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Research Review Report on Effect of Specimen Size on Shear Strength of Soil in Laboratory Test

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January 20, 2023

केन्द्रीय मृदा एवं सामग्री अनुसंधानशाला
Central Soil and Materials Research Station

रिपोर्ट सं. 01/RRR/SM/CSMRS/03/2021-22

प्रयोगशाला परीक्षण में मृदा की अपरूपण बल पर नमूने के आकार के प्रभाव पर अनुसंधान समीक्षा रिपोर्ट

RESEARCH REVIEW REPORT ON EFFECT OF SPECIMEN SIZE ON SHEAR STRENGTH OF SOIL IN LABORATORY TEST

द्वारा
by

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मार्च 2022
March 2022

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ABSTRACT

The behavior of particular soil from different studies are often compared without due attention to the differences in specimen size and its effects on soil shear behavior. This Research Review Report highlights the various specimen size in triaxial test from smaller to bigger diameter (i.e., 30 to 110 mm dia.) adopted by several international codes and standards to determine shear strength parameters under different drainage condition. The influence of specimen size and its scale effect studied by various researchers are presented and discussed in literature review. It cannot be disregarded that smaller specimen can achieve good amount of consolidation and higher compressibility as compared to bigger specimen. Consequently shear strength may increase with decrease in specimen size. The choice of bigger sample size is thus a more accurate representation of soil strength conditions with respect to field deformation. BIS codes has also provided flexibility to use four different specimen size from smaller to bigger diameter (D) i.e., 38, 50, 70, 100 mm to determine shear strength parameters in triaxial apparatus.

The lower deviator stresses in the bigger specimens is associated with the more intense strain softening resulting from shearing which may be due to smaller compressibility behavior as compared to smaller specimen.

Study on effect of specimen size on pre-peak, peak and post peak stress behavior with respect to strain can uncover major discrepancies when tested in smaller specimen. Smaller specimen may develop migration of pore water pressure quickly as compared to bigger specimen. Pore water pressure under shearing may differs by specimen size due to which large difference in the effective stress paths can be observed. To reduce specimen size effect on the smaller specimen with respect to shear strength parameters, a correction factor related to size effect can be proposed with extensive testing program. This will help to assign appropriate shear strength parameters for better representation of field soil behavior.

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RESEARCH REVIEW REPORT ON EFFECT OF SPECIMEN SIZE ON SHEAR STRENGTH OF SOIL IN LABORATORY TEST

1.0 INTRODUCTION

Shear strength is the principal engineering property which controls the stability of a soil mass under loads. Most of the problems in soil engineering are related in one way or the other with the shear strength of the soil. It governs the bearing capacity of soils, the stability of slopes, the earth pressure against retaining structures and many other problems in soil mechanics. Hence, study of the shear strength of soil is very important for stability of the structure. This Research Review Report highlights the various specimen size from smaller to bigger diameter (i.e., 30 to 110 mm dia.) adopted by several codes and standards to determine shear strength parameters under different drainage condition. It is seen that standards have included bigger size of specimen to determine shear strength, although obtaining, handling and testing with smaller size is much easier. The influence of specimen size and its scale effect studied by various researchers are presented and discussed in literature review, section 11. Soil is a natural material. Its properties changes with the change in the environment, loading and drainage conditions. The properties of the soil depend not only on its type but also on the conditions under which it exists. Consequently, soil is a highly heterogeneous and unpredictable material. Accurate assessment of shear strength parameters is required for analysis and design of soil structures. However, the size of the specimen used to determine shear strength parameters can have a marginal or significant impact on the parameters selected for analysis, making it difficult to extrapolate specimen size in laboratory to large size in real field situations. BIS codes also employs four different specimen diameter (D) i.e., 38, 50, 70, 100 mm with height equal to twice the nominal diameter. The behavior of particular soil from different studies are often compared without due attention to the differences in specimen size and its effects on soil shear behavior. Therefore, it becomes important to review the literature, codes, published papers etc., on specimen size and its scale effects on shear strength parameters obtained in laboratory test and understand its relationship.

2.0 SHEAR STRENGTH OF SOIL

In soil engineering, the shear strength parameters are crucial and useful for design work to produce safe and economic geotechnical structure design. The shear strength of the soil is its maximum resistance to shear stresses just before the failure (rupture or sliding). The shear stress is the result of gravity forces from the soil mass and any external loads (e.g. reservoir loads, equipment loads, seismic loads etc.). When the shearing stress reaches its limit value due to the failure of a loaded soil mass, soil deformation is caused. Movements of the wedge soil, behind a retaining wall or sliding in an earth embankment are some of the forms of shear failure. An improper estimation can constitute a serious damage to both property and life. Soil shear strength is derived from two main components: cohesion '*c*' and internal friction angle 'ϕ'. Cohesion is a component of the shear strength, which is independent of the normal stresses applied; the origin of this phenomenon is due to the grouting between the particles, chemical and ionic attraction between clay particles. For cohesion less soils, the friction angle plays a decisive role in the shear strength and stability behavior. Internal friction angle is developed from the frictional resistance to translocation between the individual soil particles at their contact points. The shear strength in cohesionless soil results from intergranular friction alone, while in all other soils it results both from internal friction as well as cohesion. However, plastic undrained clay does not possess internal fiction.

3.0 STRESS-STRAIN CURVE

Consider an element of soil subjected to a varying shear stress under a constant normal stress. Figure 1 shows a typical shear stress-shear strain curve with strain hardening and strain softening of soil element. Initially, when the shear stress is low, soil behaves like elastic material depicting a linear shear strain, and at a particular stress level, significant plastic shear starts to develop and the point is referred to as "yield". The shearing resistance of the soil increases with the plastic shear strain and material is said to work hardener strain harden. The strain hardening can only increase the resistance to a particular maximum shear stress (τ_f) called the peak value. The yield level is considered to be unstable. In some soils, the maximum shearing resistance decreases after this point and the soil is said to be strain softening or work softening. After a continued large strain, the shearing resistance attains a constant level and the corresponding shearing resistance is called the ultimate stress or residual strength (τ_u).

Maximum shear stress at failure is recorded with respect to each applied normal stress which are employed further to obtain a failure envelope in mohr circle, presented in section 4.

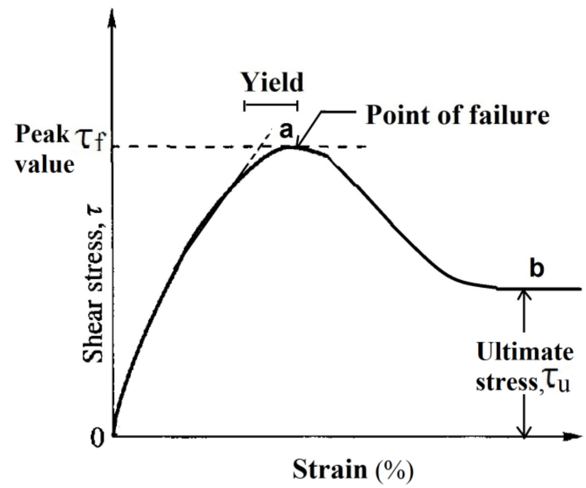


Fig. 1. Typical strain hardening and strain softening

4.0 THE MOHR-COULOMB EQUATION

The stress state that act on a point can be represented graphically in a coordinate system. The Mohr circle can be constructed from the normal stress and shear stress, and two principal stresses (σ_1 and σ_3) as shown in Figure 2. The failure occurs when the stresses are such that Mohr circle touches the failure envelope. Mohr-Coulomb failure theory is valuable function in analysis of the shear strength of soil. According to Mohr, the failure is caused by a critical combination of the normal and shear stress.

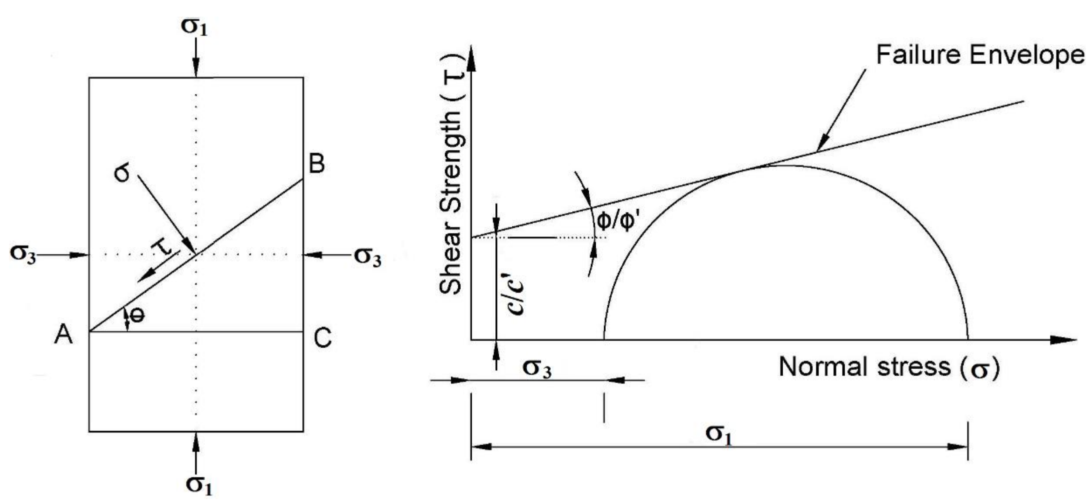


Fig. 2. Mohr's graphical representation of stresses

Mohr-Coulomb failure envelope is represented by a straight line. It represents shearing resistance of soil linearly related with'σ'. Here 'c', equals to the intercept on τ-axis and 'ϕ' is the angle which the envelope makes with the σ-axis.

$$\text{Total shear strength, } \tau = c + \sigma \tan\phi$$

For an ideal pure friction material, failure envelope passes through the origin and for purely cohesive (plastic) material; failure envelope is parallel to the σ- axis. Composite soil having both cohesive as well as frictional material is called c- ϕ soils as shown in Figure 3.

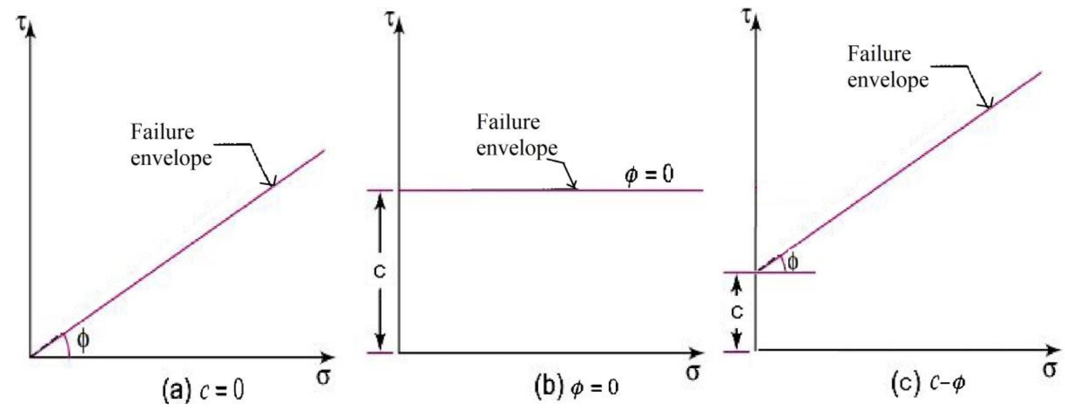


Fig. 3. Mohr coulomb envelope for three type of soil (c=0, ϕ =0 & c- ϕ)

Parameter 'c' and 'ϕ' depends upon the number of factor such as water content, drainage conditions and conditions of testing. Actual stresses which control the shear strength of a saturated soil are effective stresses and not the total stresses. Thus above equation is modified as revised Mohr-Coloumb equation for the shear strength of soil as.

$$\text{Effective shear strength, } \tau' = c' + (\sigma - u)\ \tan\phi'$$

Where,

c/c' = Cohesion for total and effective stress

ϕ/ϕ' = Angle of internal friction for total and effective stress

σ = Normal stress

$\sigma' = (\sigma - u)$ = effective normal stress

σ_1= Major principle stress

σ_3 = Minor principle stress

5.0 LABORATORY METHODS TO DETERMINE SHEAR STRENGTH PARAMETERS

The measurement of shear strength of soil involves certain test observations at failure with the help of which the failure envelope can be plotted corresponding to a given set of conditions. The shear strength parameters c and ϕ can be determined by the following methods:

a) Direct Shear Test
b) Triaxial Compression Test
c) Unconfined Compression Test

5.1 Different Drainage Conditions

Depending upon the drainage, there are three types of conditions as explained below:

1. Unconsolidated-undrained condition: During this test no drainage is permitted during consolidation stage and shearing stage. The test can be conducted quickly in a few minutes, hence sometime called quick test. The shear strength which obtained from this test is total shear strength

2. Consolidated-undrained condition: During this test condition, the specimen is allowed to consolidate in the first stage. The drainage is permitted until the consolidation is complete. In the second stage, when the specimen is sheared, no drainage is allowed. During shearing, pore water pressure is noted which helps to derive the total and effective shear strength. This test is also known as CU test.

3. Consolidated-drained condition: During this test condition, the drainage is permitted in both the stages. The specimen is consolidated in first stage. When consolidation is complete, it is sheared at a very low rate to ensure that fully drained condition exists and the excess pore water is zero. This test is also known as CD test.

