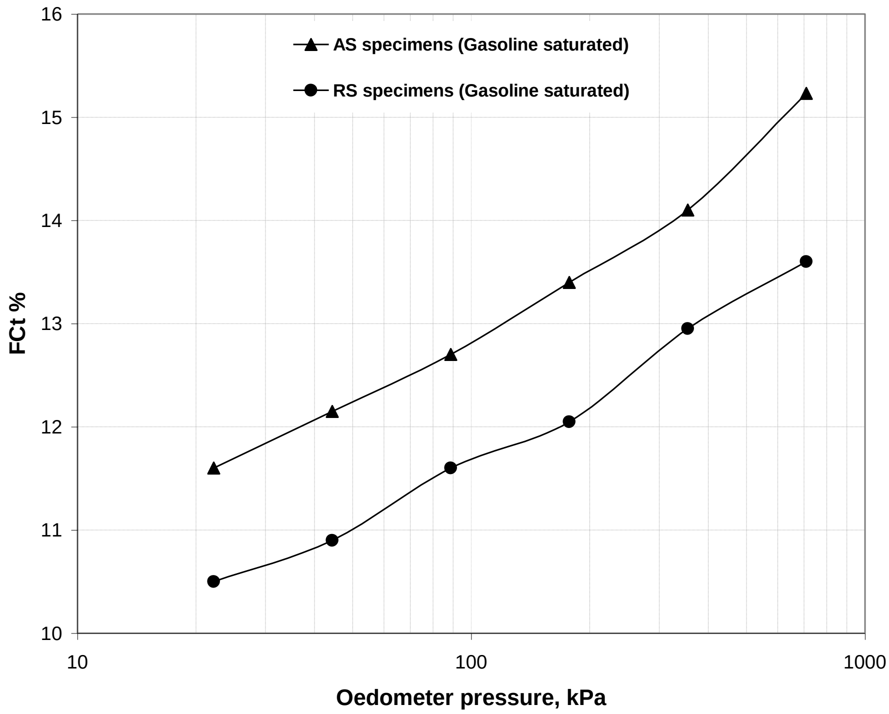


Figure 5.20 Variation of transition fines content with oedometer pressure

For calculating the FC_t values for a mixture of sands sized (0.3- 0.6 mm) with different fines content, both AS and RS specimens were tested to investigate the effects of size of coarser grain matrices on the alteration of 'FC_t' values in case of water and gasoline saturated pore fluids. Figure 5.21 and 5.22 show the change in intergranular void ratio with fines content for both specimens. It can be seen in both Figures that the e_s values increase as the clay content increases. Amount of increase AS specimens again gives higher values as it is noticed in Figure 5.13 and similarly for RS specimens the value of e_s again smaller in comparison with Figure 5.14. The difference in es vales between AS and RS specimens is due to the differences in shapes of sands which affects the initial void spaces, and the arrangement of the sand- clay mixtures as well as the clean sand particles.

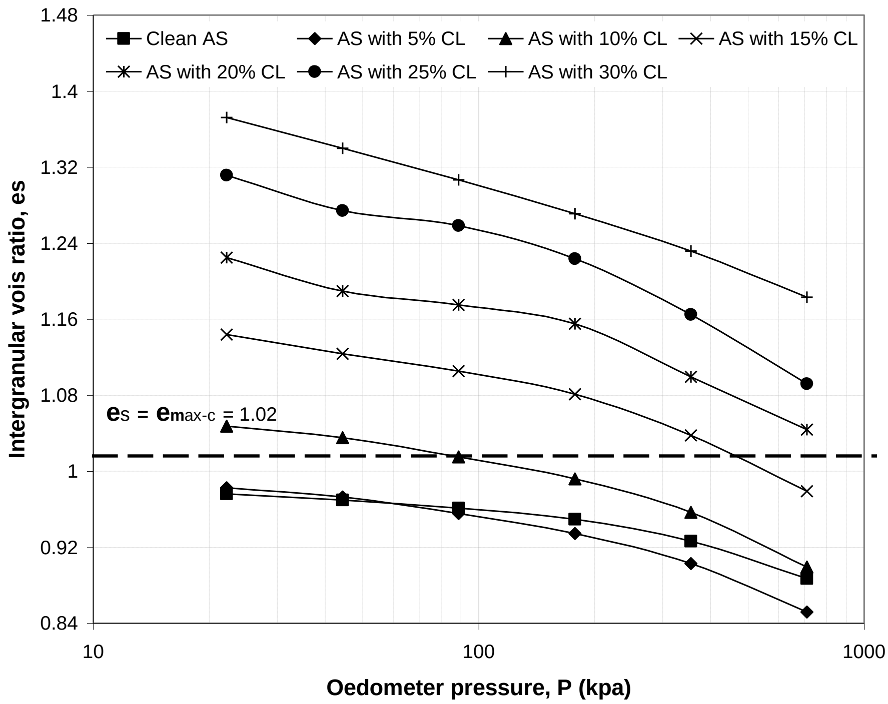


Figure 5.21 Variation of intergranular void ratio with different oedometer stresses for AS specimens

