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*Research Article*

# Effects of low clay content on the anisotropic behavior of sand-clay mixtures: laboratory investigation using TSHCA

**Vahid Mohammadi** [1], **Hadi Bahadori** [2*]

1 Department of Civil Engineering, Urmia University, Urmia (Iran), email: va.mohammadi@urmia.ac.ir
2 Department of Civil Engineering, Urmia University, Urmia (Iran), email: h.bahadori@urmia.ac.ir
*Correspondence: h.bahadori@urmia.ac.ir (H. Bahadori)

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**Abstract:** Undrained behavior of sandy soil with fines content is a challenge in geotechnical research. In this article, the effect of low clay content (plastic Kaolin) on the anisotropic behavior of sand is studied. In the technical literature, there are different data about the effect of fine particles (generally high percentage), but there are not enough studies on low fines content (especially plastic fines) and anisotropic conditions. For this purpose, 30 undrained tests are performed using a torsional shear hollow cylindrical apparatus (TSHCA) with constant (α°) and (b) values on Firoozkuh sand. The specimens had Kaolin contents of 0, 3, 5, 7 and 10%, and the inclination angle (α°) is varied from 15° to 60°. The specimens are prepared by dry deposition method and are consolidated under P'c= 100 and 200 kPa. The results of the experiments show that increasing the (α°) leads to more contractive behavior in sand. By adding clay particles to the host sand up to 3%, the peak strength of the specimen is increased (7% and 6% for $\alpha$=15°and 30°, respectively), and then with the increase of clay content up to 10%, the strength of the specimen is decreased (33% and 22% for $\alpha$=15°and 30°, respectively). But at α = 60°, with the addition of 5% clay, decrease in the peak strength is observed (about 15%) and with a further increase in the clay content, unlike the angles of 15° and 30°, increase in the peak strength of the specimen is observed, so that at 10% clay, the strength of the specimen is higher than the host sand (about 7%), which can be attributed to the cohesion nature of the clay particles. With the increase of clay content, anisotropy degree is decreased. In other words, with the increase of fines content, the anisotropic behavior is decreased.

**Keywords:** Plastic Kaolin, anisotropic behavior, torsional shear hollow cylindrical apparatus (TSHCA), firoozkuh sand, anisotropy degree.

## 1. Introduction

Anisotropy is the change in mechanic properties of a material (such as strength and stiffness), as the direction of loading rotates around a fixed direction. Casagrande (1944), who pointed out the importance of anisotropy and, defined different categories of soil anisotropy. His definition of anisotropy was split into two parts, i.e., the inherent and the induced anisotropy. Inherent anisotropy is defined as a physical property inherent in the material and completely independent of the applied strains and stresses. Particle shape and the grain size distribution are two important factors influencing inherent anisotropy of a material (Oda, 1972). However, Shibuya and Hight (1987) and Yang et al. (2016) observed that even spherical grains may develop anisotropy, indicating thus that the deposition of the grains in a gravitational field is also very important in the inherent anisotropy of a material. Most engineering structures cause the magnitude of the principal stresses to change and their axes to rotate in direction, and a soil is more likely to subject an anisotropic stress state (Ishihara & Towhata, 1983). Therefore, soil's response to loading will reflect its inherent anisotropic structure and, consequently, depend on the orientation and continuous rotation of the principal stresses (α°) and the intermediate principal stress parameter ($b = \sigma_2 - \sigma_3/\sigma_1 - \sigma_3$). These two parameters (α° and b) are defined as main parameters for studying the anisotropy of sands and, soil anisotropy is one of the most important parameters that influences soil behavior (Guo & Stolle, 2005; Radjai & Azéma, 2009; Wrzesiński & Lechowicz, 2015).

Considering that most natural sands have an anisotropic behavior (Saada, 1988). The torsional shear hollow cylindrical apparatus is one of the most suitable methods to investigate soil anisotropy (Liu et al., 2022). Many experiments have been performed by the TSHCA to show the effect of anisotropy on the behavior of sand (Bahadori et al., 2008; Lade et al., 2008; Prasanna et al., 2020; Triantafyllos, 2020; Mohammadi & Bahadori, 2024). The effect of inherent anisotropy has been studied by Yoshimine et al. (1998) in Toyoura sand. When the inclination angle (α°) becomes larger with respect to the deposition direction, the behavior clearly becomes strain-softening and shows more contractive behavior. Similar results have been reported by some researchers (Cai et al., 2013; Kumruzzaman & Yin, 2010; Sivathayalan & Vaid, 2002; Triantafyllos, 2020; Uthayakumar & Vaid, 1998). This significant dependence of the behavior of sand on the principal stress direction, indicates the inherent anisotropy in the sand. Therefore, it is obvious that inherent anisotropy has a significant effect on sand behavior, but natural sand deposits often contain varying amounts of fines (like clay) (Zeng et al., 2023), and the effect of fine grains on the behavior of sand is the subject of discussion (Bensoula et al., 2022; Bensoula et al., 2018; Chu & Leong, 2002; Guo & Cui, 2020; Krim et al., 2019; Peng et al., 2018).

The effect of clay particles on the undrained behavior of sand-clay mixtures depends on several factors such as plasticity Index, clay content, clay mineralization and sand grading. Clay content can be stated as the most important factor affecting the behavior of sandy soils. Koester (1994) by conducting a series of undrained triaxial tests on sand-clay mixture (Hoston sand and Kaolin), observed that by adding clay content to the host sand up to 15%, the strength of the specimen is decreased and then is increased again. Ghahremani and Ghalandarzadeh (2006) observed that by adding clay content (Kaolin) to sand up to 30%, the strength of the specimen is decreased and then is increased with clay content greater than 30%. Bouferra and Shahrour (2004) observed a decrease in the strength of specimen with the addition of 15% plastic clay content (Kaolin) to the sand-clay mixtures. Talamkhani and Naeini (2021) conducted a series of triaxial tests with Kaolin up to 30% and reinforced with geotextile and based on the results obtained, the addition of clay content changes the behavior of the specimens from dilative to contractive and the undrained shear strength is decreased. Georgiannou et al. (1990) studied the effect of clay (Kaolin) content on the undrained behavior of the sand-clay mixtures. They observed that with increasing clay content, the shear strength of the sand-clay matrix is decreased (with quasi-steady state behavior). Thus, the brittleness index is increased with increasing fines content.

Although several researchers studied the effect of fines content on sand behavior, they are generally concentrated in the range above 10%, and have been limited to using triaxial apparatus. By evaluating field studies conducted after large earthquakes and case histories of actual soil behavior, it is observed that many soils have a low percentage of fine grains (<10%). Ohsaki (1970) observed that during the Niigata earthquake (1964), soils with fines content less than 10% are more likely to liquefy. Many liquefied sites in central and western of Taiwan during the 1998 Chi earthquake contained silt and clay grains. Tokimatsu and Yoshimi (1983) reported that 50% of liquefied soils in 17 worldwide earthquakes had fines content less than 5%. Therefore, it is necessary to study the effect of low fine content on the behaviour of sand, especially in anisotropic conditions.

However, there is limited studies on the effect of fines grain on the anisotropic behavior of sand, and these studies have also focused on high fine contents and non-plastic silt. (Bahadori et al., 2008) conducted a comprehensive study using a hollow cylindrical apparatus on Firoozkuh silica sand and different percentages of silt (15, 30 and 70%). The value of α° varies between 15° and 75°. Their results showed that adding higher contents of silt to the host sand causes a dramatic strain-softening response in the behavior and decreases the effect of anisotropy. (Khayat et al., 2014) has reported similar results on Hamadan and Tehran sand. Both studies are focused on high silt contents and the fine percent variation is also very high.

According to the mentioned cases, the research process and the goals of this article can be stated as follows:

- Investigating the effect of kaolin clay on the undrained behavior of the host sand (Firoozkuh sand) with emphasis on the low percentage of fine grains in the anisotropic condition.
- Using five percent of fine grain (0, 3, 5, 7 and 10 %) and conducting tests using a torsional shear hollow cylindrical apparatus.
- The inclination angle (α°) values are changed from 15° to 60°.
- The results are displayed in form of stress-strain and stress path curves.
- Interpretation of results by investigation of behavioral mechanisms and dimensionless parameters.

## 2. Materials and methods

### *2.1. Materials description*

The sand used in this study is poorly graded Firoozkuh sand with uniform gradation sand (SP), called F161, with a golden yellow color and medium angular grains (Figure 1). This sand has many similarities with well-known standard sands such as Toyora. Tsuchida (1970) proposed the boundaries for most liquefiable soil. Figure 2 illustrates that the grain size distribution curve of the F161 sand is located inside the limits for the most liquefiable soil, which indicates the high liquefaction susceptibility of the used sand. The particle morphology from scanning electron microscope (SEM) of F161 is provided in Figure 1, which shows that this sand contains crushed and rounded particles. The fine material used in this study is Kaolin clay with low plasticity (PI=21). Physical properties of soils are summarized in Table 1 and the chemical analysis (XRF) of tested Kaolin clay is presented in Table2. Based on the gradation curve (Figure 2), it can be seen that with increasing clay content in sandy soil, the mean particle size ($D_{50}$) is decreased and, the particle size distribution curve is covered a larger range.