The parameters c and ϕ are not fundamental properties of the soil; they may simply be considered coefficients derived from the geometry of the graph obtained by plotting shear stress at failure against normal stress. They vary with drainage conditions of the test.

5.2 Method of applying shear force

The shear force is employed either by increasing the shear displacement at a given rate or by increasing force at a given rate.

1. Strain controlled tests: The test is conducted in such a way that the shearing strain increases at a given rate. Normally the rate of increasing of the shearing strain is kept constant and the specimen is sheared at a uniform rate. Shear force acting on the specimen is measured indirectly using proving ring or load cell.
2. Stress controlled tests: The shear force is increased at a given rate. Generally, the rate of increase of the shear force is kept constant. The shear load is increased such that the shear stresses increase at a uniform rate. The resulting shear displacements are obtained by means of a dial gauge.

6.0 DIRECT SHEAR TEST

This is the most widely used method of shear testing. It consists of forcing a soil specimen to fail along a predetermined shear plane (horizontally) and to measure the resistance to shearing deformation. A relationship between shear stresses and normal stresses on horizontal slip plane is used to establish the Mohr-Coulomb law of shearing strength, viz: $\tau = c + \sigma \tan\phi$.

Apparatus: A direct shear test is conducted on a soil specimen in a shear box which splits into two halves along the horizontal plane at its middle portion. The shear box is made up of brass. A square box of size 60 x 60 x50 mm is commonly use. The box is divided horizontally such that the dividing plane passes through the centre. The box is provided with grid plates which are toothed and fitted inside it. The gripper plates are plain for undrained tests and perforated for drained tests. The porous stones are placed at the top and bottom of the specimen in drained tests. The normal load from loading yoke is applied on the top of the specimen through a steel ball bearing upon the pressure pad. The lower half is fixed to the base plate and held in position in the large container. The large container is supported on roller which can be pushed forward at a constant rate by a gear system operated by electric motor.

A proving ring is fitted to upper half of the box to measure the shear force. The shear displacement is measured with a dial gauge fitted to the container. Another dial gauge is fitted to the top of the pressure pad to measure the change in thickness of the specimen. The direct shear test s conducted on cohesionless soils for any one of the three drainage conditions as explained above. Figure 4 shows the strain controlled direct shear apparatus.

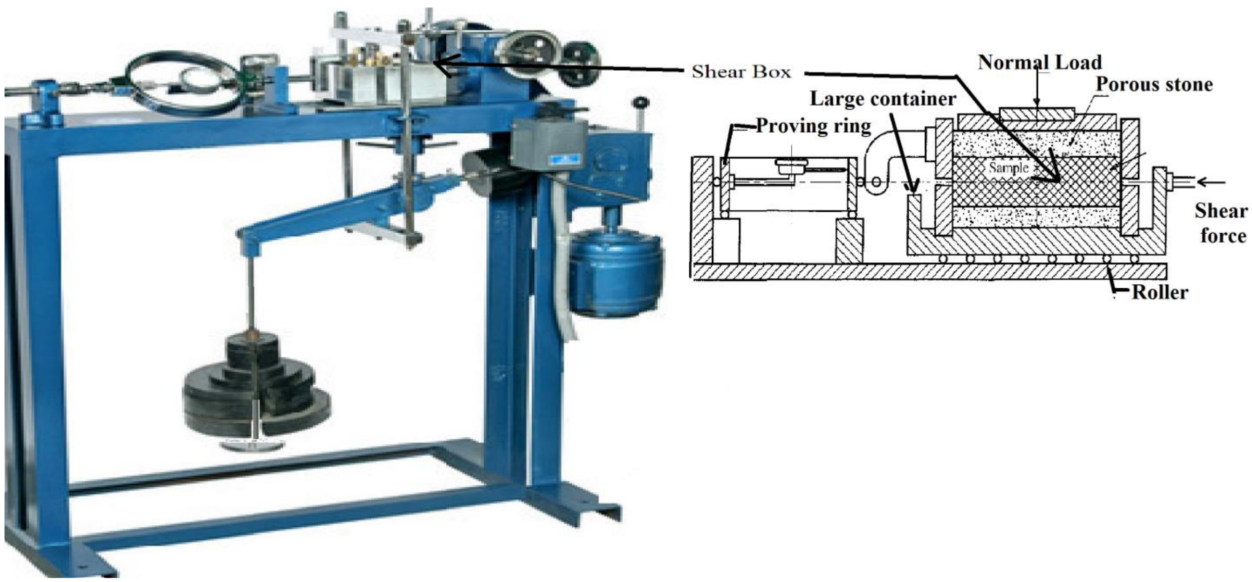


Fig. 4. Direct Shear Apparatus

7.0 TRIAXIAL COMPRESSION TEST

The triaxial test apparatus and schematic diagram are shown in Figure 5. Apparatus consists of a strain controlled loading frame, a triaxial cell (for 38 mm dia. to 100 mm dia. specimen), pressure regulators, read out attachment (load cell, LVDT and pressure transducer), pore pressure apparatus, volume change measuring device etc. The loading can be done at different rates to simulate fixed conditions. Triaxial cell is a perspex cylinder, attached to the base with rubber seals to make it water tight. The pressure cylinder maintains constant pressure in the triaxial cell. Drainage is controlled through an outlet valve.

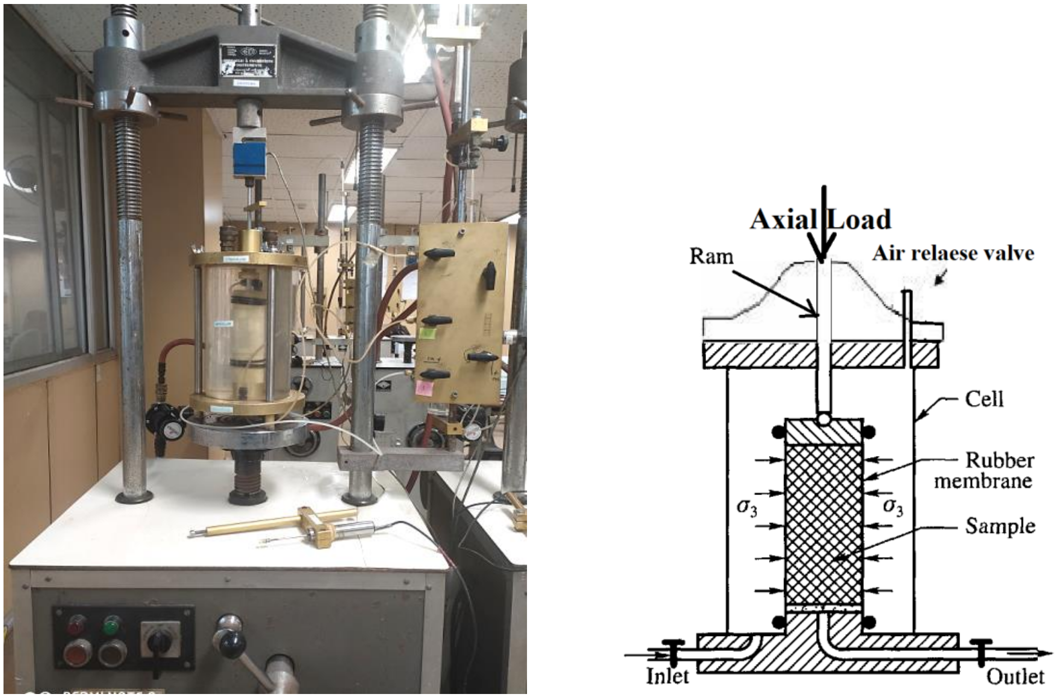


Fig. 5. Triaxial test apparatus (Courtesy: CSMRS)

In the standard triaxial test, a cylindrical sample is fitted between rigid caps and covered with latex membrane. It is then placed in a perspex cell which is filled with water. The sample is subjected to confining pressure (σ_3) by applying pressure to water in the cell. An additional (deviator) stress ($\sigma_d = \sigma_1 - \sigma_3$) is then applied by loading the sample through a ram and steadily increased until the specimen fails. During triaxial loading, the outlet valve may be kept open to induce drained condition or may be kept closed for an undrained loading. The drainage outlet is connected to the pore pressure apparatus for measuring pore pressure or the volume measuring device to measure change in volume during drained loading.

Principal Stresses in Triaxial Test:

The triaxial test can be considered as conducted in two stages as shown in Figure 6. In this first stage cell water pressure (σ_3) is applied on the specimen and in second stage deviator stress ($\sigma_d = \sigma_1 - \sigma_3$) is applied on the sample till the failure of specimen.

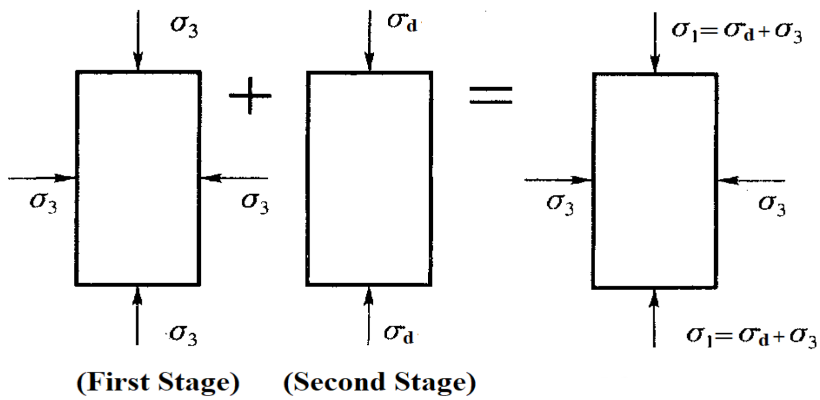


Fig. 6. Cylindrical specimen subjected to stresses different stages

Here,

σ_1= Major principle stress acting on major principle plane

σ_3 = Minor principle stress acting on minor principle plane (confining pressure)

σ_d= Deviator stress ($\sigma_d = \sigma_1 - \sigma_3$)

8.0 UNCONFINED COMPRESSION TEST

The unconfined compression test is a special case of triaxial compression test in which the all round confining pressure is zero (σ_3 = 0). The tests are carried out only on saturated samples which can stand without any lateral support. The test is, therefore, applicable to cohesive soils only. The test is an undrained test and is based on the assumption that there is no moisture loss during the test. The cylindrical specimen of soil is subjected to major principal stress σ_1 till the specimen fails due to shearing along a critical plane of failure.

Apparatus: It consists of a small load frame fitted with a proving ring to measure the vertical stress σ_1 applied to the soil specimen. Figure 7 shows an unconfined compression testing machine. The deformation of the sample is measured with the help of a separate dial guage. The ends of the cylindrical specimen are hollowed in the form of cone. The cone seating reduce the tendency of the specimen to become barrel shaped by reducing end-restraints. During the test, load versus deformation readings are taken and graph is plotted. The unconfmed compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils. The cross-sectional area A at any stage of loading of the sample may be computed on the basic assumption that the total volume of the sample remains the same. As a result corrected area $A = A_0 / (1 - \varepsilon)$. The axial stress at any stage of loading is obtained by dividing the total axial load by the cross-sectional area. The cross-sectional area of the sample increases with the increase in compression. In an unconfined compression test, $\sigma_3 = 0$. The major principal stress σ_1 is equal to the deviator stress and written as $\sigma_1 = P/A$, where P = axial load and A = corrected area.

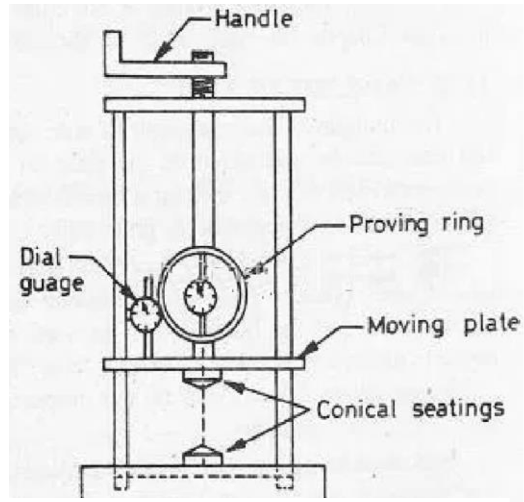


Fig. 7. Unconfined Compression Testing Machine (Arora K R, 2003)

As the minor principal stress is zero, the Mohr circle passes through the origin. The failure envelope is horizontal ($\phi_u = 0$). The cohesion intercept, C_u is equal to the radius of the circle as shown in Figure 8.