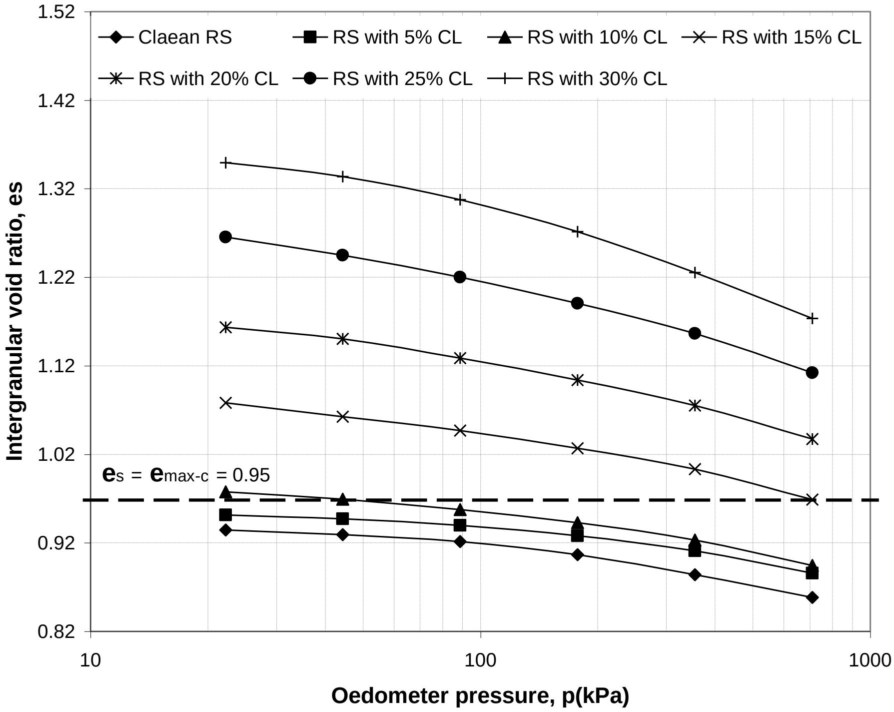


Figure 5.22 Variation of intergranular void ratio with different oedometer stresses for RS specimens

Alteration of transition fines content, FC_t were calculated for sand particle sized (0.3- 0.6 mm) mixed with different percentages of clay in water saturates state. It is observed that FC_t values increases as the clay content increases under different stresses. Amount of increase in FC_t is more than the amount of increase that determined for (1.0- 2.0 mm) sands mixed with clay. It is interpreted that the smaller particle size undergone to pressure more than the greater particle sizes, this may be due to higher stiffness of greater particle sizes of sand within the mixture. Figure 5.23 presents the change in FC_t values with fines content for AS specimens under different stresses. Amount of increase is more than the amount of increase for RS specimens which presented in Figure 5.24. This could be due to the difference in shapes of sand grains that affects the manner of arrangement (i.e., fabric) of sand- clay mixtures.

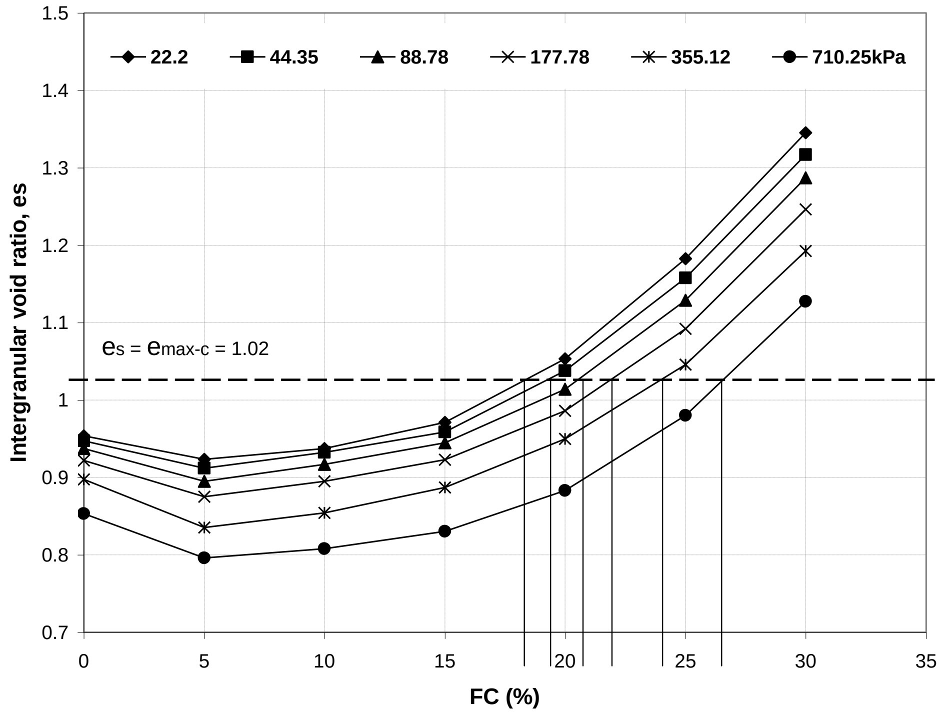


Figure 5.23 Intergranular void ratio versus fines content under different stresses (AS specimens) saturated with water