**Figure 1.** Firoozkuh sand (F161) used in this study and scanning electron microscopic (SEM) image of this sand.

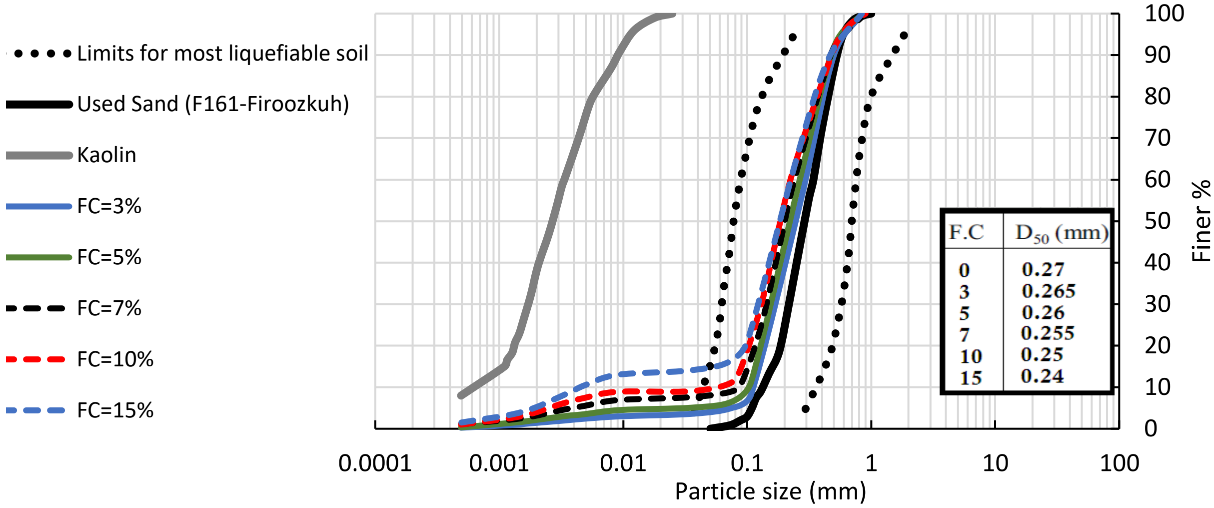



**Figure 2.** Grain size distribution curves of tested materials.

**Table 1.** Physical properties of tested materials.

| Material | Gs | *emin | *emax | Cu | Cc | $D_{10}$ (mm) | $D_{50}$ (mm) | LL (%) | PL (%) | PI (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| F161 sand | 2.65 | 0.548 | 0.874 | 1.87 | 0.88 | 0.16 | 0.27 | - | - | - |
| Clay (Kaolin) | 2.62 | - | - | - | - | | - | 49 | 28 | 21 |

*$e_{min}$= minimum void ratio, $e_{max}$= maximum void ratio; ASTM-D4253 (ASTM, 2006a) and ASTM-D4254 (ASTM, 2006b) were used to measure these parameters

**Table 2.** Chemical analysis (XRF) of tested kaolin clay.

| $SiO_2$ | $Al_2O_3$ | $Fe_2O_3$ | CaO | MgO | $K_2O$ | $Na_2O$ | TiO2 | $SO_3$ | LOI |
|---|---|---|---|---|---|---|---|---|---|
| 58.3 | 29.57 | 0.87 | 0.53 | 0.31 | 0.23 | 0.1 | 0.4 | 0.08 | 9.61 |

### *2.2. Testing apparatus (torsional shear hollow cylindrical apparatus)*

Anisotropic three-dimensional stress state and unequal principal stresses are induced in the soil in most geotechnical structures. This complex state includes the rotation of the principal stress direction (α) and different values of the intermediate principal stress ratio (b). It is not possible to control the principal stress directions in the normal triaxial shearing apparatus, but the torsional shear hollow cylindrical apparatus provides the possibility of simultaneous application of axial load, torque, internal and external pressures. Therefore, both parameters α and b are under control in the stress path approach, which provides the possibility of investigating the inherent anisotropic behavior of the soil and its effects on the stress-strain behavior (post-peak) and the stress path. Figure 3 illustrates the TSHCA device of Urmia University used in this study and its schematic form. In this apparatus, to attain the post-peak behavior, torsional strain control is programmed using a direct current (DC) motor. Performed torsional force speed of 0.5°/min is applied in the tests. In studying the effect of inherent anisotropy, α and b are kept constant during torsional shear. In order to control α and b to reach the desired stress paths, the general equations of hollow cylindrical apparatus, which are defined in the device by Bahadori et al. (2008). Geometric characteristics and stress conditions, are illustrated in Figure 4.

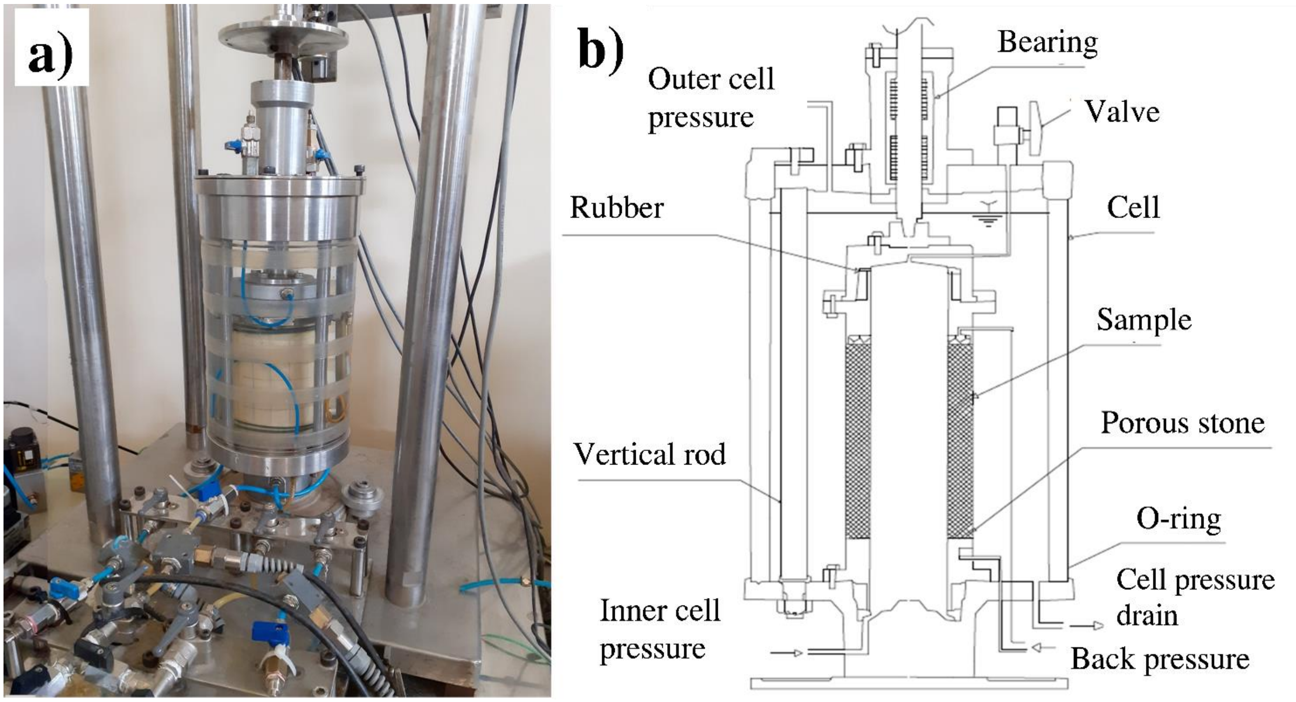



**Figure 3.** Torsional shear hollow cylindrical apparatus, (a) TSHCA of Urmia University with a specimen in the test, (b) Schematic view of TSHCA.

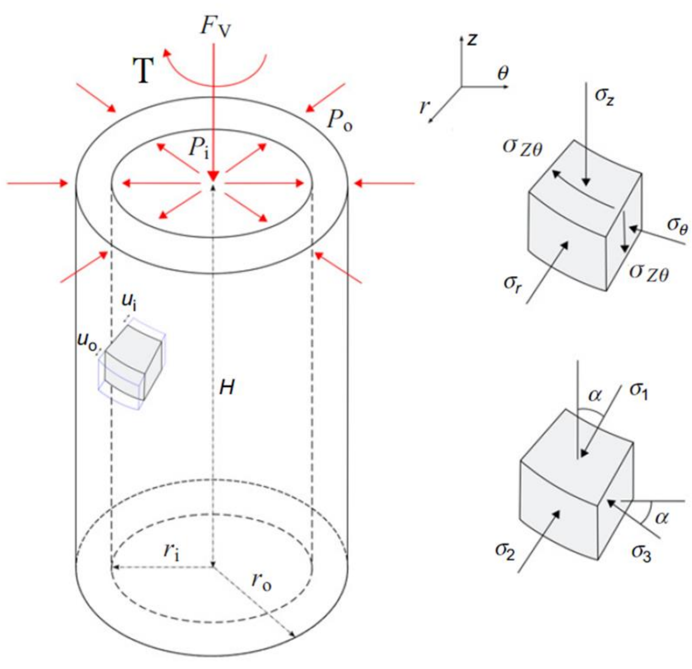



**Figure 4.** Geometric characteristics and stress conditions in TSHCA.

*2.3. Sample preparation and test procedure*

There are different sample preparation methods for granular soils at a laboratory scale. In this study, the specimens were prepared in the laboratory with the dry funnel deposition method. This method prepares more uniform silty sand specimens (Bahadori et al., 2008; Khayat et al., 2014). Many researchers considered dry funnel deposition method as the most suitable method for evaluating the strength of sands (Cherif Taiba et al., 2018). The mixture of sand-clay (with constant relative density $D_r$=20%) is deposited with the help of a long funnel (with an opening of about 12 mm) which is placed on the bottom of the mold with a falling height of almost zero. After preparation, an isotropic stress of 50 kPa was applied and $CO_2$ (carbon dioxide, using a pressure of approximately 3kPa) and de-aired water were passed through the specimen. The process of saturating the specimen was done by increasing the confining pressure in several steps and measuring the pore water pressure. Skempton's parameter (B) was recorded and found to be in excess of 0.97 (by applying a back pressure of at least 210 kPa) and therefore, the samples can be considered as fully saturated. The consolidation of the specimens was performed isotropic with P'c= 100 and 200 kPa to avoid the induced anisotropy effect. The shear stage (with the rate of torque-speed 0.5º/min) started after the consolidation stage. The void ratio of the specimens was measured at the end of the test. The dimensions of the sample of the TSHCA are 60 mm inner diameter, 100 mm outer diameter and 120 mm in height. The sample is contained in two latex membranes and surrounded by water both internally and externally. Figure 5 illustrates the specimen preparation steps.