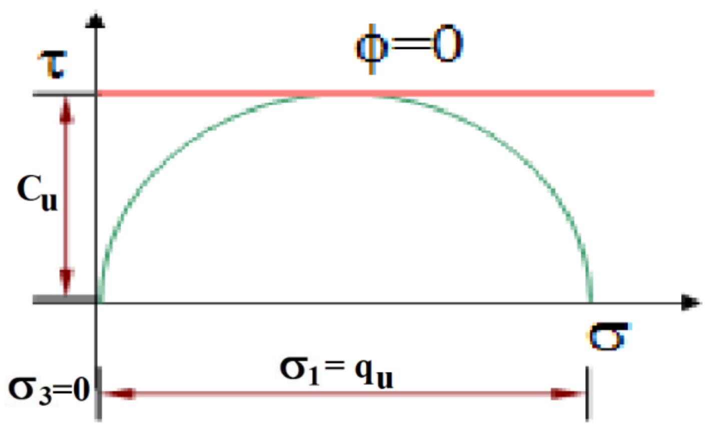


Fig. 8. Mohr circle for unconfined compression test

9.0 RECOMMENDED SIZE OF SPECIMEN BY TECHNICAL AUTHORITIES

An L/D ratio of between 2 and 3 is generally recommended by accepted scientific and technical authorities. However, there are only few published works on the effect of specimen size on soil strength on fine-grained (cohesive and semi-cohesive).

One of the most important requirements when determining soil shear strength parameters is to create samples that comply with the requirements of appropriate standards. The required size of a soil specimens used in a triaxial compression test under different drainage conditions has been specified by a number of authorities in their standards are as follow:

1. The IS codes IS 2720: Part 11- 1993 (Reaffirmed 2016) for Unconsolidated Undrained (UU) Triaxial test on saturated cohesive soils specifies that this test is limited to specimens in the form of right cylinders of nominal diameter 38, 50, 70 and 100 mm and of height approximately equal to twice the nominal diameter . In case of remoulded samples; ratio of diameter of specimen to maximum size of particle in the soil should not be less than 5.
2. IS codes IS 2720 : Part 12-1981 (Reaffirmed 2016) for Consolidated-Undrained (CU) triaxial test on all types of soils specifies that the test is limited to specimens in the form of right cylinders of nominal diameter 38, 50, 70 and 100 mm with height twice its diameter. The ratio of diameter of the specimen to the maximum size of the particle in the soil shall not be less than 5.
3. ASTM D 2850-15 for Unconsolidated-Undrained triaxial test on cohesive soils specifies that specimens shall be cylindrical and have a minimum diameter of 3.3 cm (1.3 in.). The average height-to-average diameter ratio shall be between 2 and 2.5. The largest particle size shall be smaller than one sixth the specimen diameter.
4. ASTM D 4767-11 (Reapproved 2020) for Consolidated-Undrained triaxial test on saturated cohesive soils specifies that specimens shall be cylindrical and have a minimum diameter of 3.3 cm (1.3 in.). The average height-to-average diameter ratio shall be between 2 and 2.5. The largest particle size shall be smaller than one sixth the specimen diameter.
5. ASTM D 2166 -06 for Unconfined compressive strength of cohesive soils specifies that the specimens shall have a minimum diameter of 3.0 cm (1.3 in.) and the largest particle contained

within the test specimen shall be smaller than one tenth of the specimen diameter. For specimens having a diameter of 72 mm (2.8 in.) or larger, the largest particle size shall be smaller than one sixth of the specimen diameter. The height-to-diameter ratio shall be between 2 and 2.5.

6. BS 1377-8:1990: Methods of test for soils for civil engineering purposes. Shear strength tests (effective stress) (withdrawn on 13 April 2018 and replaced by BS EN ISO 17892-9:2018 recommends that the tests apply to specimens in the form of right cylinders of nominal diameters usually from 38 mm to approximately 100 mm and of a height approximately equal to twice the diameter.
7. BSI 1377-7 (BSI 1990) recommends that the test specimen should have a diameter of between 35 and 100 mm and a length-to-diameter ratio of 2.
8. JIS A 1216 (Japanese Standards Association 1993) Method for Unconfined Compression Test of Soils requires that the cylindrical test specimen’s diameter and length are 35 and 80 mm, respectively.
9. TS 1900-2 (Turkish Standards Institution 2006) recommends that the cylindrical test specimen should have a diameter of 50 mm preferably and a height-to-diameter ratio of 2.
10. AS 1289.6.4.2: 2016 (Australian standards 2016) for consolidated undrained triaxial tests specifies specimens in the form of right cylinders with height (H) to diameter (D) ratio of 2 ± 2%. 1998 code is superseded by 2016 code, where H/D ratio between 2 and 2.5 is omitted.
11. GEOSPEC 3: 2017 (Hongkong standard) in its clause no. 15 for triaxial tests specifies, nominal diameter of 70 mm to 110 mm and height to diameter ratio of 1.9 to 2.1.
12. Das (2002) suggests that the cylindrical test specimen should have a diameter of 1.4” (35 mm) and a length to diameter ratio of 2–3.

As mentioned above, although some of the values of soil specimen size and L/D ratio recommended by the authorities are similar to the corresponding values recommended by other authorities, there is no clear consensus in the literature and among the standards.

10.0 OBJECTIVE OF THE STUDY

Shear strength of soil is basic parameter in the studies like liquefaction, slope stability and shallow foundation design, etc. The triaxial and the direct shear tests are at present the most common tests for determining the soil shear strength parameters in laboratory. The triaxial test is acknowledged to be the most widely employed method for evaluating the soil shear strength, due to complete control over the drainage condition and more uniform stress distribution across the soil samples. Various methods employed to measure shear strength consist of samples from smaller to bigger specimen size. Codes and standard has also incorporated flexibility in using various sizes of specimen with L/D ratio (see section 9). Sometimes larger samples obtained from site may have fracture, latent cracks and inhomogenety as compared to smaller specimen. These fissures in undisturbed (UD) samples give true representation of field condition. Testing these bigger specimen sizes in laboratory may provide actual behavior of stress-strain curve. Smaller size of specimen may have reduced fissures and cracks which may increase strength of stress-strain curve significantly. In certain cases there may be some practical difficulties in obtaining UD samples; in such cases samples shorter than specified in the standards are only available for laboratory test. It is also imperative to understand that bigger specimens irrespective of will exhibit a stiffer compression behavior and smaller compressibility during compression as compared to the smaller specimens, and therefore may mobilize smaller shear strength and effective friction angles. Such specimen will also experienced less volumetric strains at the same effective confining pressure. The smaller specimen will exhibit more compressible behavior during consolidation, therefore larger shear strength parameters may be mobilized in smaller specimen during shearing. Shear strength increases with increasing confining pressure and therefore possibility of size effect can be more significant with higher confining pressure. It cannot be disregarded that smaller specimen can achieve good amount of consolidation and higher compressibility as compared to bigger specimen. Consequently shear strength may increase with decrease in specimen size. The choice of bigger sample size is thus a more accurate representation of soil strength conditions with respect to field deformation. BIS codes has also provided flexibility to use four different specimen size from smaller to bigger diameter (D) i.e., 38, 50, 70, 100 mm to determine shear strength parameters. The samples normally tested are of size 38 mm in diameter and 76 mm height, although samples with larger in size (i.e., 50, 70, 100 mm dia. size) can also be tested with correct set-up of equipment The behavior of particular soil from different studies are often compared without due attention to the differences in specimen size and its effects on soil shear behavior.

The impact of specimen size on the shear strength parameters and design is overlooked in engineering practice. Therefore, the main objective of this report is to review the literature, codes, published papers etc., on the influence of specimen size and its scale effects on shear strength parameters obtained in laboratory test and submit conclusion and future scope for the study.

11.0 LITERATURE REVIEW

Jonas et al et al (2009) specified number of factor effecting deformation characteristic i.e the sample height, the specimen self-weight factor, the actual sample ends conditions etc., and proposed H/D ratio to 1 for reducing an inaccuracy related to the non-uniform stress-strain distribution during testing. Sarunas et al (2019) has also shown that the shear strength parameters are influenced by the clay specimen size and is more significant with higher confining pressure. Aktas. 1991 carried out a series of unconsolidated undrained triaxial tests on clay specimens with the same length to diameter ratio (L/D = 2.0) but different diameters (36 and 50 mm), increasing the diameter was found to decrease the undrained friction angle (ϕ_u) and E_{50} and reduces C_u by a negligible Sivadass et al (2013) studied that the effect of sample size is more significant at higher confining pressures. From the result, total and effective peak friction angles decreases with increasing sample size. Chew et al (2011) conducted traxial test on marine clay and found out that undrained shear strength of large diameter sample is 20-70 % lower than that obtained from small diameter sample. Unlike in rock engineering, research into the effect of specimen size or the ratio of length to diameter on soil shear strength is scare. As a result an effort is made to understand weather influence of specimen size is possible in case of soils. Detail study of literature review is presented below to understand influence of specimen size on shear strength and deformation behavior of soil.

11.1 Effect of large diameter sample testing for offshore site investigation, Chew et al (2011)

Chew et al (2011) conducted testing of large diameter sampling for soil investigation work of the soft marine clay. Author have express that small size sample for determination of shear strength and consolidation may suffer from the limitations of the small sample size which may not truly represent the fabric and structure of the soils of the site. The in-situ soil is not as uniform or homogenous as represented by this small soil sample. Hence, in conjunction with long-term mega near-shore development project, a large diameter sampling and testing research project was envisaged.

In this project, large diameter sampler to collect high quality undisturbed soil sample of up to 200 mm diameter and up to 1000 mm height was made. The aim of this research was to study the effect of sample size on strength and consolidation properties of marine clay so that appropriate properties of marine clay can be used for design of foundation system for offshore structure.

The large diameter triaxial test apparatus used in this study is shown in Figure 9. The whole set-up consists of a strained controlled triaxial cell with accessories, an autonomous data acquisition unit (ADU). It was thus setup and tested in large diameter triaxial cell using similar procedure as UU test as per BS 1377-7, 1990 and K. H Head, 1998. Three types of test samples (labeled L-L, L-S & S-S) were prepared to evaluate the effect of sample size and the soil fabric onto its strength and consolidation characteristics. Samples were all obtained from same or nearer bore hole for fair comparison. Details of sample are listed in Table 1.

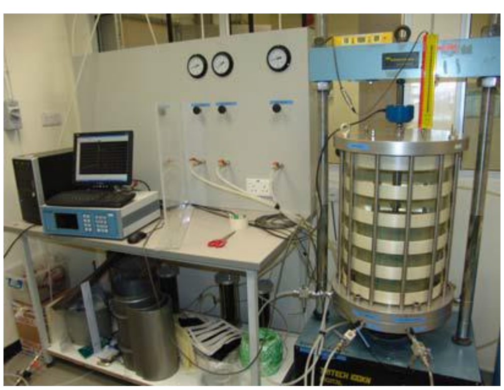


Fig. 9. Large diameter triaxial apparatus

Table 1. Large and small sample for triaxial and consolidation test

Given code to sample	Sampler	Size of Triaxial sample	Size of Oedometer sample
L-L	Large diameter sampler	200 mm Φ	150 mm Φ
S-L	250 mm Φ	38 mm Φ	70 mm Φ
S-S	Conventional small diameter sampler 102 mm Φ	38 mm Φ	--

The results from the large diameter Triaxial and Oedometer sample tests were compared with the conventional small diameter Triaxial and Oedometer sample test. The results showed significant difference between the two tests. In addition to the mechanical tests, X-ray diffraction and SEM analysis tests were also carried out to study microstructure of undisturbed clay sample. The micro-structural studies indicated that the constituent particles were arranged in an open network, or flocculated structure.

Results and Conclusion:

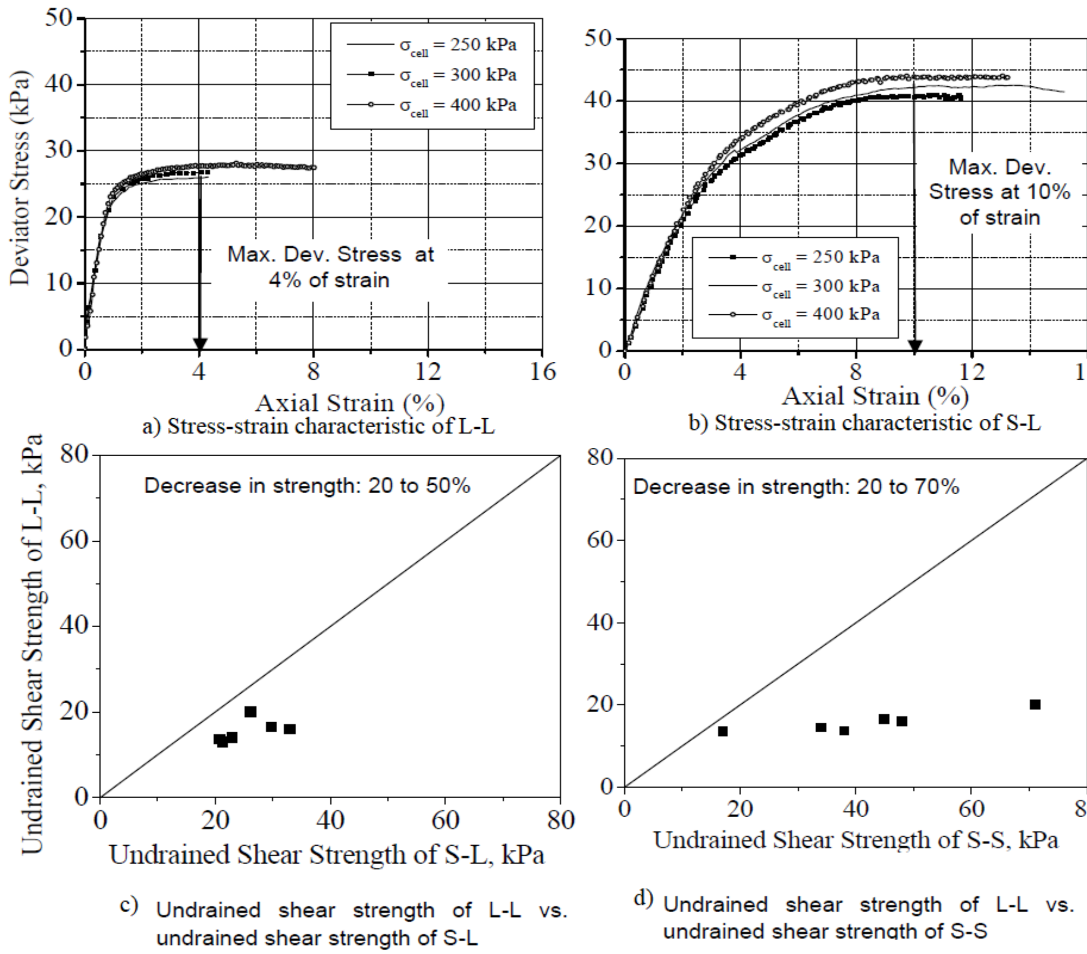


Fig. 10. Comparison of Deviator-stress graph and undrained shear strength

Figure 10 (a & b) shows the deviator stress-strain behavior of LL and S-L respectively. These figures show clearly that the maximum deviator stress of L-L was attained faster than S-L. The strain of L-L and S-L at maximum deviator stress was about 4% and 10% respectively. The figures also show that

L-L was stiffer than S-L for the strain range of 0 to 1%. However maximum deviator stress of L-L was much lower than S-L.