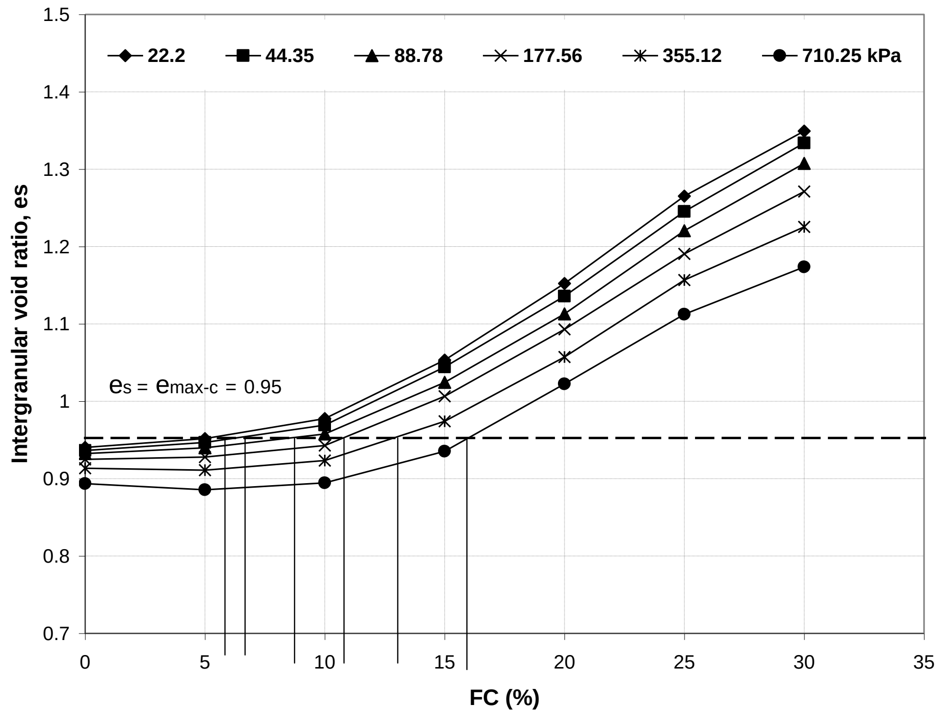


Figure 5.24 Intergranular void ratio versus fines content under different stresses (RS specimens) saturated with water

FC_t values and the corresponding stress for both AS and RS specimens are tabulated in Table 5.7 and illustrated in Figure 5.25.

Table 5.7 Transition fines content for different oedometer stresses

Oedometer pressure (kPa)	FC_t (%) for AS specimens	FC_t (%) for RS specimens
22.2	11.6	10.5
44.35	12.15	10.9
88.78	12.7	11.6
177.56	13.4	12.05
355.12	14.1	12.95
710.25	15.23	13.6

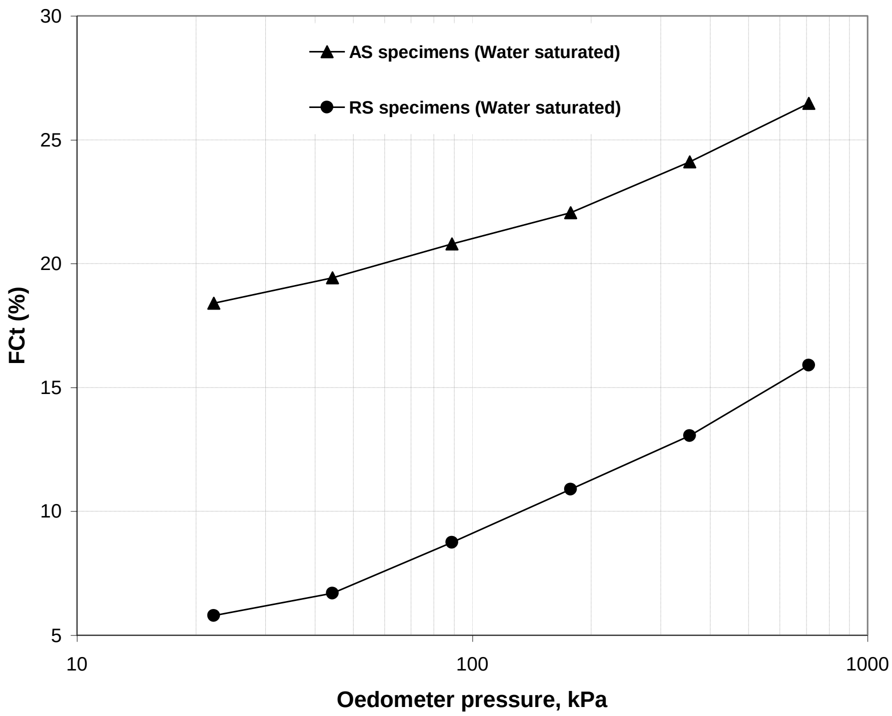


Figure 5.25 Variation of transition fines content with oedometer pressure

Figure 5.26 and 5.27 represents intergranular void ratio with fines content for AS and RS specimens (0.3- 0.6 mm) saturated with gasoline. It can be seen that FC_t values decreased noticeably. From a comparison between specimens of both mixed soils of (1.0- 2.0 mm) particle sizes and specimens of (0.3- 0.6 mm) particle sizes. It is observed that the viscosity of the medium (gasoline) have affected FC_t values in specimens of (0.3- 0.6mm) sands mixed with clay more than the FC_t vales for specimens of (1.0- 2.0 mm) sands with clay. It is interpreted that in smaller particle size specimens, the gasoline clogs the relative motion in the void spaces due to the high viscosity.