30 undrained shear tests (CU) were performed, in which the values of parameter b and inclination angle (α) were kept constant during the entire shearing process of the soil specimens, fines content (FC), initial mean effective stress (P'c), inclination angle, void ratio and relative density were our variables. All test results are displayed in the form of stress-strain and stress path curves, and the comparison of the results are performed based on these curves using the classification of the undrained behavior of sand. Based on this classification, the undrained behavior of sands can be divided into three general groups (Figure 6).

1. Strain-Hardening (non-flow)
2. Strain-Softening & Hardening (limited flow)
3. Strain-Softening (flow)


Fixing two thin latex membranes on the cell pedestal


Assembly of the mold around the cell pedestal


Deposition of sample into the mold by funnel


Disassembly of the split mold


Assembly of the cell chamber and top cap


Filling the inner and outer chambers simultaneously with de-aired water

**Figure 5.** (a-f) specimen preparation steps.

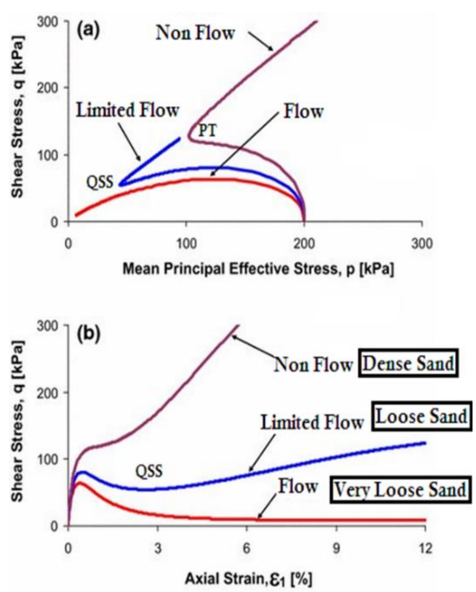



**Figure 6.** Schematic view of undrained behaviors of sand: (a) stress path, (b) stress-strain.

## 3. Experimental results and analysis

To study the effect of low clay content on the anisotropic behavior of sand–clay mixtures, a series of undrained TSHCA tests were performed on the Firoozkuh sand. Remolded sand specimens were sheared at fixed α° while b was set to 0.5. Figure 7 illustrates the loading conditions without restricted zones (No-go), where significant stress non-uniformity occurs (Symes, 1983). Experiments were carried out on specimens with confining stress of 100 and 200 kPa and clay content of 0 to 10%. Also, the inclination angle varied from 15° to 60°. Characteristics of the performed tests are provided in Table 3. The effect of plastic Kaolin and inclination angle on the undrained behavior of sand–clay mixtures will be discussed, as followed.

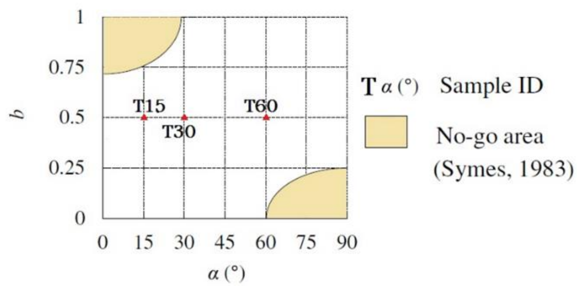



**Figure 7.** Schematic indicating loading conditions during undrained TSHCA tests. The no-go area was defined by Symes (1983).

**Table 3.** Summary of TSHCA tests performed during the present study.

| No. | FC (%) | P'c (kPa) | Inclination angle (α°) | Void ratio after consolidation, $e_f$ | Loading type |
|---|---|---|---|---|---|
| T1 | 0 | 100 | 15 | 0.738 | Compressional |
| T2 | 0 | 100 | 30 | 0.739 | Compressional torsional |
| T3 | 0 | 100 | 60 | 0.737 | Extensional torsional |
| T4 | 0 | 200 | 15 | 0.727 | Compressional |
| T5 | 0 | 200 | 30 | 0.728 | Compressional torsional |
| T6 | 0 | 200 | 60 | 0.729 | Extensional torsional |
| T7 | 3 | 100 | 15 | 0.734 | Compressional |
| T8 | 3 | 100 | 30 | 0.733 | Compressional torsional |
| T9 | 3 | 100 | 60 | 0.732 | Extensional torsional |
| T10 | 3 | 200 | 15 | 0.724 | Compressional |
| T11 | 3 | 200 | 30 | 0.726 | Compressional torsional |
| T12 | 3 | 200 | 60 | 0.722 | Extensional torsional |
| T13 | 5 | 100 | 15 | 0.732 | Compressional |
| T14 | 5 | 100 | 30 | 0.73 | Compressional torsional |
| T15 | 5 | 100 | 60 | 0.729 | Extensional torsional |
| T16 | 5 | 200 | 15 | 0.722 | Compressional |
| T17 | 5 | 200 | 30 | 0.719 | Compressional torsional |
| T18 | 5 | 200 | 60 | 0.72 | Extensional torsional |
| T19 | 7 | 100 | 15 | 0.722 | Compressional |
| T20 | 7 | 100 | 30 | 0.718 | Compressional torsional |
| T21 | 7 | 100 | 60 | 0.717 | Extensional torsional |
| T22 | 7 | 200 | 15 | 0.71 | Compressional |
| T23 | 7 | 200 | 30 | 0.713 | Compressional torsional |
| T24 | 7 | 200 | 60 | 0.711 | Extensional torsional |
| T25 | 10 | 100 | 15 | 0.691 | Compressional |
| T26 | 10 | 100 | 30 | 0.69 | Compressional torsional |
| T27 | 10 | 100 | 60 | 0.689 | Extensional torsional |
| T28 | 10 | 200 | 15 | 0.678 | Compressional |
| T29 | 10 | 200 | 30 | 0.681 | Compressional torsional |
| T30 | 10 | 200 | 60 | 0.68 | Extensional torsional |

*3.1. Effect of inclination angle (α°) on the undrained stress-strain behavior of sand*

The main way to investigate anisotropic strength of soil is to determine how α° affects the soil behavior. In other words, by shearing soil specimens at different values of α° while holding b and P'c constant (Nishimura et al., 2011). Figure 8 illustrates the results of two series of tests on clean sand under 100 and 200 kPa confining pressure. The sand specimen exhibits strain-softening and hardening behavior, in which is characterized by the initial maximum shear strength at a small strain, then the shear strength decreases to the minimum value at the medium strain. The minimum shear strength is called the quasi-steady state point (QSS) and is defined as the point when the undrained behavior is changed from contraction to dilation. After reaching the minimum shear strength, it increases to its maximum value at large strain. This behavior of sand is called flow type with limited deformation or limited flow (Ishihara, 1993). The specimen at $\alpha$=15° showed the highest undrained peak shear strength ($q_{peak}$=54 kPa) compared with $\alpha$=30° ($q_{peak}$=42 kPa) and $\alpha$=60° ($q_{peak}$=27.5 kPa) when subjected to P'c=100 kPa for the same relative density. Therefore, by increasing inclination angle, a decrease in the peak shear strength (22% and 49% for the 30° and 60°, respectively) was observed compared to the 15°. So, as the inclination angle (α°) became higher, the behavior became softer and more contractive. Such strain-softening has been attributed to the inherent anisotropy in the sand fabric during sedimentation (Arthur & Menzies, 1972). The minimum strength occurs when α = 60°.

One of the basic parameters that has a significant effect on the mechanical behaviour of soils is the initial confining stress (P'c). The peak strength is increased as the consolidation stress is increased from 100 kPa to 200 kPa. Axial strain at peak strength is also increased by increasing P'c. For example, for $\alpha$ =15°, the peak strength reaches a value of 54 kPa at P'c = 100 kPa and 0.34% strain, whereas at P'c=200 kPa and 0.37% strain, the peak strength is increased to 92 kPa.