Figure 10(c) clearly shows that undrained shear strength of L-L was lower than S-L. The decrease in undrained shear strength of L-L was about 20% to 50% of S-L. This clearly indicates the effect on non-representativeness of small diameter soil sample which is not conservative in terms of strength. Further decrease in undrained shear strength was more pronounced (i.e. 20% to 70%) when undrained shear strength of L-L was compared with S-S indicating the combined effect (Figure 4d).

11.2 Influence of specimen size on unconfined compressive strength and deformation characteristics of cohesive soils, Kamei et al (1991)

Kamei et al (1991) studied the effect of specimen size on unconfined compressive strength and deformation characteristics of cohesive soils. Unconfined compression tests are performed on two kinds of reconsolidated cohesive soils as shown in Table 2.

Table 2. Index properties of soil samples

Soil sample	Gs	w_L(%)	w_P(%)	I_P	Sand(%)	Silt(%)	Clay(%)
Kaesa Soil	2.707	54.8	18.7	36.1	44.8	31.3	23.9
Kamimura Soil	2.612	49.4	35.9	13.5	20.1	38.3	41.6

Cylindrical specimens with 1.0, 2.0, 3.5, and 5.0 cm diameters (D) are used which have been molded so that the L/D ratios become 1.0, 1.5, and 2.0. The unconfined compression strength increases with the decrease in the diameter and L/D ratio as shown in Figure 11. By using correction coefficient, the deformation modulus E_{50} increases with the decrease in the diameter and L/D ratio.

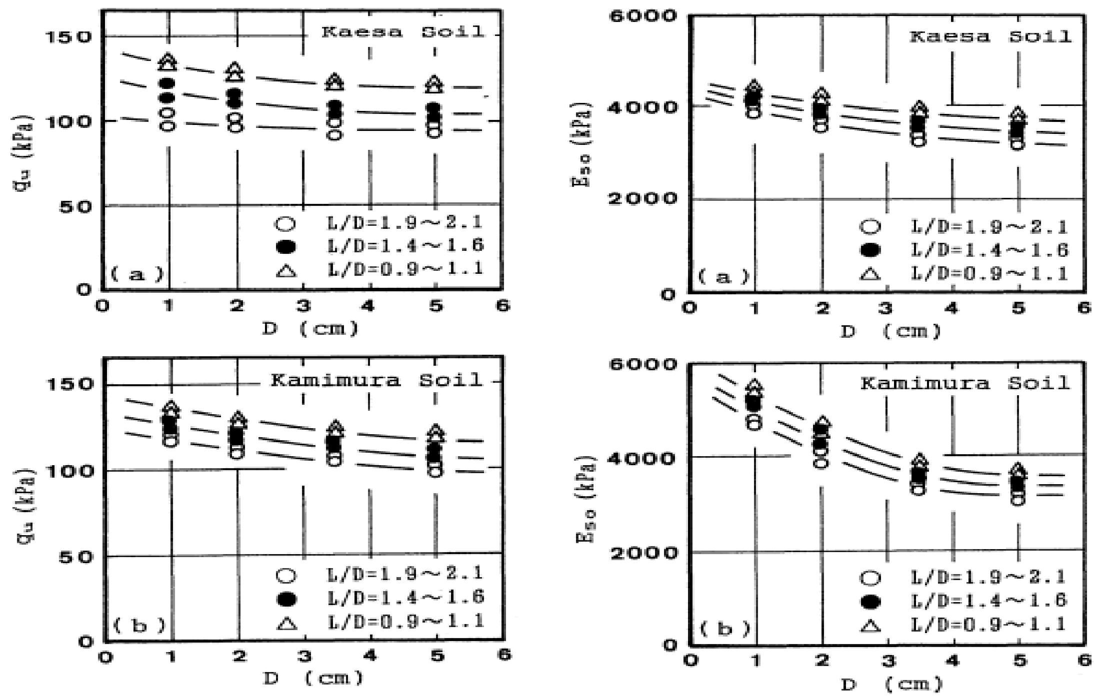


Fig. 11. Relationship between q_u vs. D and E_{50} vs. D

Axial compression strength q_u and E_{50} of the standard size specimens can be evaluated to some extent by q_u and E_{50} of sizes other than standard. It can be considered that the correction coefficient of q_u and E_{50} can be expressed as functions of L/D ratio (see Figure 12).

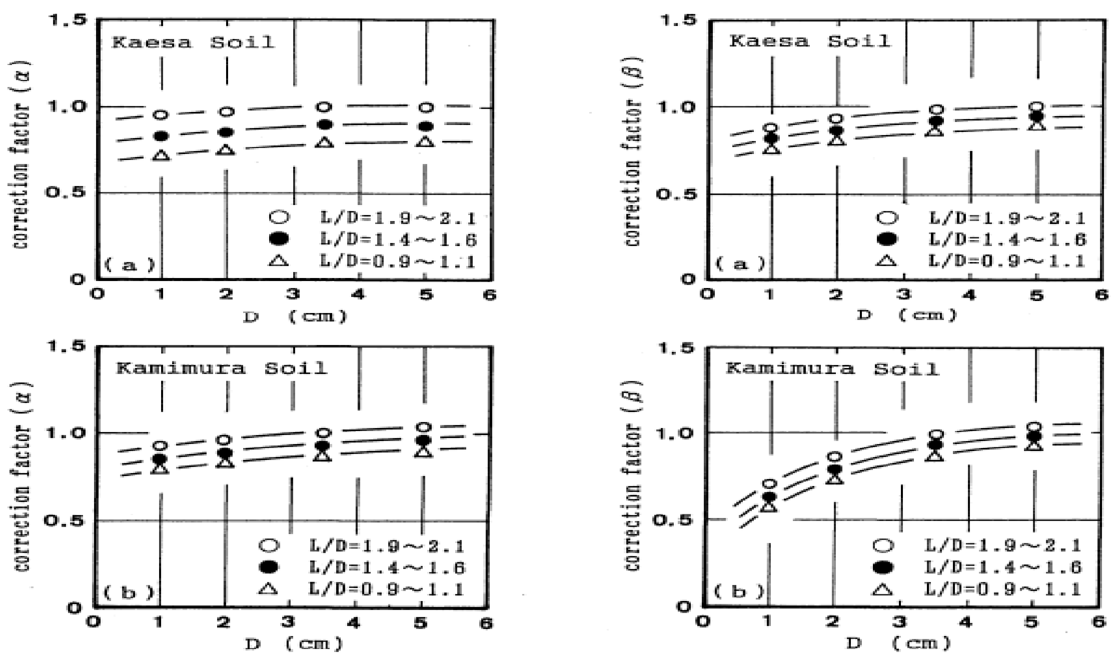


Fig. 12. Relationship between correction factor and D for qu and E_{50}

Therefore, it can be said that the results of experiments made with small size specimens can be valued to a certain degree from the engineering point of view if the correction factor is know. Table 3 provide the correction factor for both the cohesive soil with L/D ratio.

Table 3. Correction factor for both the cohesive soil with L/D ratio

a. Correction factor for specimen size in unconfined compressive strength in the present study

Soil Sample	L/D=1.9~2.1	L/D=1.4~1.6	L/D=0.9~1.1
Kaesa Soil	1.00	0.86	0.76
Kamimura Soil	1.00	0.92	0.85

b. Correction factor for specimen size in deformation modulus in the present study

Soil Sample	L/D=1.9~2.1	L/D=1.4~1.6	L/D=0.9~1.1
Kaesa Soil	1.00	0.95	0.91
Kamimura Soil	1.00	0.97	0.90

The following conclusions were obtained based on the results obtained from the present study:

i) Both the unconfined compressive strength and deformation modulus increased with the decrease in specimen size.

ii) Method of accounting for the effect of specimen size on the unconfined compressive strength and deformation modulus is proposed

11.3 Effects of specimen size on unconfined compressive strength properties of natural deposits, Takaharu Shogaki (2007)

The unconfined compressive strength (q_u) is widely used in Japan for stability analysis of clay foundations under undrained conditions. This is mainly because the average value of $q_u/2$ well describes the undrained shear strength on the failure surface in a ground and in addition to this; the testing procedure for the q_u value is simple and economical. The specimen size usually used in Japan for unconfined compression tests (UCT) is the O (or Ordinary size) specimen, 35 mm in diameter and 80 mm in height. However, for O specimens, the preparation for testing is difficult due to latent cracks or homogeneity.

Takaharu Shogaki in the year 2007 studied the effect of specimen size on unconfined compressive strength properties of natural clay, organic and mudstone deposits in small size (15 mm in dia. and 35 mm in height) and ordinary size (35 mm in dia. and 80 mm in height) obtained from 75 mm dia. sampler as shown in Figure 13.

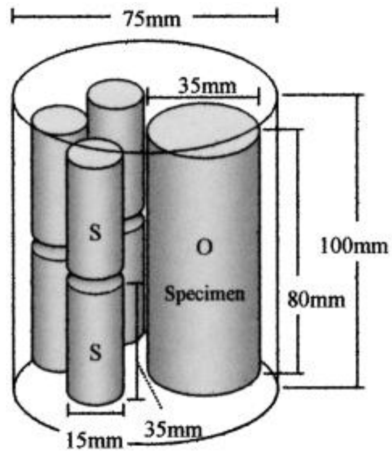


Fig. 13. Small size (15 mm dia.) & ordinary size (35mm dia.) sample obtained from 75 mm dia. sampler

The value of q_u was determined to be the maximum stress corresponding to axial strain of 15 % or less. The secant modulus, E_{50} is given by $(q_u/2)/\varepsilon_{50}$, in which ε_{50} is the strain at the value of $q_u/2$.

Results and Discussion

Pore water pressure under shearing differs by specimen size, namely the pore water pressure of O specimens changed from minus to plus under an axial strain of from 0.5 % to 1 % before maximum axial stress and the amount of change of pore water pressure was greater than that of S specimens. From the relationships between axial stress and axial strain, as shown in Fig. 14, the deviation stresses are almost similar for the O and S specimens. However, there is a large difference in the effective stress paths between the O and S specimens. In some organic soil the plasticity index and natural water content are as high as 150-370 % and 136-592 % respectively and the qu value is as small as 25-33 kPa.

Conclusions

The standard deviations of the ratios of q_u and E_{50} values of the S specimens to O specimens were in the range of 0.09 to 0.16. The 10 % variation from the mean value reflects the soil homogeneity since the coefficients of variations of the undrained shear strength for the undisturbed and reconstituted soils were 8-17 % (Matsuo and Shogaki, 1988).