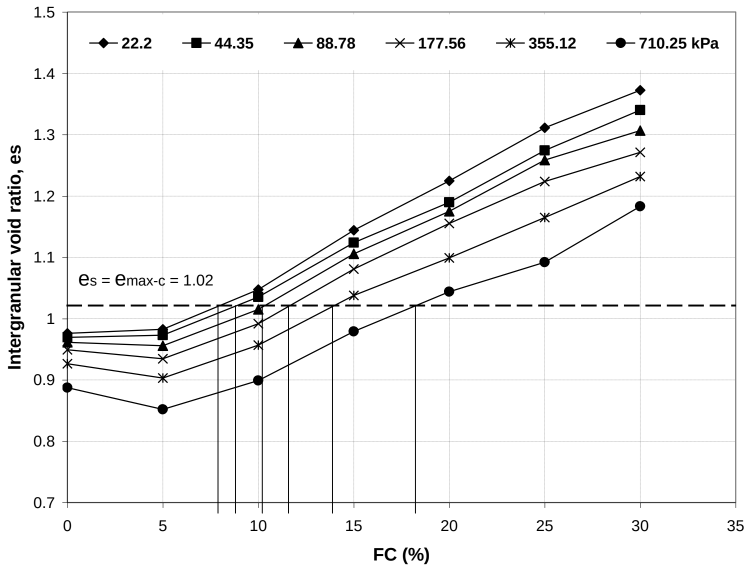


Figure 5.26 Intergranular void ratio versus fines content under different stresses (AS specimens) saturated with gasoline

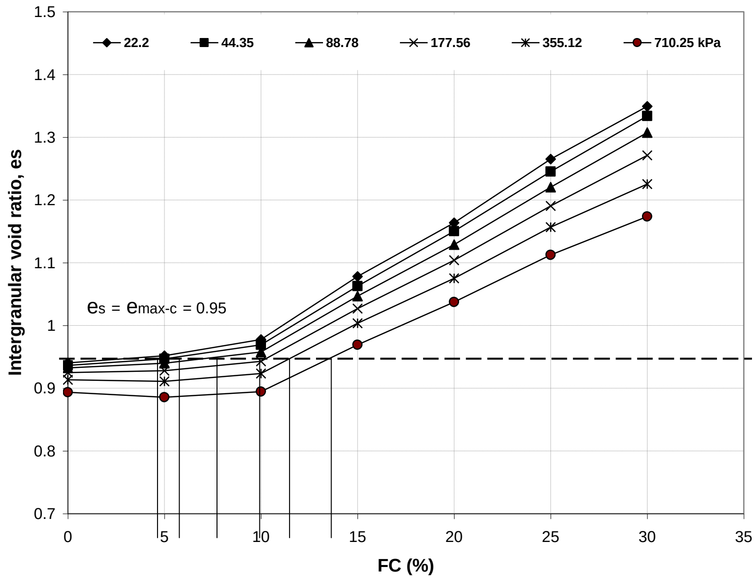


Figure 5.27 Intergranular void ratio versus fines content under different stresses (RS specimens) saturated with gasoline

Figure 5.28 shows the effects of gasoline on alteration of FC_t values for both AS and RS specimens. The FC_t values and the corresponding stresses are tabulated in Table 5.8.

Table 5.8 Transition fines content for different oedometer stresses

Oedometer pressure (kPa)	FC_t (%) for AS specimens	FC_t (%) for RS specimens
22.2	7.95	4.65
44.35	8.86	5.8
88.78	10.23	7.72
177.56	11.6	10.0
355.12	14.0	11.6
710.25	18.3	13.65