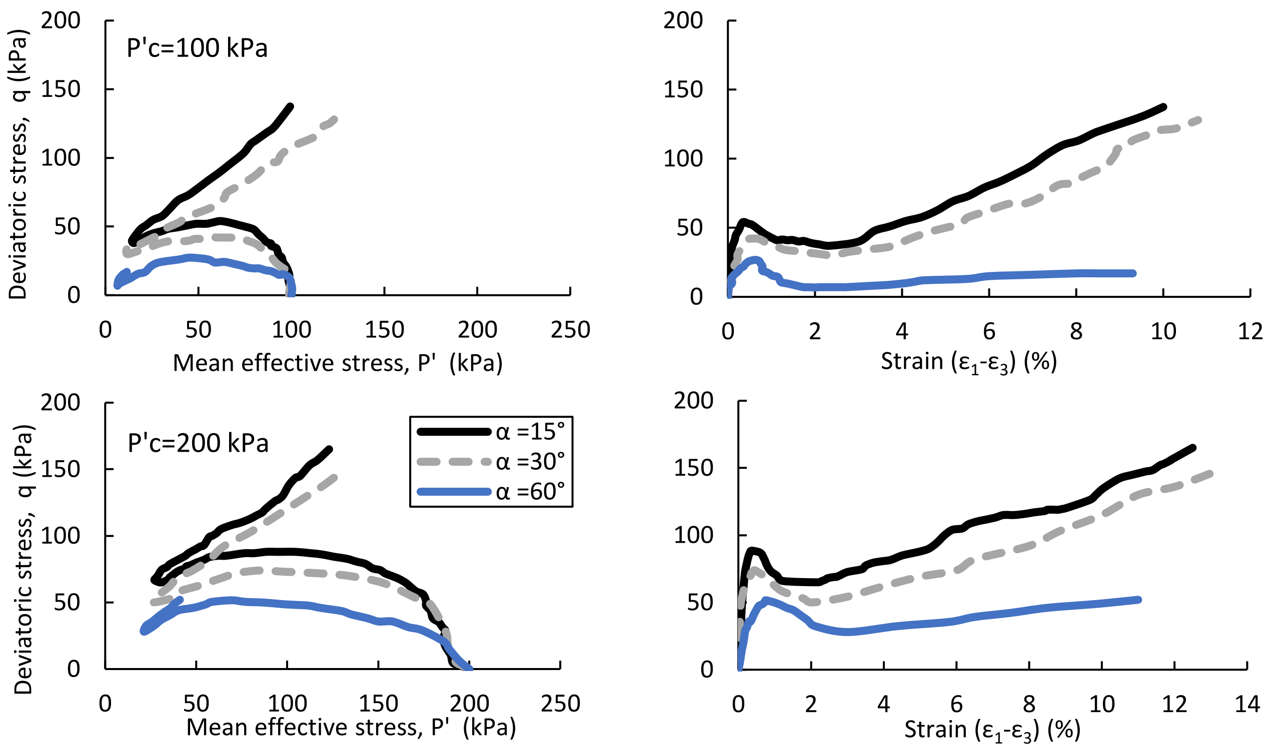



**Figure 8.** Effect of inclination angle (α°) on the behavior of clean sand (0% clay).

The entanglement between sand grains along the long axis parallel to the direction of the shear plane is the worst case. In other words, sand particles have the greatest tendency to slip on each other when the major principal stress is parallel to the direction of the longitudinal axis of the sand particles (Li & Yu, 2009; Xiong et al., 2016). Figure 9 illustrates (schematically) how the load is transferred and the performance of the sand grains with the rotation of the main stress angle. Assuming the width of the grain (w) and the long dimension of the grain (l) and hypothetical column dimension (b), in the case that $\alpha > 0°$, the width of the column decreases, which leads to an increase in instability and high deformation due to the strong tendency to rotation and collapse under torque produced by some eccentricity of forces (Seyedi Hosseininia, 2012). In other words, particles under $\alpha > 0°$ are vulnerable to sliding, rotation and collapse due to the torque caused by force eccentricities in such columnar microstructures. This explanation can be a good reason for the decrease of the resistance of the specimen with the increase of α°. This behavior has been described by Yoshimine and Ishihara (1998) for Toyora sand, Bahadori et al. (2008) for Firoozkuh sand and Yang et al. (2016) for Leighton sand. Similar behavior was observed in the current study.

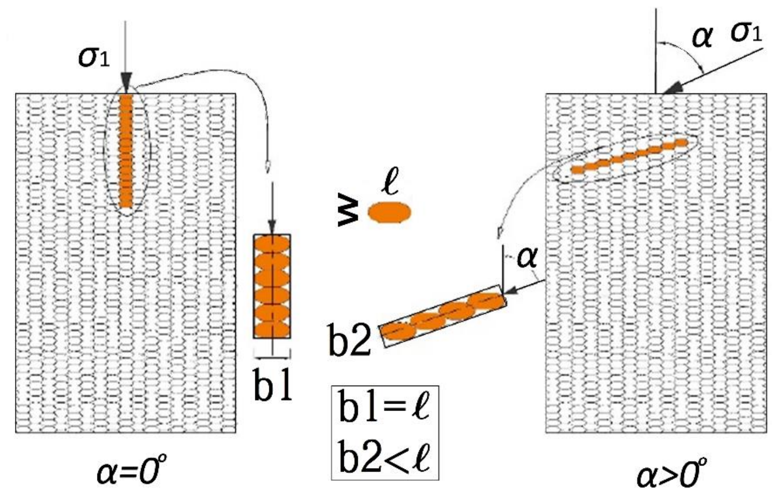



**Figure 9.** Simple explanation of how the load is transferred for columnar microstructures under inclined principal stress (concept from Seyedi Hosseininia ( 2012)).

### *3.2. Effect of clay content (plastic kaolin) on the mechanical behavior of sand-clay mixtures under different inclination angle (α°)*

Figure 10 illustrates the effect of adding different percentages of clay (3, 5, 7 and 10%) on the behavior of sand under the inclination angle of 15° (compressional loading). Addition of 3% clay, the peak shear strength of the sample is increased slightly (about 7%), this small percentage filled the empty space between the sand particles, so that, provides a certain amount of continuity in the sand matrix without causing sliding, But by adding higher percentage of clay up to 10%, the strength of the samples is decreased and the behavior becomes more contractive (about 33%), in fact, the samples up to 10% of clay have an open microstructure with low shear strength. This contractive behavior can be attributed to the two factors, the accumulation fine grains (clay) on the contact surfaces of the sand grains (which tend to lubricate and smooth over on the surfaces) and the high compressibility of clay particles. It should be noted that the greatest percentage of decrease in strength is up to 7% of clay content, so that with the addition of 10%, the decreasing slope of the sample's strength is decreased. This behavior has been observed under both initial confining stress 100 and 200 kPa, but with the increase of initial confining stress, the effect of fine grains on the structure of the host sand is decreased. Similar results (trend of decreasing strength with increasing clay content) have been observed by Talamkhani and Naeini (2021), but with the addition of high percentages of clay (more than 10%), the shear strength of specimen is increased, that is, the effective stress is increased and soil instability is decreased. Therefore, 3% and 10% of clay are turning points of sand's behavior. Figure 11 illustrates the effect of adding clay content (3, 5, 7 and 10%) under $\alpha = 30°$ on the behavior of sand. According to the results, under this angle with compressive-torsional loading, similar to loading at 15°, a slight increase in peak shear strength is observed with the addition of 3% clay (about 6%). Also, by adding a higher percentage of clay up to 10%, as in the case of compressive loading ($\alpha = 15°$), the strength of the specimen is decreased and the behavior becomes more contractive, but this increase is less than the case of compressive loading (about 22%). In other words, with the increase of the inclination angle and applying of torsional loading in addition to compressive loading, the effect of fine grains on the sand structure is decreased.