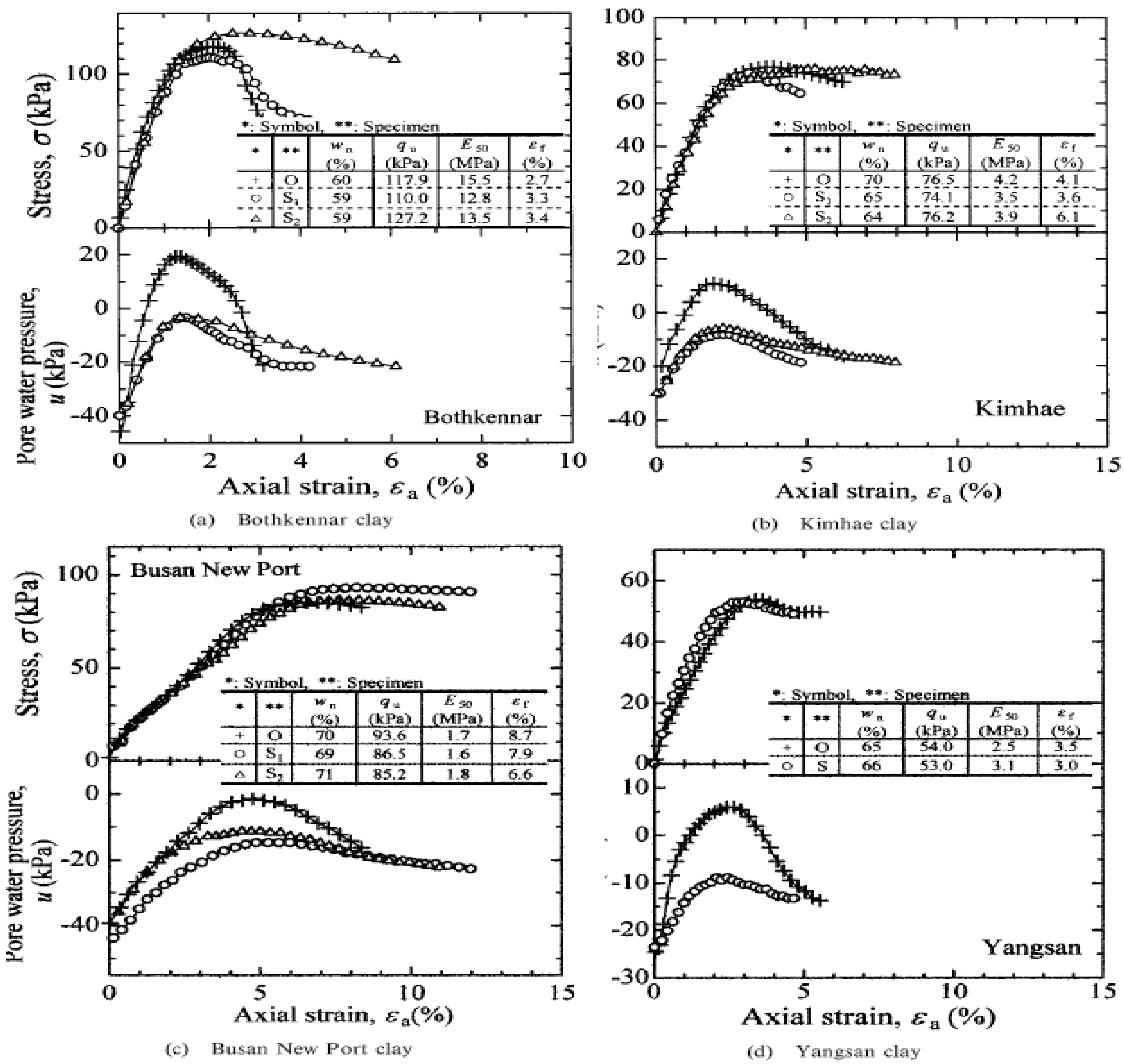


Fig. 14. Result of axial stress, pore water pressure and axial strain for clay deposits

11.4 Specimen Diameter Influence on Effective Shear Strength Parameters in Triaxial Tests, Fernando et al (2016)

Fernando et al (2016) studied effective shear strength parameters, measured in a triaxial tests on remolded sand soils specimens with 50mm and 38mm diameters. In this study, twenty-one consolidated undrained (CU) tests were performed on remolded specimens with consideration of confining pressure of 100, 200, and 300 kPa. The result shows that the angle of internal friction of the 50mm and 38mm specimens presented the value of 33º and 29º. While the results for the cohesion intercept showed values of 18.25 and 68.12 kPa. The results show that the specimen diameter influence on effective shear strength parameters for triaxial tests on different diameters specimens is small regarding the internal friction angle. On the other hand, soil cohesion presented a significant variation.

Tri-axial Test

The triaxial test is carried out in a cell on a cylindrical soil sample having a length to diameter ratio of 2. The usual sizes are 76 mm x 38 mm and 100 mm x 50 mm. Three principal stresses are applied to the soil sample, out of which two are applied water pressure inside the confining cell and are equal. The third principal stress is applied by a loading ram through the top of the cell and is different to the other two principal stresses.

Material and Method

The soil used in this study was classified as a SM-SC (Silty sand) according to the Unified Soil Classification System. Table 4 shows the basic properties of soil. The soil samples were compacted in a cylindrical mold with moisture content of 10.5% and 19.17 kN/m³ density and set-up in a triaxial apparatus in different stages are shown in Figure 15. The CU triaxial tests were performed under three different cell pressures of about 100, 200, and 300 kPa using specimens of 50mm and 38mm diameter. The specimens in the CU triaxial test were sheared with a strain rate of 0.083 mm/min.

Table 4. Basic properties of soil

Moisture Content (%)	Specific Gravity	Particle Size Distribution				Atteberg Limits		Soil Classification (USCS)
		Clay (%)	Silt (%)	Sand (%)	Gravel(%)	Plastic limit (%)	Liquid Limit (%)	
10,5	2,48	14	3	83	0	0	0	SM-SC

Fig. 15. Set-up of specimen in triaxial apparatus

Results and Discussion

From the pick of the best set of specimens having consistent results from Table 5, Mohr-Coulomb failure envelopes were computed for 50 mm and 38 mm diameter specimen as shown in Figure 16.

Table 5. Triaxial results for 50mm and 38 mm diameter samples

50mm diameter samples						
Specimen #	σ3 (kPa)	σd (kPa)	σ1 (kPa)	u (kPa)	σ'1 (kPa)	σ'3 (kPa)
3	100	712	812	-163	975	263
5	100	698	798	-171	969	271
2	200	860	1060	-114	1174	314
6	200	758	958	-147	1105	347
23	200	791	991	-2	993	202
24	200	804	1004	-110	1114	310
4	300	1143	1443	-139	1582	439
7	300	1120	1420	-137	1557	437
38mm diameter samples						
Specimen #	σ3 (kPa)	σd (kPa)	σ1 (kPa)	u (kPa)	σ'1 (kPa)	σ'3 (kPa)
8	100	609	709	-131	840	231
13	100	501	601	-137	738	237
16	100	386	486	-103	589	203
17	100	395	495	-109	604	209
12	200	718	918	-124	1042	324
14	200	547	747	0	747	200
18	200	734	934	-70	1004	270
11	300	699	999	-83	1082	383
15	300	849	1149	-109	1258	409
19	300	611	911	-62	973	362
20	300	948	1248	-84	1332	384
21	300	759	1059	-53	1112	353
22	300	973	1273	-78	1351	378

The result shows that the angle of internal friction of the 50 mm and 38 mm specimens presented the value of 33º and 29º. While the results for the cohesion intercept showed values of 18.25 and 68.12 kPa.

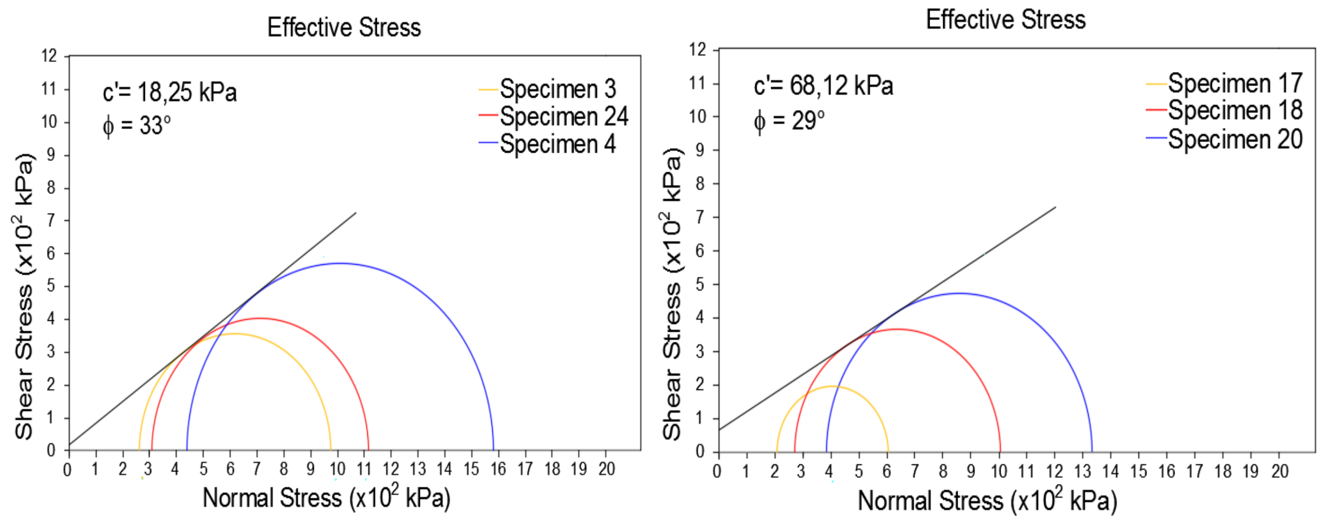


Fig. 16. Mohr-Coulomb failure envelope for 50 mm and 38 mm diameter specimens

Conclusions

The results show that the specimen diameter influence on effective shear strength parameters for triaxial tests on different diameters specimens is small regarding the internal friction angle. On the other hand, soil cohesion presented a significant variation. The results of the triaxial tests showed that 50mm diameter samples presented a cohesion intercept of 18.25 kPa and the internal friction angle of 33º. Meanwhile, the 38mm diameter samples presented a cohesion intercept of 68.12kPa and a 29º internal friction angle. The internal friction angle exposed a 12.12 % difference between the different diameter specimens and cohesion showed a 73.20% divergence, which is a considerable variance.

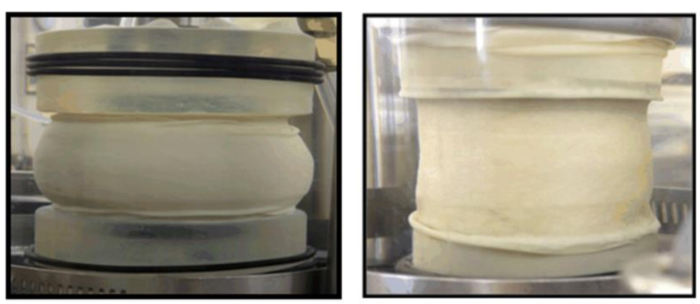
During the tests performance, it was noted a great difficulty on working with the 38mm diameter samples, due to its fragility. Some specimens were ruined during the use of rubber sheath, to a point that the test would not proceed, because the specimens were damaged.The shear strength of the soil is affected by the quality of the sample. Remolded samples will usually present lower values of effective shear strength parameters than undisturbed samples because residual soils are sensitive to disturbances and disruptions incurred during sampling that affect the results of the tests

11.5 Effect of triaxial specimen size on engineering design and analysis, Tarek Omar et al (2015)

Tarek Omar et al (2015), studied the effect of triaxial specimen size on the consolidation, drained and undrained shear behavior of cohesionless soil. The measured sand shear strength and friction angle in triaxial compression tests on three different specimen sizes are used to explain some of the scale effects.

Material Preparation

Triaxial compression tests were performed on sand classified as SP, with different cylinder sizes sealed in a water-tight rubber membrane and confined in a water-filled acrylic cell. Cylindrical specimens of 38, 50, and 70 mm diameters were prepared with equal height and diameter (H/D = 1). All specimens were prepared at an initial void ratio (ei) of 0.821. Average mean particle size (D_{50}), coefficient of uniformity (Cu) and coefficient of curvature (Cc) of the sand is 0.22 mm, 1.71 and 1.07 respectively. Specific gravity of sand particles, maximum and minimum void ration is 2.65, 0.821 and 4.87 respectively. In a triaxial compression test complete specimen saturation was necessary for accurate volume change and pore pressure measurement, the saturation procedure was proceeded with a backpressure saturation phase. A back pressure of 200 kPa was applied to the specimens pore water in order to drive any remaining air into solution. Saturation was verified by ensuring that a Skempton's pore water pressure parameter, B of at least 0.98 was achieved in all specimens. Figure 17 demonstrates the deformation pattern of a loose sand specimen (a) without, and (b) with enlarged and lubricated end platens in triaxial compression tests at 30 % axial strain.



(a) (b)

Fig. 17. Demonstrates the deformation pattern of a loose sand specimen (a) without, and (b) with enlarged and lubricated end platens in triaxial compression tests at 30 % axial strain.

Results and Discussion

Figure 18 presents the compression lines following the consolidation phase for the different specimen sizes tested in this study where the 38 mm specimens display the most compressive response during compression (i.e., steepest compression line) followed by the 50 and 70 mm specimen sizes. In other words, the 70 mm specimens experienced less volumetric strains than the 38 and 50 mm specimens at the same effective confining pressure (p') in all tests. This indicates that larger sand specimens exhibit markedly stiffer compression behavior and smaller compressibility during compression compared to the smaller specimens. Effect of specimen size on ϕ' and S_u from undrained triaxial compression shear tests is also shown in Figure 19.