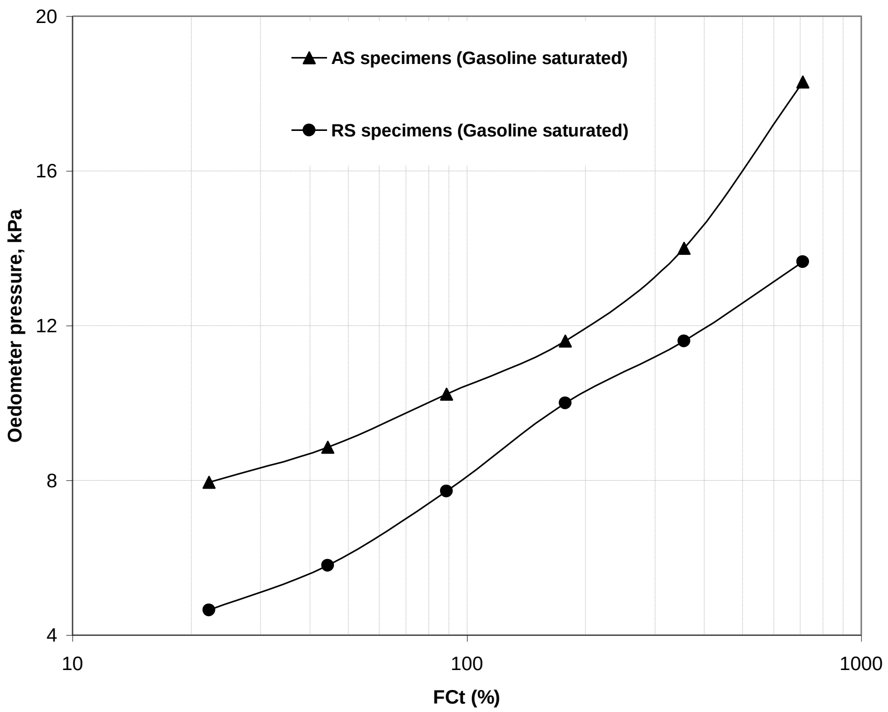


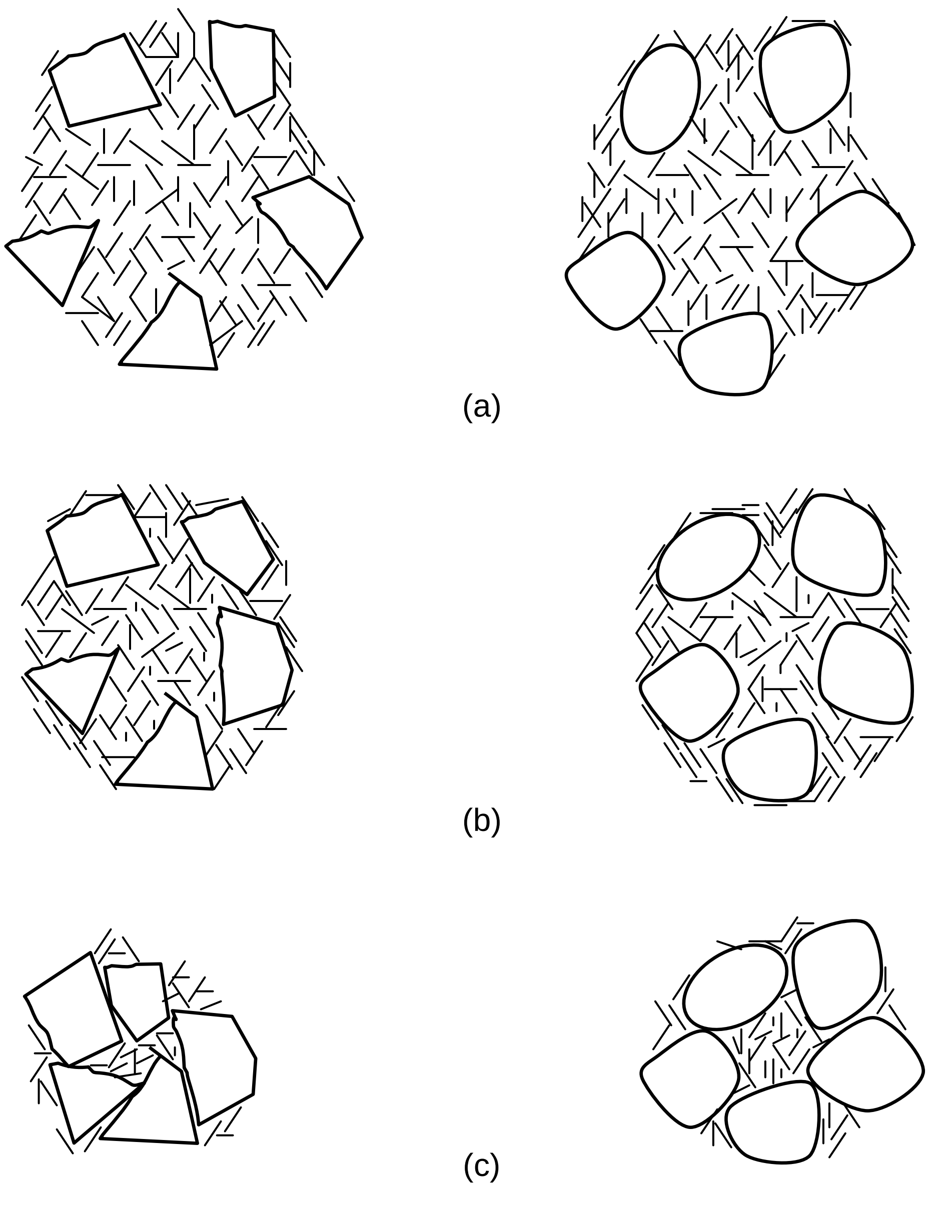
Figure 5.28 Variation of transition fines content with oedometer pressure

5.2.4 Variation of compression indices with fines content for different viscous pore fluids

In this study both granular compression and global compression indices were studied for (angular and rounded) shaped particles of sands mixed with various amounts of clay in order to investigate the role of shape on compression. Also the effects of different particle size of sands were used in this investigation to inspect the effects of size of coarse particles on compression behavior of mixed soils. Different viscous fluids were used for this study, water and gasoline saturated pore fluids in order to show the effects of the viscosity of the medium on compression indices of mixed soils.

During compression, coarser and finer particles rearrange themselves into various modes depending on the initial condition and applied stress. The mode of arrangement of coarser particles within the mixture could be varied for different particle shapes, which leads to change the initial condition of the sample. Figure 5.29 from a) to c) show different arrangements for different particle shape of sands within the mixture. The establishment of direct grain contact of the coarser grain matrix can be assumed to initiat when intergranular void ratio of the mixture becomes equal to the maximum void ratio of the host granular material (Monkul and Onal, 2006; Cabalar, 2011).

Figure 5.30 presents the changes in compression index ($C_{c\text{-}s}$) estimated for only sand grains (angular or rounded), and compression index (C_c) for the mixture of sand and clay under 88.78 kPa oedometer pressure. The results were give for both AS and RS specimens tested with water. It can be seen that the compression index values ($C_{c\text{-}s}$, C_c) of both mixed soils increases sharply in the specimens of clay content more than 15%. The reasons can be explained by exchanging of the role that governs the behavior of the mixture, from sand to clay at this percentage. The transition fines content (FC_t) value in this study are in a good aggrement with the previously published findings by (Monkul and Ozden, 2007; Cabalar, 2010), they interpreted that transition fines content (FC_t) value should be around 15% clay content.



(a)

(b)

(c)

Figure 5.29 Different arrangements of matrices for different particle shape of sands under 1 D compression

As can be seen in Figure 5.30 that C_{c-s} and C_c values for AS specimens are higher than the values of RS specimens. This is because of the initial void ratio values (e_o) of AS specimes are higher than the e_o of RS specimens (i.e., open fabric). Figure 5.31 shows the variations of compression indices with clay content under 710.24 kPa oedometer pressures.