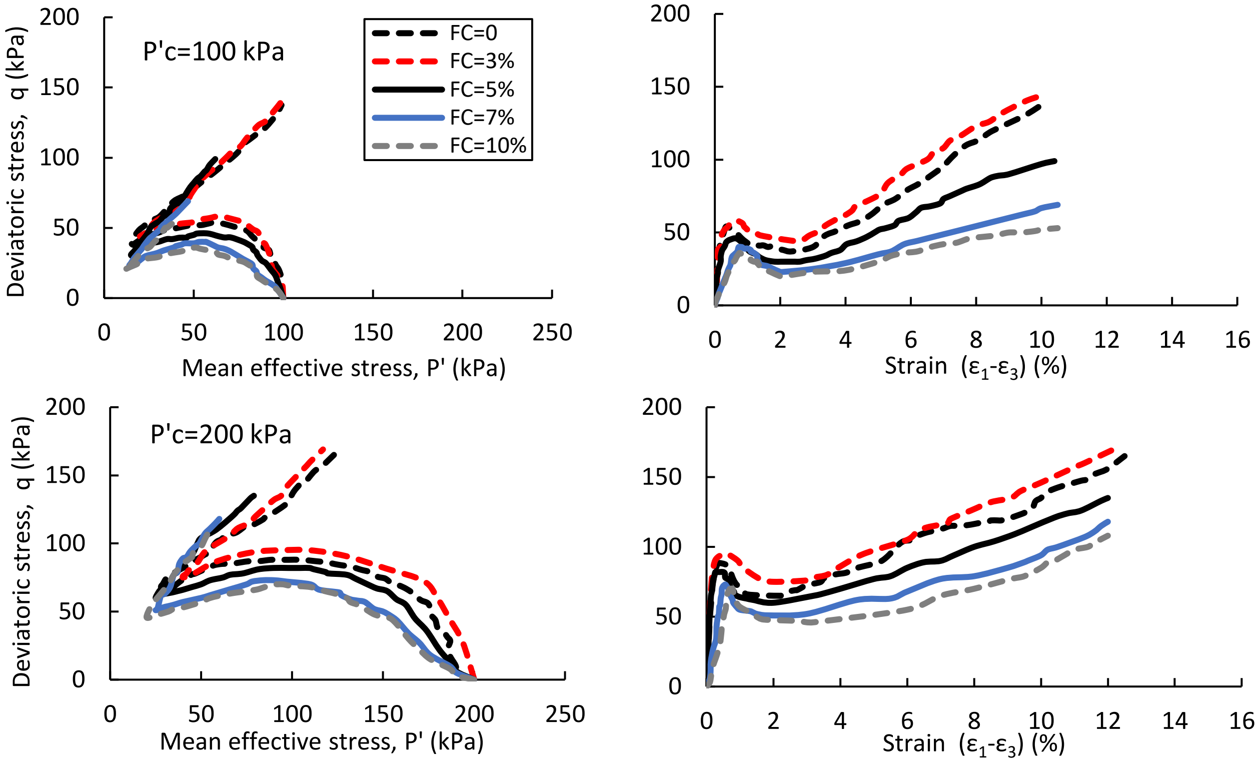



**Figure 10.** Effect of clay content on the mechanical behavior (undrained) of sand–clay mixtures at α=15°

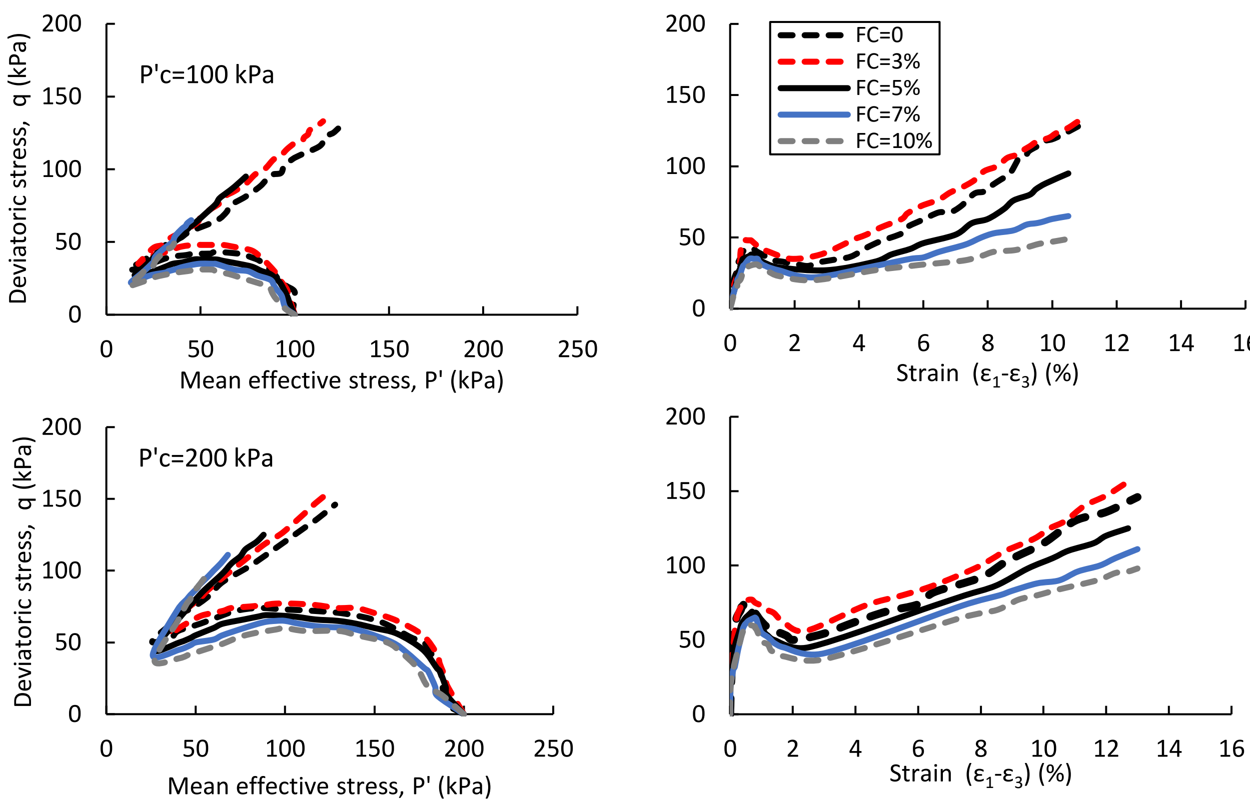



**Figure 11.** Effect of clay content on the mechanical behavior (undrained) of sand–clay mixtures at α=30°

Figure 12 illustrates the effect of adding clay content (3, 5, 7 and 10%) at α = 60° (extensional torsional loading) on the behavior of sand. The specimen under this angle shows a completely contractive and softening behavior, and with the addition of 5% clay, the maximum reduction in peak shear strength is observed, which is less compared to the angles of 15° and 30º, but with a further increase in the clay content, unlike α = 15° and 30°, an increase in the strength of the sample is observed, so that at 10% of clay, the strength of the sample is greater than the host sand (about 7%), which can be attributed to the cohesion nature of the clay particles and resistance to tensile loading. To compare the effect of fine grains in different inclination angle, the results of the relative strength of samples with fine grains to the host sand are presented in Figure 13. Under α = 15°, the highest effect of clay content is observed, which first is increased and then is decreased. A similar behavior is observed at α = 30°, but at α = 60°, a decreasing and then increasing trend is observed.

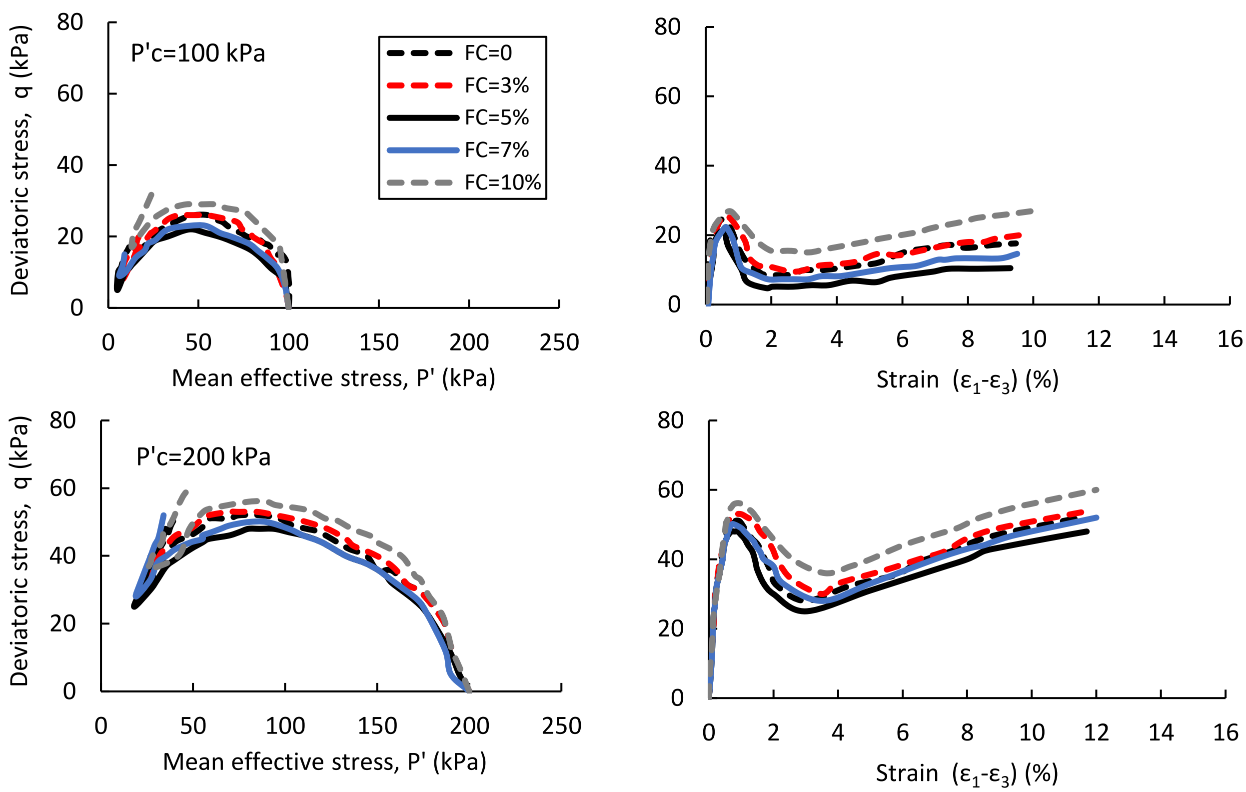
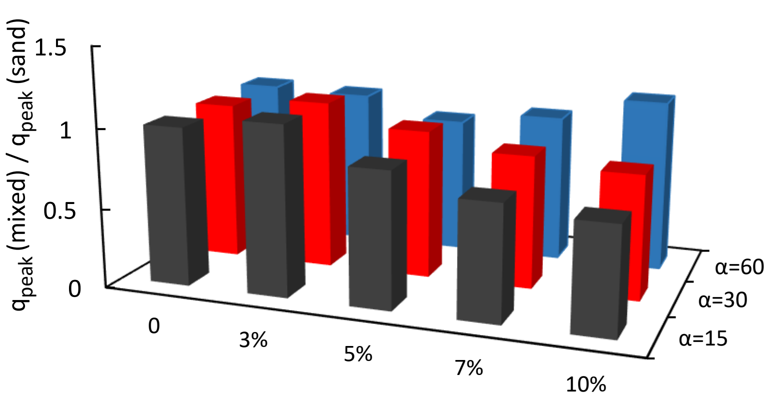



**Figure 12.** Effect of clay content on the mechanical behavior (undrained) of sand–clay mixtures at α=60°.




| FC (%) | 0 | 3 | 5 | 7 | 10 |
|---|---|---|---|---|---|
| α=15 | 1 | 1.07 | 0.85 | 0.72 | 0.67 |
| α=30 | 1 | 1.06 | 0.93 | 0.83 | 0.78 |
| α=60 | 1 | 0.98 | 0.85 | 0.92 | 1.07 |

**Figure 13.** Comparison of the peak strength increase ratio of sand-clay mixture to the host sand at different inclination angles.