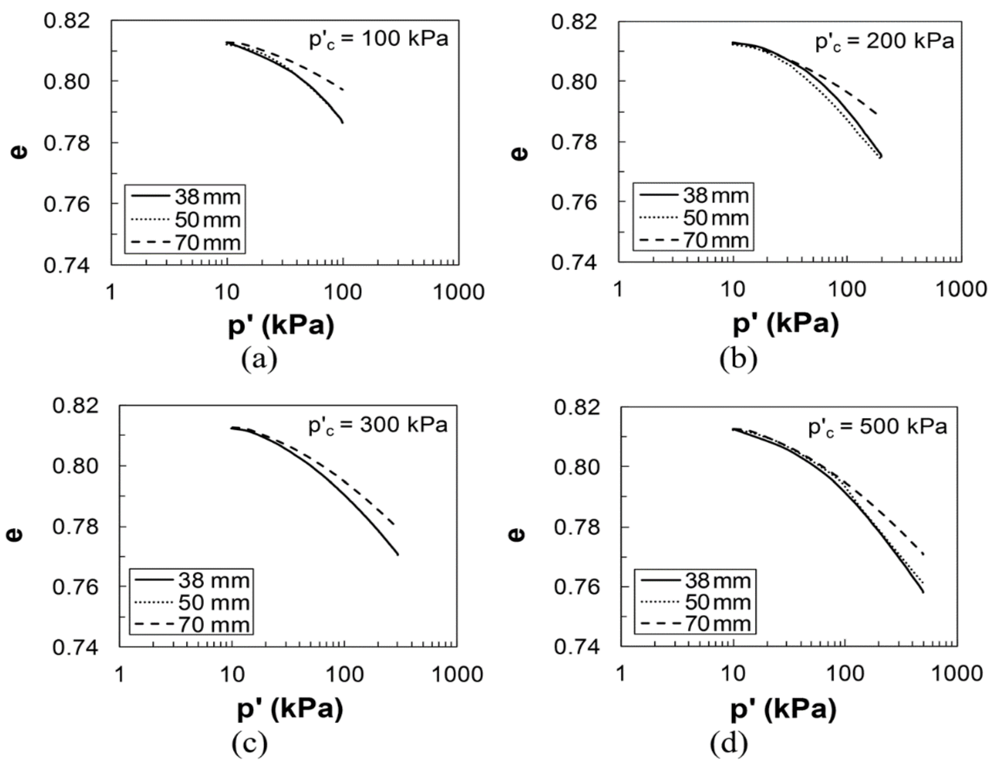


Fig. 18. Effect of specimen size on the compression behavior of loose sand at (a) p' = 100 kPa, (b) p'= 200 kPa, (c) p' = 300 kPa, and (d) p'= 500 kPa

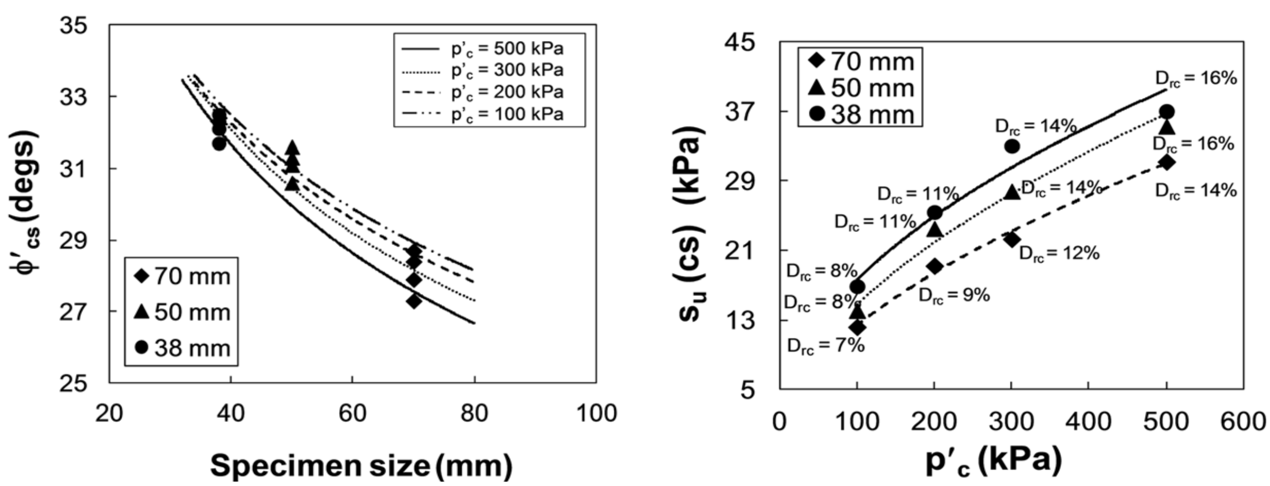


Fig. 19. Effect of specimen size on ϕ' and S_u from undrained triaxial compression shear tests

During shear, the smaller specimens exhibited higher compressibility and mobilized larger friction angles and shear strength as shown in Figure 19. Shear strength, S_u increase not only with increasing p'_c but also with decreasing specimen size.

Sample size effect on lateral earth pressure: The measured ϕ' for different sample sizes indicate lower lateral active earth pressure coefficient, k_a for larger specimens. Accordingly, the ϕ' differences realized as a result of differences in specimen size could significantly affect the lateral earth pressures and therefore the sliding and overturning analysis and design of retaining structures. Therefore, obtaining the soil strength properties by testing of larger specimens allows for building safer structures. These results further explain the reduced cone penetration resistance of larger cones (Eid 1987), and the smaller effective lateral stresses of large diameter piles (Lehane et al. 2005).

Conclusion

The triaxial test result on loose sand indicates that sand behavior was affected by the size of the specimen during compression as well as during drained and undrained shear. The larger specimens presented less compressible isotropric compression behavior than the smaller specimen The smaller volumetric fraction of the more compressible sand zones near the specimen sides and the stiffer regions adjacent to the specimen caps as well as the longer and larger number of shear bands and failure planes in the larger specimens could have produced the observed specimen size effects. The shear strength parameters obtained from testing samples with different sizes could largely affect almost all geotechnical engineering applications in which soil shear strength and friction angle play important roles. Testing larger specimen sizes provides better representation of field shear and deformation behavior, and therefore building safer structures.

11.6 Effect of length-to-diameter ratio on the unconfined compressive strength of cohesive soil specimens, Hakan Guneyli et al (2015)

Hakan Guneyli et al (2015) studied the influence of the length-to-diameter ration (L/D) on the unconfined compressive strength (UCS) of cohesive soil specimens. L/D ratio ranging from 0.5 to 3 was assessed to study the effect of specimen shape on the UCS values of four clay soils by testing compacted cylindrical specimens. They were tested under similar conditions and in accordance with the same technical standards. The work has demonstrated that changing the length of a cohesive soil specimen while keeping its diameter constant has a significant effect on the strength and failure pattern of the specimen.

Table 6. Index Properties and compaction parameters of the soils tested

Parameter	Soil name			
	Handere clay	Almanpınarı clay	Sam-Tekin clay	Kaolinite
LL (%)	43	45	59	41
PI (%)	22	22	33	21
Specific gravity, G_s	2.74	2.74	2.75	2.67
Fines content (%)	99	88	95	51
Sand size (%), 4.75–0.075 mm (%)	1	12	5	49
Silt size (%), 0.075–0.002 mm (%)	62	33	40	33
Clay size (%), <0.002 mm	37	55	55	16
Activity	0.5	0.42	0.61	0.93
Initial void ratio (e_0)	0.662	0.589	0.820	0.689
Classification				
USCS	CL-CH	CL-CH	CH	CL-CH
AASHTO	A 7-6	A 7-6	A 6	A 7-6
Group name	Inorganic clays of medium plasticity	Inorganic clays of medium plasticity	Inorganic clays of high plasticity	Inorganic clays of medium plasticity
Standard Proctor compaction				
MDD (kN/m^3)	16.2	16.8	16.8	17.2
OMC (%)	19.3	19.1	26	21.4

Properties of Materials

Four clay soils with different characteristics were selected for investigation in this study: kaolinite used in the ceramic industry; Handere clay which is predominantly montmorillonite; Almanpınarı clay and Sam-Tekin clay, which is predominantly illite. Index Properties and compaction parameters of the soils tested are presented in Table 6.

Sample Preparation

The clay samples for unconfined compression testing were prepared with the optimum moisture content (OMC) in order to eliminate the effects of factors such as the moisture content (w), void ratio (e), and natural density (γ_n) on the UCS. After compaction, a thin walled stainless steel sampling tube with an inner diameter of 48 mm, a length of 19 cm, and a wall thickness of 1.5 mm was driven into the soil in the standard Proctor mold with a hydraulic jack. The soil was immediately extruded from the sampling tube using another hydraulic jack and its ends were cut to make it the desired length.

Fig. 20. Photograph of the specimens used in the UCS tests (from left to right: Handere clay, Almanpınarı clay, kaolinite, and Sam-Tekin clay)

Cylindrical soil samples with 11 different length-to-diameter (L/D) ratios (from 0.5:1 to 3:1) were prepared for all four clay soils as shown in Figure 20. The lengths of the cylindrical specimens of each soil type ranged from 24 to 144 mm. In each case, the diameter was 48 mm.

Results and discussion

The trend of UCS with increasing L/D ratio observed for the four soil types are presented in Figure 21. The test results indicated that the UCS value decreases linearly with increasing L/D ratio. This decrease becomes particularly steep when the L/ D ratio exceeds 1, but then becomes less steep when the L/D ratio is between 1.25 and 2.5. In addition, the failure pattern generally changes from ductile to brittle with increasing L/D ratio in the range. Especially in the range $1.25 \leq$ D/L ≤ 2.5, brittle deformation predominates. At larger L/D ratios ≥ 2.75, the failure mechanism develops is quite complex and chaotic. The standard deviation of the unconfined compressive strength value for a soil specimen increases considerably with increasing L/D ratio. On the other hand, the increase in the standard deviation in the range $2.75 \leq$ D/L ≤ 3 is much greater than the increase seen when L/D < 2.75.

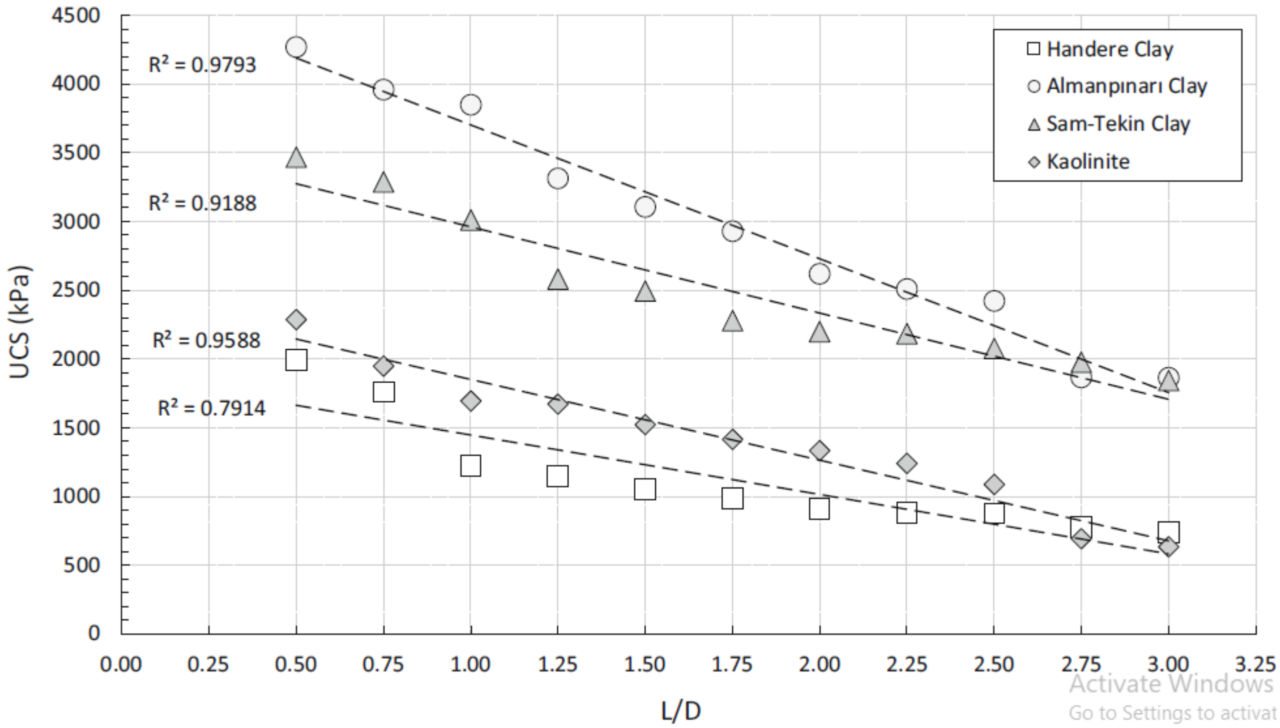


Fig. 21. Graph showing UCS values and L/D ratio for the four different soils

Based on the stress–strain curves and on observations of the specimens during and after the UCS tests, despite some exceptions, the specimens generally tended to reach their peak strengths at a smaller axial strain as the L/D ratio increased. In addition, the sharpness of the peak and the drop after the peak in the stress–strain curve increases greatly with increasing L/D ratio, implying brittle failure behavior.

The work has demonstrated that changing the length of a cohesive soil specimen while keeping its diameter constant has a significant effect on the strength and failure pattern of the specimen.