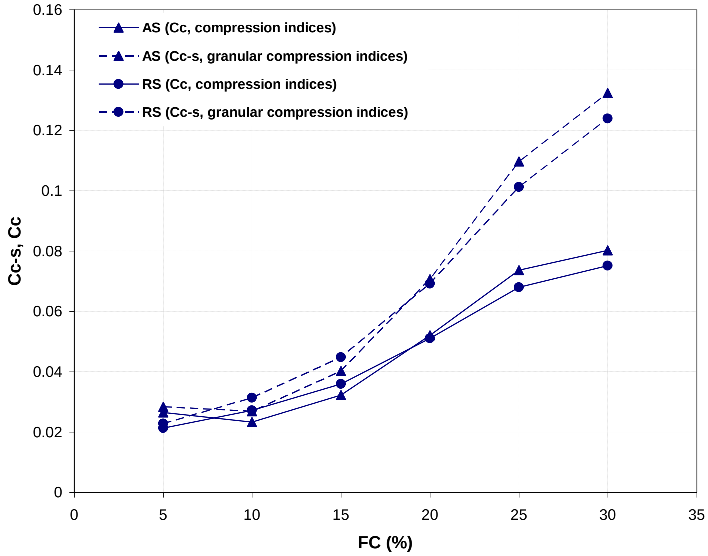


Figure 5.30 Variation of compression indices ($C_{c\text{-}s}$, C_c) for specimens saturated with water, under 88.78 kPa oedometer pressure

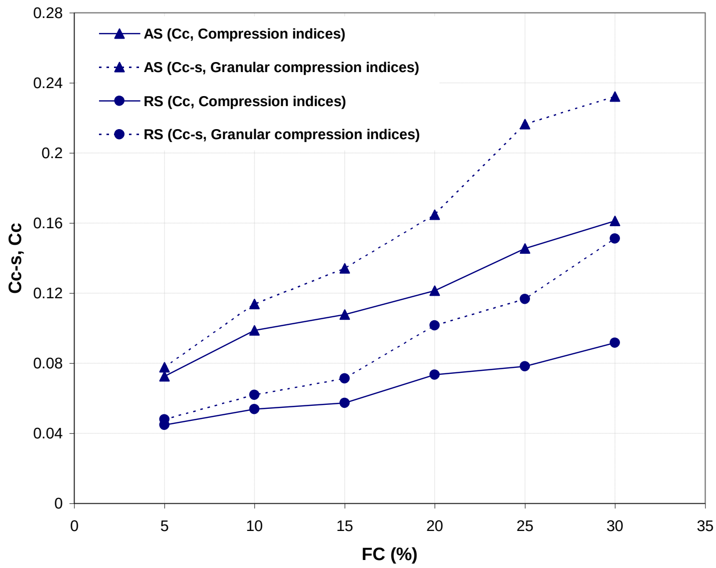


Figure 5.31 Variation of compression indices ($C_{c\text{-}s}$, C_c) for specimens saturated with water, under 710.24 kPa oedometer pressure

As can be observed from the Figure 5.3 there is no clearly seen FC_t value within the measured strain levels. However, from the previous experiences, it is estimated that an FC_t value could take a place over larger strain levels. It is also observed that in high oedometer pressure, the variation in C_{c-s} and C_c values for AS specimens are higher than the values for RS specimens due to the more interlocking of AS specimens during compression.

Variations in C_{c-s} and Cc values for the mixtures under 88.78 kPa oedometer pressure and gasoline saturated specimens are illustrated in Figure 5.32. It is observed that the compression index values (C_{c-s}, C_c) for both specimens increased sharply with clay content of 15% as it noticed in water saturated specimens. The Figure 5.32 also show that the compression index values (C_{c-s}, C_c) between the AS specimens and RS specimens get closer by increasing clay content in gasoline, and the gasoline decreases the C_{c-s} and C_c values for both mixtures. It could be because of the high viscosity of the pore fluid, which hinder the particles to subject the compressions.

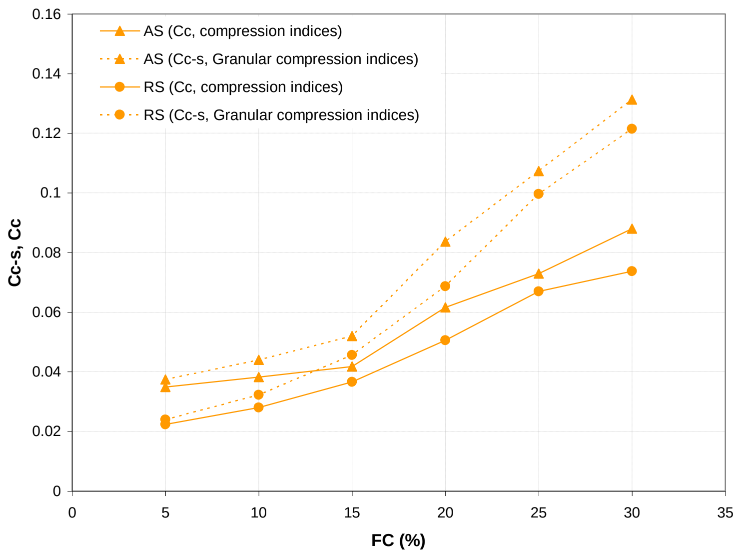


Figure 5.32 Variation of compression indices (C_{c-s}, C_c) for specimens saturated with gasoline, under 88.78 kPa oedometer pressure