### *3.3. Evaluation of different dimensionless parameters*

3.3.1. Brittleness index ($I_B$) and stress ratio

As seen in the stress-strain curves (Figures 10 to 12), the sand-clay mixtures exhibited a strain-softening & hardening behavior; After the peak shear stress, instability occurred in the specimen and the shear strength is decreased to minimum state. The degree of strain softening and strength reduction can be expressed in terms of the brittleness index ($I_B$) defined by Bishop (1971) according to Equation (1).

$$I_B = \frac{q_{peak} - q_{min}}{q_{peak}} \quad (1)$$

where $q_{peak}$ and $q_{min}$ are the peak undrained shear strength prior to quasi steady state and minimum shear strength. $I_B$ could be considered as a good index for the flow potential of a contractive soils (Keramatikerman et al., 2018). The value of $I_B$ is ranged from 0 to 1. Non-flow (non-brittle) behavior is observed when $I_B = 0$. However, static liquefaction (brittle behavior) is associated with $I_B = 1$. As illustrated in Figure 14(a), the brittleness index for mixtures is decreased with increasing confining pressure and is increased with the increase of the inclination angle, so that at α = 60°, the greatest increase in the index is observed. This increase in the brittleness index demonstrates that the liquefaction susceptibility of sandy soil is increased with increasing inclination angle, which is consistent with the results of Sivathayalan and Vaid (2002). Under inclination angles of 15° and 30°, with the addition of 3% clay, the index is decreased slightly, which indicates a decrease in contractive behavior, but with a further increase in clay, the index is increased. But at α = 60°, $I_B$ is increased by 5% clay and then a decreasing trend is observed with a further increase in clay.

Stress ratio ($q_{min}/q_{peak}$) is the ratio between minimum shear strength to peak shear strength and can be a suitable parameter to assessment the liquefaction susceptibility of sandy soil. Static liquefaction is observed when $q_{min}/q_{peak}$=0. Non-flow (non-brittle) behavior is also observed when $q_{min}/q_{peak}$ = 1. As seen from Figure 14(b), the Stress ratio of sand specimens is increased with increasing confining pressure and is decreased with the increase of the inclination angle. Under angles of 15 and 30° when the compressional load is noticeable, the negative effect of Kaolin on the liquefaction susceptibility of sand-clay mixtures is related to the role of Kaolin in reducing the stability of sand fabric.

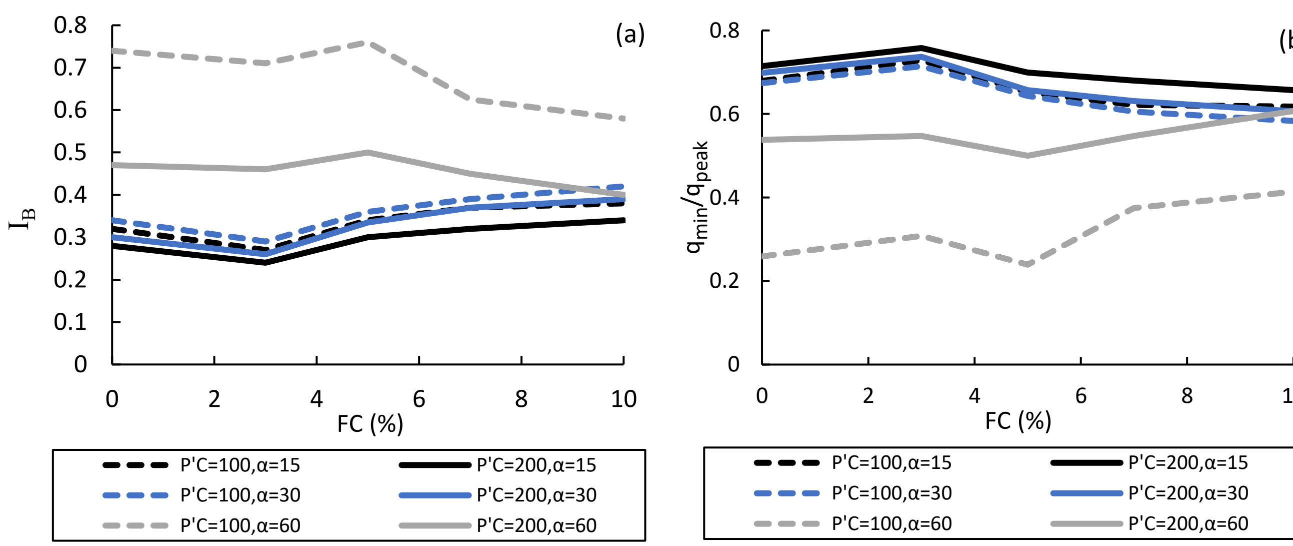
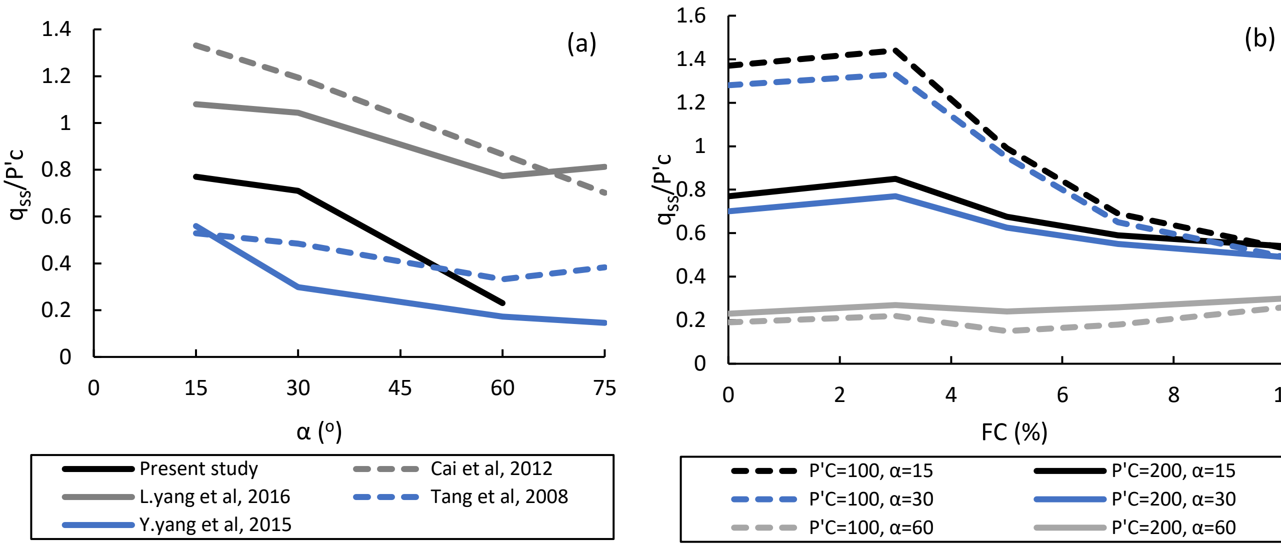



**Figure 14.** Effect of clay content and inclination angle on the behavior of sandy soil: (a) brittleness index, (b) stress ratio.

3.3.2. Steady state strength index ($q_{ss}/P_c'$)

Although shear strength can be represented by a variety of parameters, in order to compare specimens in large strains and steady state strength, this parameter is defined as the ratio of steady state deviatoric stress to confining pressure ($q_{ss}/P_c'$). This parameter has been investigated by various researchers to investigate the effect of the inclination angle, which is presented in Figure 15(a) with the results of this research for host sand (P'c=200 kPa). Based on the results, the steady state strength is decreased with the increase of the inclination angle. As seen from Figure 15(b), at α =15 and 30°, with the addition of 3% clay, the steady state strength index is increased slightly, which indicates a decrease in contractive behavior, but with a further increase in clay, the index is decreases. This decrease in steady state strength that occurred in sand up to FC = 10% is attributed to a small number of fines acting as a lubricating layer between particles. In sandy soil, friction between particles is an effective factor of shear strength and, thus, the presence of fine grains as a layer between particles can cause instability; and consequently, decrease strength of the soil, because the particles can move and slide easily under shearing. But at α = 60° with extensional loading, steady state strength is decreased by 5% clay and then an increasing trend is observed with a further increase in clay. So that at 10% of clay, the strength of the specimen is greater than the host sand, which can be attributed to cohesion nature of the clay particles, that provides a certain amount of cohesion to the sand matrix and creates resistance against extensional loading.