11.7 Study of the effects of specimen shape and remoulding on shear strength characteristic of fine alluvial sand in direct shear test, Omer Altaf et al (2016).

Omer Altaf et al (2016) studied the effects of specimen shape on the results obtained from circular and square shear boxes from direct shear test. Moreover remoulding effects on the shear strength characteristics of granular soils are also evaluated. For this purpose Ravi sand is tested in both circular and square shear box apparatuses. Similarly undisturbed and remoulded samples were tested in circular shear box. Remoulded (RM) samples were remoulded on in-situ density and moisture content. Results from two apparatus shows that there is no much significant difference in angle of internal friction for the specimen tested in a direct shear test. However the angle of internal friction values obtained from circular shear box apparatus are higher (0.8° to 3.7°) as compared to square shear box. Similarly, samples which were tested in circular shear box show more horizontal resistance values and higher shear stress values. Similarly, there are little differences in values of fine alluvial sand due to remolding. But the values of angle of internal friction in this research from undisturbed samples (UDS) are higher and more accurate then remolded specimens. This paper recommended that undisturbed specimens should be used for more accurate results where possible. But the results are more sensitive to other factors like density, moisture content and type of soil then remolding and shape of specimen in direct shear test.

Material Properties and Method

Index properties of soil taken from three different depths upto 1.5 m are shown in Table 7. The direct shear tests were performed taking the normal load value similar to that the sample is experiencing in field i.e., overburden load.

Table 7. Index Properties of soil selected

Sample taken from (m)	Specific gravity	Moisture Content (%)	Dry density (kg/m^3)	Relative density (%)
0.5	2.69	3.98	1067.25	0.83
1		4.31	1147.97	1.17
1.5		5.84	1142.68	1.19

Direct shear tests were performed on both apparatus having circular and square shear box as per ASTM D 3080. Difference between both apparatus is presented in Table 8. Samples were taken from three different depths (0.5 m, 1m and 1.5 m). UDS were extruded from tubes and RM samples were remolded on in-situ density and moisture content.

Table 8. Difference between circular and square shear box apparatus, ASTM D 3080

Property	Direct Shear Test	
	Circular shear box	Square shear box
Sample tested	UDS/RM	RM
Specimen Dimensions	63.7 dia., 20mm thickness	60mm x 50 mm, 20mm thickness
Area (cm^2)	31.22	36
Volume (cc)	62.44	72
Least count of dial guage	0.1 mm	0.0127 mm
Proving ring constant	2.002 N	3.649 N
Liver arm assembly	yes	No
Volume change	yes	No

Results and Discussion

Results obtained from direct shear test by two different apparatuses i.e. square shear box and circular shear box are compared in Table 9.

Table 9. Comparison of results of UDS and RM

	UDS		RM (circular)		RM (square)	
	ϕ	Max. displacement (m)	ϕ	Max. displacement (m)	ϕ	Max. displacement (m)
1	39.98	0.0034	37.1	0.0032	33.4	0.02667
2	38.1	0.0034	37.8	0.004	34.3	0.02667
3	38.01	0.004	36.4	0.004	35.6	0.0254

Figure 22 shows the failure envelopes for samples tested in circular and square shear box apparatus. Results show that Angle of internal friction obtained from circular shear box does not vary significantly with remolding of specimen as ϕ values for both undisturbed and remolded specimens are very similar. Although ϕ values for undisturbed samples are high then remolded, sensitivity for remolded samples is greater than 0.92. Results obtained from square shear box shows that sample resist less shear forces and show lower values of horizontal displacement. Failure envelopes show that the angle of internal friction values is quite similar for three remolded samples.

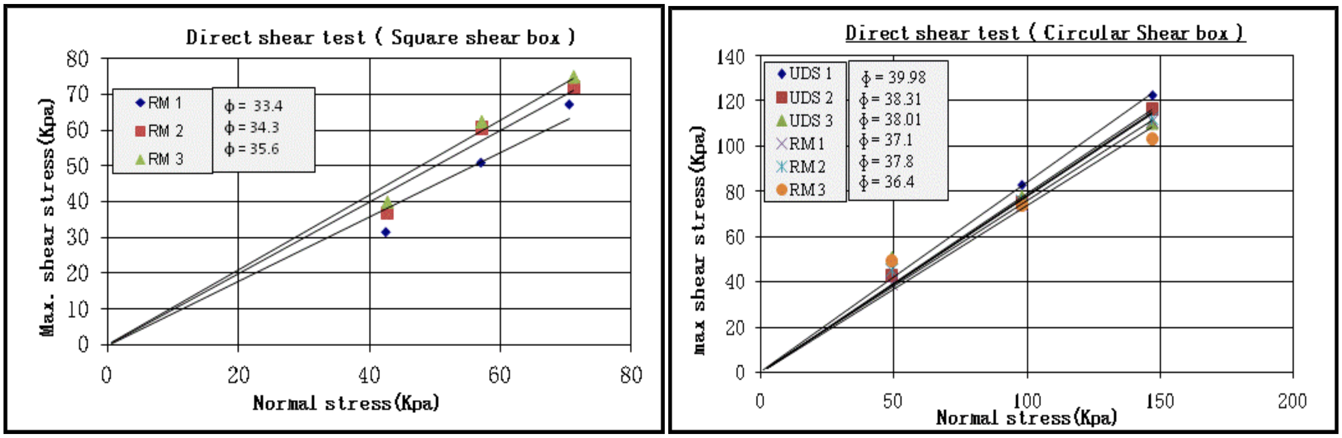


Fig. 22. Failure envelops for direct shear test (Circular specimens and Square specimens)

Conclusions

Results from two apparatus shows that there is no much significant difference in angle of internal friction for the specimen tested in direct shear test by two apparatuses i.e. square shear box and circular shear box. However the angle of internal friction values obtained from circular shear box apparatus are higher (0.8° to 3.7°) as compared to square shear box. Similarly samples which were tested in circular shear box show more horizontal resistance values and higher shear stress values. However, the values of angle of internal friction from undisturbed samples are higher and more accurate then remolded specimens. It can be concluded that although there are little difference in results from two shapes of specimen in direct shear test but any of these two apparatus can be used for the determination of shear strength with full confidence, keeping in view the other factors like specimen size, apparatus assembly, loading arrangements and calibration of apparatus etc.

11.8 Effects of specimen size and some other factors on the strength and deformation of granular soil in direct shear tests, Po-Kai Wu et al (2017)

Po-Kai Wu et al (2017) studied the effects of apparatus having different sizes with the specimen lengths ranging from 40 to 800 mm on strength and deformation of granular soil in Direct Shear (DS) Tests as shown in Table 10. The vertical and shear stresses acting on the shear zone were measured. Noticeable effects of specimen shape were observed. The effects of specimen size were evaluated by performing constant pressure DS tests on fine poorly graded sand in the small, semi-medium, medium and large DS apparatus. The residual shear strength of sand was independent of the specimen size and initial density. The peak strength decreased with an increase in the specimen size.

Table 10. Direct Shear apparatuses with four different specimen sizes used in the study

Direct Shear (DS) apparatus	Specimen size (mm)
Small	Disk-shaped: ϕ =60; & H=20 Rectangular-shaped: L=W=40 & H=20
Semi-medium	L=W=H=120
Medium	L=W=H=300
Large	L=800; W=500; & H=600

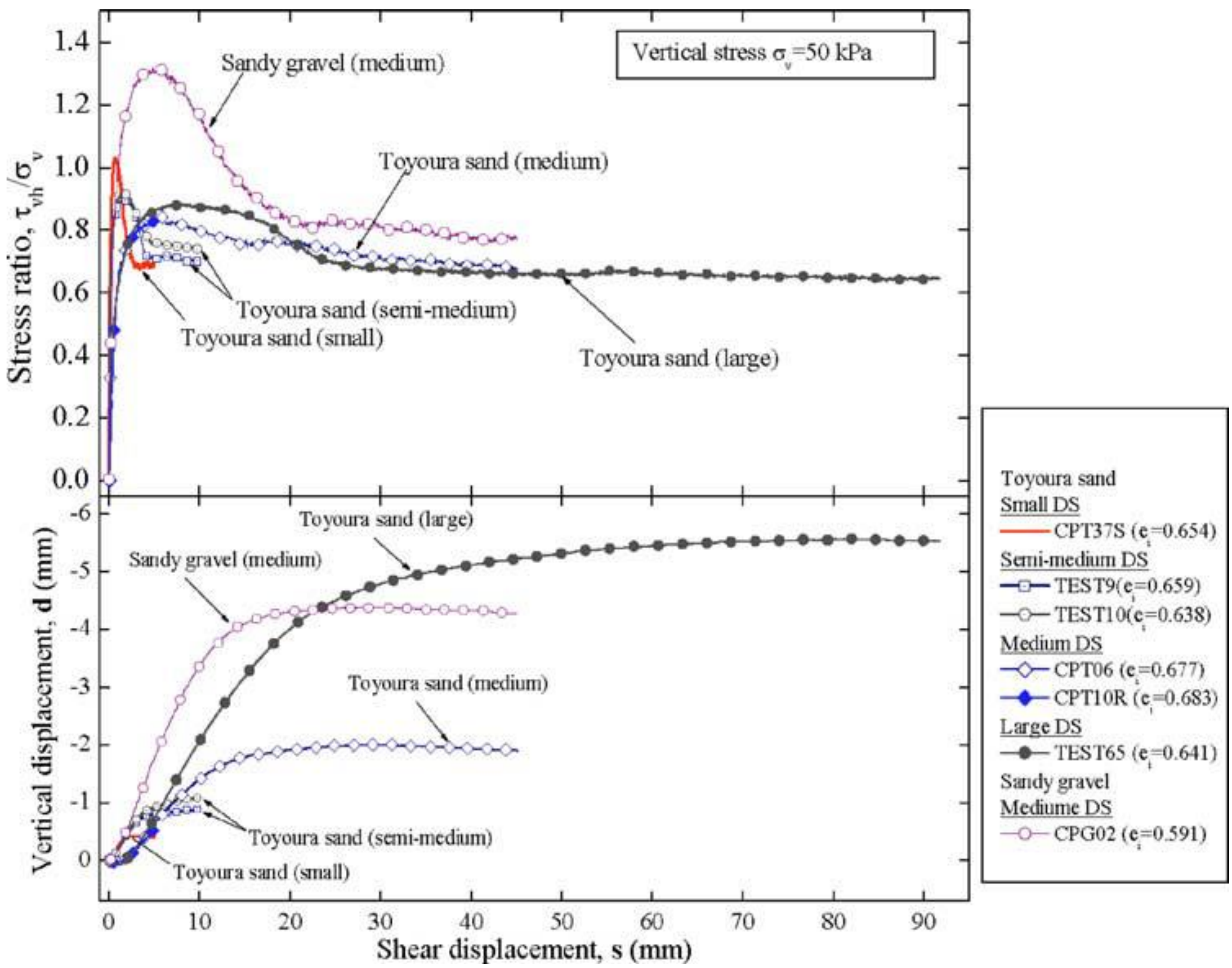


Fig. 23. Scale effect in DS test using different specimen sizes

The failure of dense sand was more progressive in disk-type specimens having a circular cross section than in those having a square or rectangular cross section. It is suggested to use the later shape to more accurately evaluate the direct shear behavior of granular material. Figure 23 presents the scale effect in DS test using different specimen sizes.

The peak strength by DS shear tests of dense sand decreases with an increase in the ratio of specimen length to the mean particle diameter i.e., L/D_{50} from 40 mm/ 0.17 mm = 235 (i.e., conventional small DS tests) toward 800 mm/ 0.17 mm = 4700 (special very large DS tests). The residual strength of sand was essentially independent of the specimen size effects in terms of L/D_{50} as well as the initial void ratio.

11.9 Evaluation of Soil Shear Strength Parameters via. Triaxial Testing by Height versus Diameter Ratio of Sample, Jonas et al (2009)

Jonas et al et al (2009) studied the evaluation of soil shear strength parameters via triaxial testing by height versus diameter ratio of sand sample in loose and dense state. To reduce an inaccuracy related to the non-uniform stress-strain distribution, author conducted test by reducing the sample height/diameter ratio from 2 to 1, eliminating friction between the sample ends and plates. Having not eliminated the above mentioned influence, factors during the testing procedure the angle of internal friction ϕ and the cohesion c for the sample of $\phi \neq 0$ are determined larger than the actual ones. Author proposed height/diameter H/D = 1 for determining the angle of internal friction ϕ and the cohesion c, when testing the soil sample.

The non-uniformity in sample deformation can be caused due: the actual sample ends conditions, those restraining the free displacements in horizontal directions; the sample height; the insufficient drainage in sample; the sample rubber membrane, the specimen self-weight factor etc. The finite element method (FEM) analysis also shows the non-uniform distribution of stress and strain in the sample when modelling the triaxial testing. Author has used reference of Airey 1991; Hinokio, Nakai 2005; Jeremic et al. 2004.

Author has used reference of Head, 1986 in which eliminating of the friction has an insignificant effect when the standard height is employed (Head 1986). Hettler and Gudehus (1985) carried out the standard triaxial tests for samples of H/D ratio H/D = 21.1 cm/10 cm using the non-guided cap and the non-lubricated ends. They determined the ϕ to be less by 5˚ versus the sample of H/D = 28/78 cm.