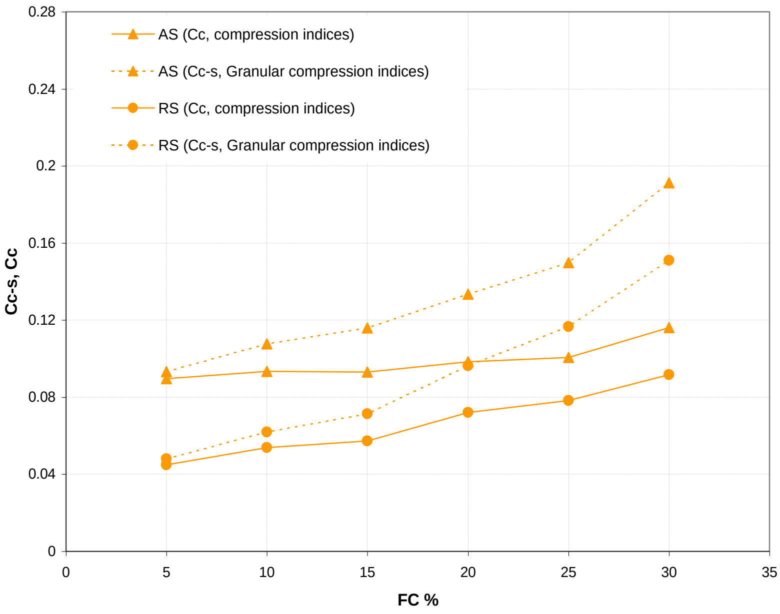


Figure 5.33 Variation of compression indices ($C_{c\text{-}s}$, C_c) for specimens saturated with gasoline, under 710.24 kPa oedometer pressure

Alteration of granular compression index ($C_{c\text{-}s}$) and global compression index (C_c) parameters for sands sized (0.3- 0.6 mm) mixed with different percentages of clay were tested, in order to investigate the effects of smaller particle sizes of coarser granular materials on compression behavior of mixed soils. Variation of $C_{c\text{-}s}$ and C_c values for both AS and RS specimens under 710.24 kPa oedometer pressure are illustrated in Figure 5.34. It can be seen that the compression index values ($C_{c\text{-}s}$, C_c) of both mixed soils increases sharply in the specimens with clay content more than 10%. It can be explained by exchanging of the role that governs the behavior of the mixture, from sand to clay at this percentage. From the study by Cabalar, (2010) it is interpreted that the transition fines content (FC_t) value could be around 15% clay content. From a comparison between the Figures 5.31 and 5.34, again there is no clearly seen FC_t values taken place in the measured strain levels. However, there are some differences observed in compression index values ($C_{c\text{-}s}$, C_c) between both of the two figures. It can be seen that the compression index parameters in Figure 5.30 gives higher values than those of Figure 5.32. It may be due to the initial condition of specimens which controlled by the difference in particle sizes of coarser grain matrix. Figure 5.35 shows the variation of compression index ($C_{c\text{-}s}$, C_c) for mixed soils saturated with gasoline. It can be noticed that the gasoline decreases the $C_{c\text{-}s}$ and C_c values for both AS and RS specimens, it may be due to the high viscosity of the medium. As an overall view, it is also seen that $C_{c\text{-}s}$ and C_c values of AS specimens are higher than the values of RS specimens. The main reason of this might lie in the fact that initial void ratio values (e_o) of the AS specimens are higher than the e_o of RS specimens (i.e., open fabric).

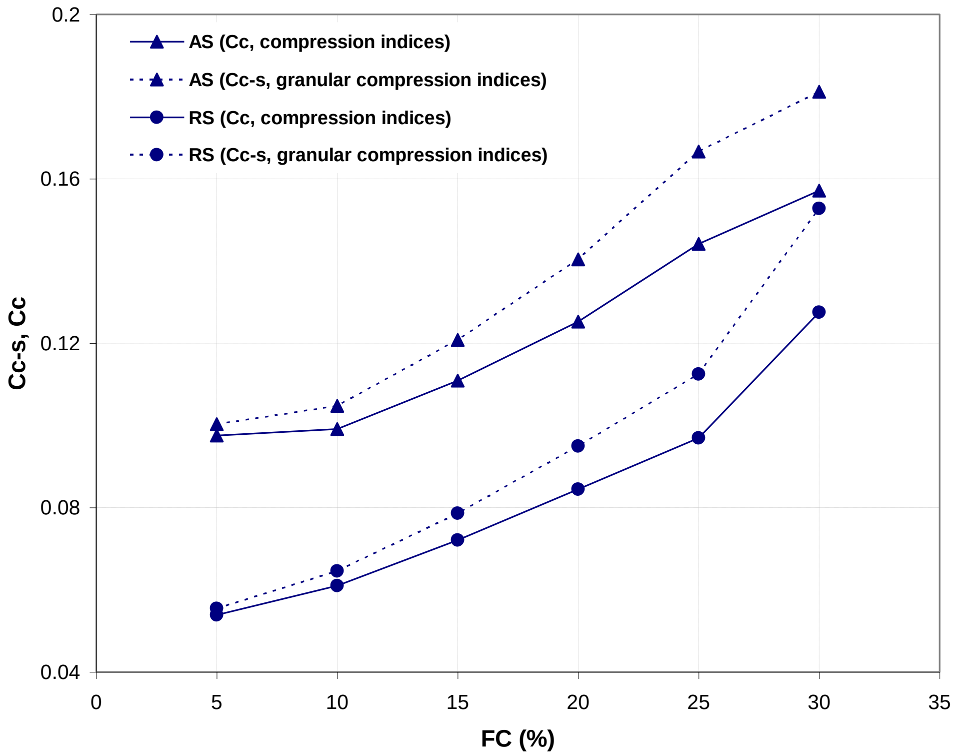


Figure 5.34 Variation of compression indices ($C_{c\text{-}s}$, C_c) for specimens saturated with wate, under 710.24 kPa oedometer pressure