Note: Data on previous studies obtained from (Al-Rkaby et al., 2017)

**Figure 15.** Steady state strength index: (a) steady state strength index at inclination angle (α), and comparison with previous studies for sand; (b) Effect of clay content and inclination angle on the steady state strength index of sand–clay mixtures.

3.3.3. Anisotropy degree ($D_{A,St}$)

Quantitative assessment of anisotropy is very important because doing so, influence of anisotropy on soil behavior can be determined. Admittedly, it seems impossible to propose a single index to quantify anisotropy. In this section, we focus on the strength anisotropy. For this purpose, the anisotropy degree (strength anisotropy) parameter is defined according to Equation (2). This parameter evaluates the effects of different values of inclination angle (α°) and clay content on the anisotropic behavior of the sandy soils.

$$D_{A,St} = \frac{Steady\ State\ Strength\ \ at\ (\alpha^o)}{Steady\ State\ Strength\ \ at\ (\alpha_{max} = 60^o)} = \frac{q_{ss}\ (\alpha^o)}{q_{ss}\ (\alpha_{max} = 60^o)} \tag{2}$$

where $q_{ss}$ $(\alpha^{o})$ = shear strength of steady-state at α°, and $q_{ss}(\alpha = 60^{o})$ = shear strength of steady-state at α=60°. Figure 16 indicates the anisotropy degree of the studied soil. With the increase of initial confining stress, the anisotropy degree is decreased. Also, with the increase in the percentage of fines (clay), anisotropy degree is decreased. In other words, with the increase in the percentage of fine grains, the anisotropic behavior is decreased, which results are consistent with the studies of Bahadori et al. (2008).

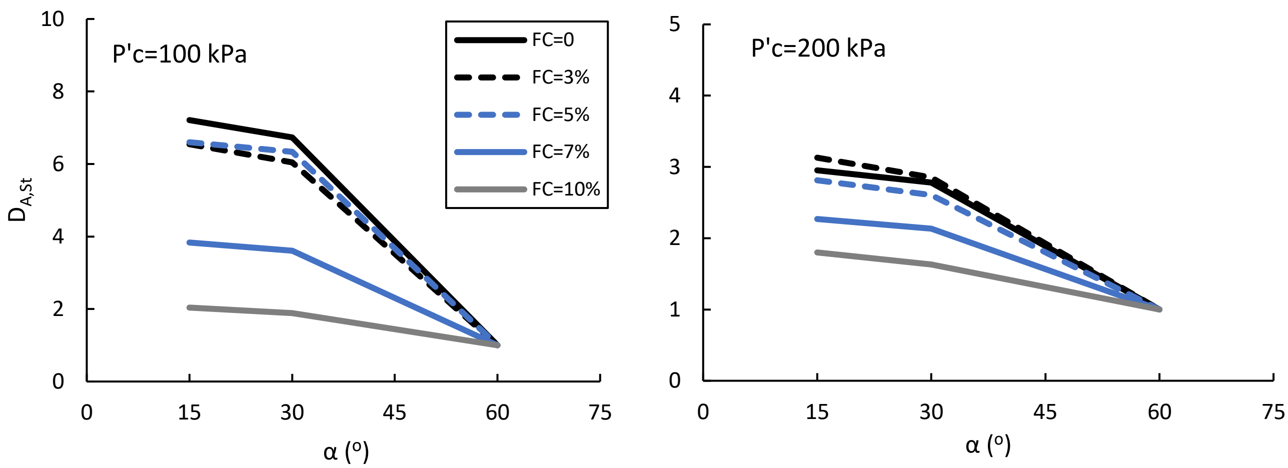



**Figure 16.** Effects of different values of inclination angle (α°) and clay content on the anisotropy degree ($D_{A,St}$).

## 4. Conclusions

This paper presents an experimental investigation of the anisotropic behavior of sands with low plastic clay (kaolin) content as established by a series of undrained torsional shear hollow cylinder apparatus tests. The main findings and conclusions of this article can be summarized as follows:

1. The behavior of the examined sand is a strain softening-hardening behavior. By increasing inclination angle, a decrease in the peak shear strength (22% and 49% for the 30° and 60°, respectively) was observed compared to the 15°. So, as the inclination angle (α°) became higher, the behavior became softer and more contractive. Such strain-softening has been attributed to the inherent anisotropy in the sand fabric during sedimentation. At α=60°, the behavior of the specimen is strongly contractive and the greatest decrease can be observed in the strength of sand specimen. This increase in contraction behavior increases the brittleness index and decreases the stress ratio parameter. In other words, the liquefaction susceptibility of sandy soil is increased with increasing inclination angle.
2. Under the inclination angles of 15° and 30°, with the addition of 3% clay, the peak shear strength of the sample is increased slightly (7% and 6% for $\alpha$=15°and 30°, respectively), this small percentage filled the empty space between the sand particles, so that, provides a certain amount of continuity in the sand matrix without causing sliding, But by adding higher percentage of clay up to 10%, the strength of the samples is decreased and the behavior becomes more contractive (33% and 22% for $\alpha$=15°and 30°, respectively), in fact, the samples up to 10% of clay have an open microstructure with low shear strength. It should be noted that the number of changes in the strength of the specimen at α=30° is less than α=15°.
3. At α = 60° (extensional torsional loading), The specimen shows a completely contractive and softening behavior, and with the addition of 5% clay, the maximum reduction in peak shear strength (and steady state strength) is observed, which is less compared to the angles of 15° and 30°, but with a further increase in the clay content, unlike α = 15° and 30°, an increase in the strength of the sample is observed (about 7%), so that at 10% of clay, the strength of the sample is greater than the host sand, which can be attributed to the cohesion nature of the clay particles, that provides a certain amount of cohesion to the sand matrix and creates resistance against extensional loading.
4. Under inclination angles of 15° and 30°, when the compressional load is noticeable, with the addition of 3% clay, the brittleness index is decreased slightly (about 15%) and stress ratio is increased (about 6%), which indicates a

decrease in contractive behavior. But with a further increase in clay, the brittleness index is increased and stress ratio is decreased. In other words, the liquefaction susceptibility of sandy soil is increased with increasing clay content. The negative effect of kaolin on the liquefaction susceptibility of sand-clay mixtures is related to the role of kaolin in reducing the stability of sand fabric.

5. For quantification of anisotropy, the anisotropy degree (strength anisotropy, $D_{A,St}$) parameter was defined. This parameter evaluates the effects of different values of inclination angle (α°) and clay content on the anisotropic behavior of the sandy soils. With the increase of initial confining stress, the anisotropy degree is decreased. Also, with the increase of fines content (clay), anisotropy degree is decreased. In other words, with the increase in the percentage of fine grains, the anisotropic behavior is decreased.
6. By evaluating field studies conducted after large earthquakes and case histories of actual soil behavior, it is observed that many soils have a low percentage of fine grains (<10%), Therefore, it is necessary to conduct more studies to investigate the behavior of these soils with variables such as plasticity of fines, clay minerals, ample preparation technique, morphological parameters and grading parameters of sands.

## Nomenclature

| List of symbol and abbreviations | |
|---|---|
| α° | Inclination angle |
| b | Intermediate principal stress parameter |
| TSHCA | Torsional shear hollow cylindrical apparatus |
| SEM | Scanning electron microscope |
| XRF | X-Ray fluorescence |
| B | Skempton's parameter |
| FC | Fines content |
| P'c | Initial mean effective stress |
| QSS | Quasi-steady state point |
| $I_B$ | Brittleness index |
| $D_{A,St}$ | Anisotropy degree |

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## References

Al-Rkaby, A. H., Chegenizadeh, A., & Nikraz, H. (2017). Anisotropic strength of large scale geogrid-reinforced sand: experimental study. Soils and foundations, 57(4), 557-574. https://doi.org/10.1007/s40891-017-0111-9

Arthur, J., & Menzies, B. (1972). Inherent anisotropy in a sand. Géotechnique, 22(1), 115-128.

ASTM. (2006a). ASTM. D4253: Standard test methods for maximum index density and unit weight of soils using a vibratory table. In: West Conshohocken, PA, USA: ASTM International.

ASTM. (2006b). ASTM. D4254: Standard test methods for minimum index density and unit weight of soils and calculation ofrelative density. In: West Conshohocken, PA, USA: ASTM International.

Bahadori, H., Ghalandarzadeh, A., & Towhata, I. (2008). Effect of non plastic silt on the anisotropic behavior of sand. Soils and foundations, 48(4), 531-545. https://doi.org/10.3208/sandf.48.531.

Bensoula, M., Bousmaha, M., & Missoum, H. (2022). Relative density influence on the liquefaction potential of sand with fines. Revista de la construcción, 21(3), 693-702. http://dx.doi.org/10.7764/rdlc.21.3.692

Bensoula, M., Missoum, H., & Bendani, K. (2018). Liquefaction potential sand-silt mixtures under static loading. Revista de la Construcción. Journal of Construction, 17(2), 196-208. https://doi.org/10.7764/RDLC.17.2.196

Bishop, A. W. (1971). Shear strength parameters for undisturbed and remolded soil specimens. In Roscoe Memorial Symposium, Cambridge University Press, Cambridge, 3-58.