It is seen from the experimental work conducted by the author, the deviator stress increases when L/D ratio is changed from 2 to 1 as shown in Figure 24.

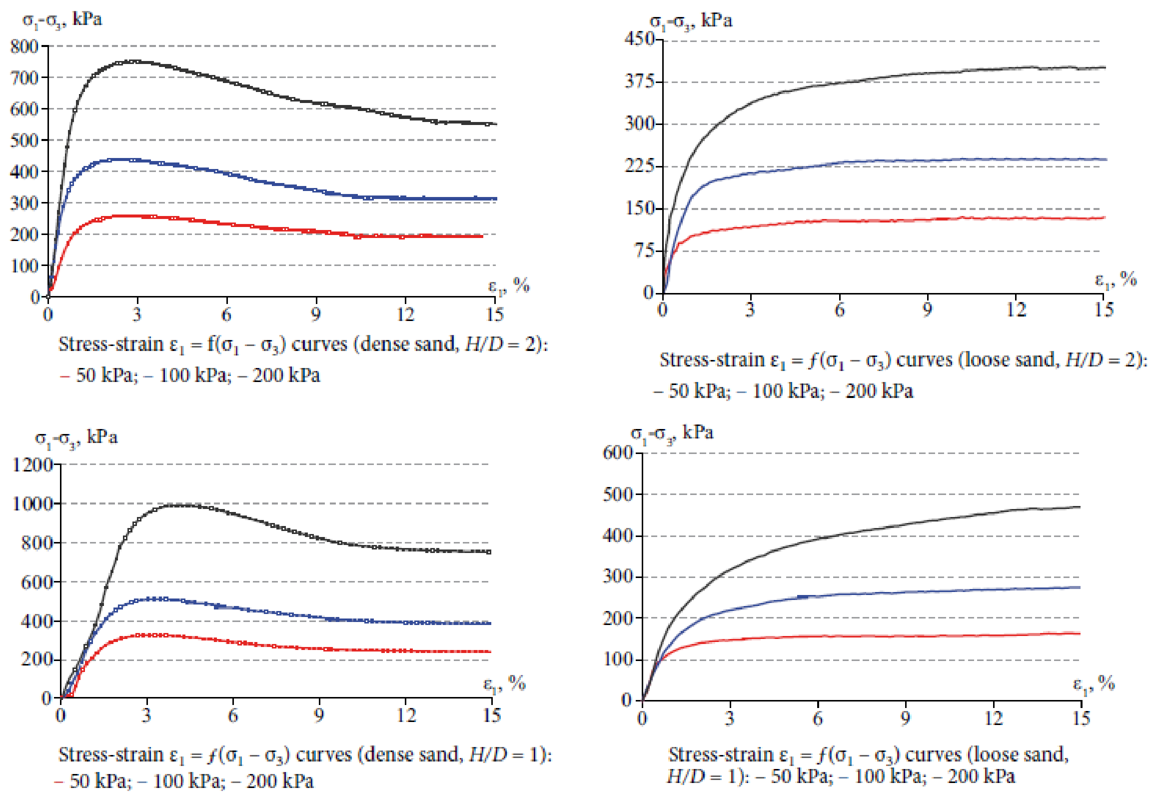


Fig. 24. Stress-strain curves for H/D = 1 and H/D = 2

11.10 Change of soil mechanical properties due to triaxial sample size, Sarunas Skuodis et al (2019)

Sarunas Skuodis et al (2019) studied the effect of specimen size and scale effect on the soil shear strength. Several consolidated drained triaxial compression tests were performed on two specimen sizes of sand and clay. The result indicates that the effect of triaxial clay sample size is more significant with higher confining pressures. Test on sand samples shows that sample size does not influence the results of the shear strength.

Experimental work on Triaxial

The consolidated drained (CD) tests were performed for the specimens of height/diameter ratio of H/D = 2. For investigation of scale effect the triaxial compression tests were performed on two specimen sizes of sand and clay: first, samples of 50 mm diameter and 100 mm height (50/100); second, samples of 100 mm diameter and 200 mm height (100/200). The saturation wasn't performed. The clay samples have been sheared under three horizontal pressure σ_3 = 150 kPa, 250 kPa, 350 kPa. The sand samples have been cut under σ_3 = 100 kPa, 200 kPa, 300 kPa. An axial strain rate of 0.02 %/min was used for clay and 0.95 %/min for sand. Each of the tests was continued up to deformation, which is equal to 15%. The circles representing the state of stress in the traxial samples at failure are shown in Figure 25-26.

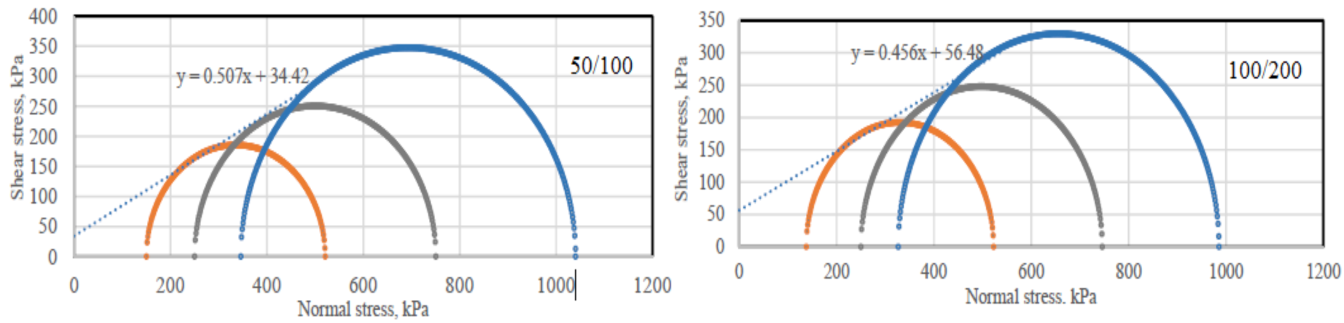


Fig. 25. Mohr circles and strength envelope of clay

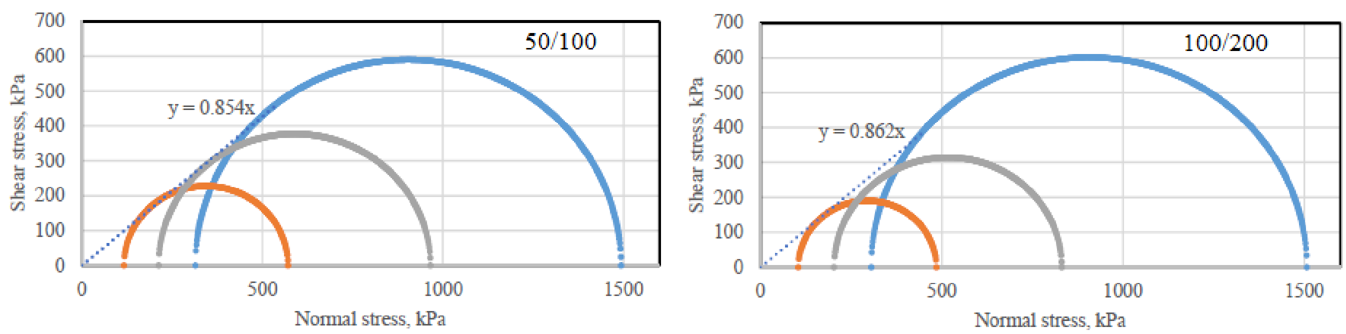


Fig. 26. Mohr circles and strength envelope of sand

The first type of tests carried out on over-consolidated moraine clay demonstrates that the angle of peak internal friction in term of effective stress shows variation with sample size. It was determined ϕ'=26.87º for samples of 50 in diameter and ϕ'=24.50º for samples of 100 in diameter (Table 11). It was found that the effective peak friction angle decreases and max deviator stress decreases as the sample size increases. Whereas effective cohesion shows rise from 34.42 kPa to 56.48 kPa with sample size increment. The second type of tests carried out on sand samples shows that sample size effect doesn't influence the results of drained triaxial compression tests (see Table 11). The cohesion is c' = 0 (kPa) for both type of samples. The comparison of test data on samples 50 and 100 mm in diameter, for which the slenderness is the same, does not reveal any difference in the peak angle of internal friction.

Table 11. The peak values of shear strength from tiaxial tests

Type of soil	Cohesion c', kPa		Internal friction angle, φ' °	
	50/100	100/200	50/100	100/200
clay	34.42	56.48	26.87	24.50
sand	0	0	40.51	40.77

Author's show that clay analyses can be significantly affected by the choice of the specimen size used to determine shear strength parameters.

12.0 CONCLUSION

Many laboratories perform triaxial tests on sand or clay to determine the shearing strength of the soil sample. However, there are different size of triaxial cells in use today and the behavior of particular soil from different studies are often compared without due attention to the differences in specimen size and its effects on soil shear behavior. The main reason which can be attributed may be problem with achieving saturation, big particles appearance in shearing plane and long term investigations and etc. In this report, the influence of specimen size on shear strength parameter is introduced. Looking at the various codes & standards having varying specimen size from 30 mm to 110 mm diameter and understanding the size effect from various literature & published papers, it is suggested that this concepts is important in evaluating soil behavior and assigning appropriate shear strength parameters. Triaxial tests are often used to determine the behavior and strength characteristics of soils subject to wide range of stress paths and loading condition. Reliability of shear strength parameters for engineering applications and analysis depends upon the true condition of sample, better representation of specimen size and resemblance in testing procedure. A small diameter sample may not truly represent the structure of the soils at the site. Testing bigger samples sizes gives better representation of field shear and deformation behavior. The reason for different results for bigger and smaller is mainly due to the presence of non-uniformity and non-homogeneity which can be somewhat captured in bigger size sample but limited in a small size sample. The lower deviator stresses in the bigger specimens could be associated with the more intense strain softening resulting from shearing which may be due to smaller compressibility behavior as compared to smaller specimen

It is concluded from the study that shear strength parameters obtained from smaller specimen sizes has the potential to affect geotechnical engineering applications in which soil shear strength and friction angle play important roles. Testing bigger specimen sizes may provides better representation of stress-strain behavior.

13.0 FUTURE SCOPE FOR THE STUDY

In geotechnical engineering, the shear strength parameters are crucial and useful for design work to produce safe and economic geotechnical structure design. Geotechnical analysis can be affected by variations in strength parameters of the same soil determined from different specimen sizes. While using small size samples for determining shear strength parameters might result in un-conservative design, the choice of a large sample size may provide a more accurate representation of soil strength conditions and field deformations. The triaxial test is a most widely used laboratory method for determining the soil shear strength. The test is also acknowledged to be the most reliable method employed for simulating a stress-strain state of ground. It is assumed that a soil sample deforms uniformly during triaxial testing. But one often faces a case when the sample in the triaxial apparatus deforms on the contrary. The non-uniformity can be caused by the end restraining effect; the sample height influence factor, sample self-weight factor, and the insufficient drainage, etc. Using soil strength parameters to determine bearing resistance usually are defined with some errors. If we know the reasons of errors, short comings in specimen size, sample type, apparatus, testing procedure could be eliminated; method of data evaluation on specimen size effect can be developed.

Shear strength parameters (c & ϕ) and deformation obtained can be studied for its relation between different sizes of the specimen. Size effect can be studied in analyzing stress-strain curve with respect to increasing confining pressure and its significance at higher confining pressures. Effect of specimen size on pre-peak, peak and post peak strength behavior with respect to strain can be researched. Axial strain corresponding to peak stress behavior can be studied in the small and bigger size sample for any major discrepancies. Smaller specimen may develop migration of pore water pressure quickly as compared to bigger specimen. Pore water pressure under shearing may differs by specimen size due to which large difference in the effective stress paths can be observed. To reduce specimen size effect on shear strength parameters, a correction factor related to size effect can be proposed with extensive testing program. This will help to assign appropriate shear strength parameters for better representation of field soil behavior. This may also reduce the error that could arise due to calibration of constitutive model based on smaller specimen (i.e., 38 mm dia.) for modeling larger soil masses using numerical analysis for field application.

It is hoped that this information will be useful to potential users who are looking for new sources. The intension of this literature search is to uncover the technical information and make them available for the potential users. It will help to make proper choice for specific problems/projects and it will stimulate the further development in this field.

14.0 DISCLAIMER

This Research Review Report is submitted with an objective to enhance the understanding and in no way reduces the importance of tests carried out by standard procedures given in the established codes.

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ACKNOWLEDGEMENT

I express a deep sense of gratitude towards CSMRS for providing opportunity to learn Tri-axial Technology during laboratory testing. I acknowledge the contribution of the authors of various research papers and standard codes for bringing out their meaningful content which has helped to gain knowledge in preparing this technical report.

I would like to acknowledge and give my warmest thanks to all the officers of CSMRS who directly or indirectly offered their support.

I am very much thankful to Dr. Chandresh H. Solanki, SVNIT, Surat and Dr. G.V.Ramana, NIT Warangal for sparing their valuable time and reviewing this Research Review Report. Their suggestions are greatly appreciated.

Many have generously contributed their time, ideas and support during the course of work. Thank you to each of you.

Maurya Suresh Seopal, Scientist 'D'