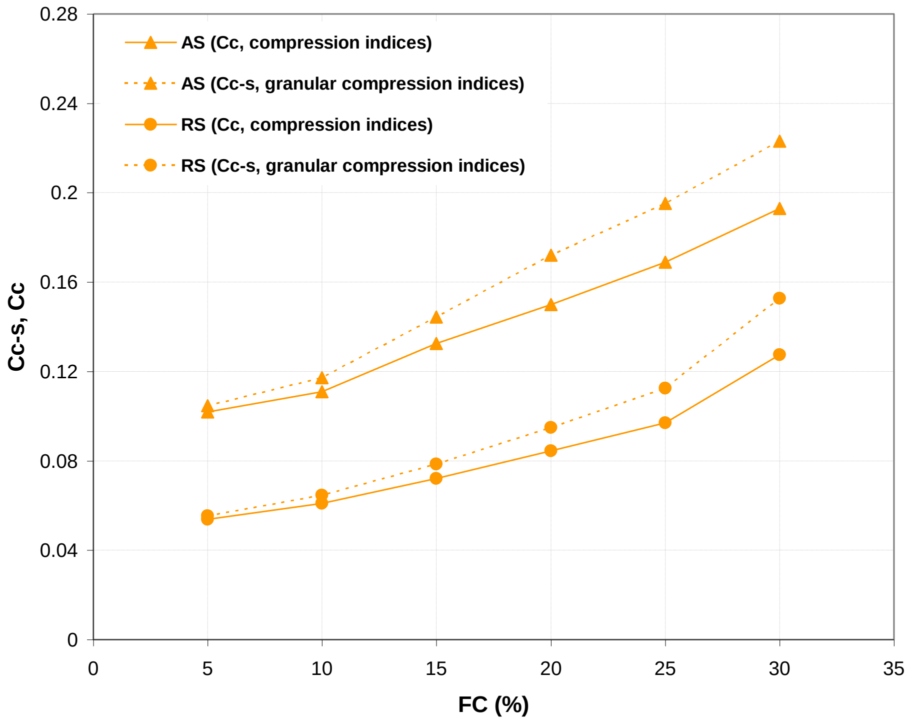


Figure 5.35 Variation of compression indices ($C_{c\text{-}s}$, C_c) for specimens saturated with gasoline, under 710.24 kPa oedometer pressure

CHAPTER 6

SUMMARY AND CONCLUSIONS

Depending on the findings of deep experimental program presented in the previous chapters, the study leads to the following conclusions:

- The influence of particle size on shear strength shows a reduction, as particle size of sands decreased for both Crushed stone sand and Trakya sands. Crushed stone sand gives higher internal friction angle than Trakya sand, because of angularity of particles of Crushed stone sand. Also more decrease is observed in angular sand as particle size decreases, and a significant difference was noticed in shear strength for different particle sizes of both sands.

- As angularity of particles increase, the arrangement of grains could be edge to edge or corners contact which leads to produce more void spaces. Conversely for rounded shape particles, because of having no sharp edges and corners. Therefore, more interlocking is observed in angular particles during shearing and applied normal force which leads to observe a higher shear strengh.

- In general, pore fluid viscosity affects the shear strength of sands. The shear strength parameters (c, ϕ) of the sands was decreased as the pore fluid viscosity increases because of a possible slippage of particles along the contact surfaces and simplifying the rolling of particles over each others.

- A more decrease in angle of internal friction was observed in sands having rounded shaped particles due to the presence of closed pores in rounded sand samples, which leads to sustain fluids within the void spaces.

- Compression characteristics of sands saturated with different viscosity pore fluids (water, petrol, gasoline), generally decrease with increasing the viscosity, however in some cases, particularly in more viscous environment, the compressibility of sand could decrease. This may be due to the mineralogy of sand grains, or due to cloging effects of fluid in the pores on the compression characteristics of sand.

- The initial void ratios of sand specimens changed due to the difference of particle shapes. Therefore, the shape affects the arrangement of particles (e.g., interlocking open fabric) of the sand- clay mixtures as well as the clean sands grains.

- Shear stress values in the specimens with less clay content increases in a much stepper trend than those with more clay content within the measured strain level. Therefore, the mixtures with high amount of clay have a lower shear strength

- In all types of fluids used, the compressibility of angular sand – clay mixtures are higher than rounded sand- clay mixtures, because of more interlocking of angular sand particles within the mixture.

- Granular compression index (C_{c-s}), and global compression inex (C_c) values of both mixed soils tested at a lower vertical pressure increase sharply in the specimens with clay content more than 15%. Therefore, it is thought that FC_t value of the specimen at the specified stress value is to be around 15% clay content.
- Gasoline decreases the C_{c-s} and C_c values of both angular and rounded sand- clay mixtures, because of high viscosity of the medium which the particles are immersed in it.

CHAPTER 7

FUTURE WORKS

The study recommended the following points:

- A detailed investigation on the shape of granular materials is required, For example; using a scanning electron microscope (SEM) would be very useful for a better understanding of form and dimensions of granular materials.
- A detailed study on the mineralogical compositions for different granular materials needs to be conducted by using X-ray diffraction method to investigate the effects of mineralogical composition on engineering behavior of granular materials.
- Observations on dynamic properties of granular materials with different viscosity fluids needs to be made in order to inspect the effects of viscous fluids on dynamic behaviors of sandy soils.
- An investigate on chemical composition of contaminant fluids are necessary for a better understanding of physicochemical interactions on the mineralogical composition of coarse and fine grained soils.

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