Bouferra, R., & Shahrour, I. (2004). Influence of fines on the resistance to liquefaction of a clayey sand. Proceedings of the Institution of civil engineers-ground improvement, 8(1), 1-5. https://doi.org/10.1680/grim.2004.8.1.1

Cai, Y., Yu, H.-S., Wanatowski, D., & Li, X. (2013). Noncoaxial behavior of sand under various stress paths. Journal of Geotechnical and Geoenvironmental Engineering, 139(8), 1381-1395. https://doi.org/10.1061/(ASCE)GT.1943-5606.00008

Casagrande, A. (1944). Shear failure of anisotropic materials. Proc. Boston Soc. Civ. Engrs, 31, 74-87.

Cherif Taiba, A., Mahmoudi, Y., Belkhatir, M., Kadri, A., & Schanz, T. (2018). Experimental characterization of the undrained instability and steady state of silty sand soils under monotonic loading conditions. International Journal of Geotechnical Engineering, 12(5), 513-529. https://doi.org/10.1080/19386362.2017.1302643

Chu, J., & Leong, W. (2002). Effect of fines on instability behaviour of loose sand. Géotechnique, 52(10), 751-755. https://doi.org/10.1680/geot.2002.52.10.751

Georgiannou, V., Burland, J., & Hight, D. (1990). The undrained behaviour of clayey sands in triaxial compression and extension. Géotechnique, 40(3), 431-449. https://doi.org/10.1680/geot.1990.40.3.431

Ghahremani, M., & Ghalandarzadeh, A. (2006). Effect of plastic fines on cyclic resistance of sands. In Soil and rock behavior and modeling, 406-412. https://doi.org/10.1061/40862(194)54

Guo, C., & Cui, Y. (2020). Pore structure characteristics of debris flow source material in the Wenchuan earthquake area. Engineering Geology, 267, 105499. https://doi.org/10.1016/j.enggeo.2020.105499

Guo, P. J., & Stolle, D. F. (2005). On the failure of granular materials with fabric effects. Soils and foundations, 45(4), 1-12. https://doi.org/10.3208/sandf.45.4_1

Ishihara, K. (1993). Liquefaction and flow failure during earthquakes. Géotechnique, 43(3), 351-451.

Ishihara, K., & Towhata, I. (1983). Sand response to cyclic rotation of principal stress directions as induced by wave loads. Soils and foundations, 23(4), 11-26. https://doi.org/10.3208/sandf1972.23.4_11

Keramatikerman, M., Chegenizadeh, A., Nikraz, H., & Sabbar, A. S. (2018). Effect of flyash on liquefaction behaviour of sand-bentonite mixture. Soils and foundations, 58(5), 1288-1296. https://doi.org/10.1016/j.sandf.2018.07.004

Khayat, N., Ghalandarzadeh, A., & Jafari, M. K. (2014). Grain shape effect on the anisotropic behaviour of silt–sand mixtures. Proceedings of the Institution of Civil Engineers-Geotechnical Engineering, 167(3), 281-296. https://doi.org/ 10.1680/geng.11.00093

Koester, J. P. (1994). The influence of fines type and content on cyclic strength. In Ground failures under seismic conditions, ASCE, 17-33.

Krim, A., Arab, A., Chemam, M., Brahim, A., Sadek, M., & Shahrour, I. (2019). Experimental study on the liquefaction resistance of sand–clay mixtures: Effect of clay content and grading characteristics. Marine Georesources & Geotechnology, 37(2), 129-141. https://doi.org/10.1080/1064119X.2017.1407974

Kumruzzaman, M., & Yin, J.-H. (2010). Influences of principal stress direction and intermediate principal stress on the stress–strain–strength behaviour of completely decomposed granite. Canadian Geotechnical Journal, 47(2), 164-179. https://doi.org/10.1139/T09-079

Lade, P. V., Nam, J., & Hong, W. P. (2008). Shear banding and cross-anisotropic behavior observed in laboratory sand tests with stress rotation. Canadian Geotechnical Journal, 45(1), 74-84. https://doi.org/10.1139/T07-078

Li, X., & Yu, H.-S. (2009). Influence of loading direction on the behavior of anisotropic granular materials. International Journal of Engineering Science, 47(11-12), 1284-1296. https://doi.org/10.1016/j.ijengsci.2009.03.001

Liu, X., Zhang, X., Kong, L., Yin, S., & Xu, Y. (2022). Shear strength anisotropy of natural granite residual soil. Journal of Geotechnical and Geoenvironmental Engineering, 148(1), 04021168. https://doi.org/10.1061/(ASCE)GT.1943-5606.0002709

Mohammadi, V., & Bahadori, H. (2024). Influence of Low Silt Content on the Anisotropic Behaviour of Sand. International Journal of Civil Engineering. 22, 1507–1522. https://doi.org/10.1007/s40999-024-00964-3

Nishimura, S., Jardine, R., & Minh, N. (2011). Shear strength anisotropy of natural London clay. In Stiff Sedimentary Clays: Genesis and Engineering Behaviour: Géotechnique Symposium in Print, Thomas Telford Ltd, 97-110. https://doi.org/10.1680/ssc.41080.0009

Oda, M. (1972). Initial fabrics and their relations to mechanical properties of granular material. Soils and foundations, 12(1), 17-36.

Ohsaki, Y. (1970). Effects of sand compaction on liquefaction during the Tokachioki earthquake. Soils and foundations, 10(2), 112-128.

Peng, D., Xu, Q., Liu, F., He, Y., Zhang, S., Qi, X., Zhao, K., & Zhang, X. (2018). Distribution and failure modes of the landslides in Heitai terrace, China. Engineering Geology, 236, 97-110. https://doi.org/10.1016/j.enggeo.2017.09.016

Prasanna, R., Sinthujan, N., & Sivathayalan, S. (2020). Effects of initial direction and subsequent rotation of principal stresses on liquefaction potential of loose sand. Journal of Geotechnical and Geoenvironmental Engineering, 146(3), 04019130. https://doi.org/10.1061/(ASCE)GT.1943-5606.0002182

Radjai, F., & Azéma, E. (2009). Shear strength of granular materials. European journal of environmental and civil engineering, 13(2), 203-218. https://doi.org/10.1080/ 19648189.2009.9693100

Saada, A. (1988). Hollow cylinder torsional devices: Their advantage and limitations, advanced triaxial testing of soil and rock. ASTM STP, 977, 766-789.

Seyedi Hosseininia, E. (2012). Investigating the micromechanical evolutions within inherently anisotropic granular materials using discrete element method. Granular Matter, 14(4), 483-503. https://doi.org/10.1007/s10035-012-0340-5

Shibuya, S., & Hight, D. (1987). A bounding surface for granular materials. Soils and foundations, 27(4), 123-136.

Symes, M. J. P. R. (1983). Rotation of principal stresses in sand. Ph.D. thesis, College of Science, Technology and Medicine, University of London.

Talamkhani, S., & Naeini, S. A. (2021). The undrained shear behavior of reinforced clayey sand. Geotechnical and Geological Engineering, 39, 265-283. https://doi.org/10.1007/s10706-020-01490-4

Tokimatsu, K., & Yoshimi, Y. (1983). Empirical correlation of soil liquefaction based on SPT N-value and fines content. Soils and foundations, 23(4), 56-74.

Triantafyllos, P. K. (2020). Experimental study of the anisotropic flow deformation and critical state of sand. PhD thesis, National Technical University of Athens, Athens, Greece.

Tsuchida, H. (1970). Prediction and countermeasure against the liquefaction in sand deposits. In Proceedings of the seminar of the Port and Harbour Research Institute, 3, 3.1-3.33.

Uthayakumar, M., & Vaid, Y. (1998). Static liquefaction of sands under multiaxial loading. Canadian Geotechnical Journal, 35(2), 273-283.

Wrzesiński, G., & Lechowicz, Z. (2015). Testing of undrained shear strength in a hollow cylinder apparatus. Studia Geotechnica et Mechanica, 37(2), 69-73.

Xiong, H., Guo, L., Cai, Y., & Yang, Z. (2016). Experimental study of drained anisotropy of granular soils involving rotation of principal stress direction. European journal of environmental and civil engineering, 20(4), 431-454. https://doi.org/10.1080/19648189.2015.1039662

Yang, L.-T., Li, X., Yu, H.-S., & Wanatowski, D. (2016). A laboratory study of anisotropic geomaterials incorporating recent micromechanical understanding. Acta Geotechnica, 11, 1111-1129. https://doi.org/10.1007/s11440-015-0423-7

Yoshimine, M., & Ishihara, K. (1998). Flow potential of sand during liquefaction. Soils and foundations, 38(3), 189-198.

Yoshimine, M., Ishihara, K., & Vargas, W. (1998). Effects of principal stress direction and intermediate principal stress on undrained shear behavior of sand. Soils and foundations, 38(3), 179-188.

Zeng, Y., Shi, X., Chen, W., & Feng, W. (2023). Equivalent Compression Curve for Clay–Sand Mixtures Using Equivalent Void-Ratio Concept. International journal of geomechanics, 23(2), 06022039. https://doi.org/10.1061/(ASCE)GM.1943-5622.0002643