Effect of Stress Direction on the Undrained Monotonic and Cyclic Behaviour of Dense Sands

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ABSTRACT:

Geotechnical design may be unsafe if the anisotropic behaviour of soil is not considered. The behaviour of anisotropic materials depends on the principal stresses and their directions. A detailed experimental programme was conducted to study the effect of stress direction on the monotonic and cyclic behaviour of dense sand. A total of 20 undrained tests were performed at a constant mean confining stress ($\sigma'_0m$) constant intermediate principal stress ratio ($b= (\sigma_2-\sigma_3)/(\sigma_1-\sigma_3)$), and principal stress directions ($\alpha$). Two fine sands, Babolsar and Toyoura, were selected as the test materials. The isotropic consolidated specimens were prepared using the wet tamping technique. The results showed that the major principal stress direction had little considerable effect on the mobilized friction angle at steady state or phase transformation. The results showed that stress direction had a significant effect on the non-coaxiality between the principal strain increment direction and the principal stress direction. The soil fabric was led to significant non-coaxiality value before the peak shear strength. Increasing the octahedral shear strains decreased the non-coaxiality value due to destruction of the soil particle interlock (soil fabric). The effect of stress direction on non-coaxiality and excess pore water pressure generation was also investigated.

Keywords: direction of major principal stress, sand, pore water pressure, non-coaxiality,
1. Introduction

The effect of major principal stress direction, $\alpha$, relative to the vertical axis on the monotonic and cyclic behaviour of soil is relevant to the development of constitutive models. No comprehensive investigation on the effect of stress direction on the monotonic and cyclic behaviour of soil has been published. In particular, the axes of the principal stress vary in many conventional tests apparatuses and cannot be controlled during testing. However, laboratory tests such as simple shear test, directional cell test, and torsional hollow cylinder test are able to control the direction of the principal axes of stress [1-7]. The hollow cylinder apparatus is widely-used device that can be used to investigate the anisotropic behaviour of soil [5,6,8-10]. A brief review of this apparatus can be found in Hight et al. [7].

The impact of anisotropy on the behaviour of sand has long been recognized [11 and 12]. Anisotropy refers to any directional dependence in mechanical characteristics. Many research works showed the anisotropy is one of the most important parameters influencing soil behaviour [11-22]. Casagrande and Carillo [23] observed that soils exhibit two types of anisotropy (i.e. inherent and induced anisotropy). During the deposition process the soil particles tend to adopt a stable position. The soil masses are formed depending on particle shape, voids, type of deposition, spatial arrangement, and associated contact. This will led to an initial anisotropy in the fabric of granular soils which is defined as inherent or fabric anisotropy. The inherent anisotropy will cause the response of soil be dependent the orientation of principal stresses with reference to the depositional plane, $\alpha$. Numerous researchers systematically investigated the fabric anisotropy of sand [12, 24-28]. These research works showed that even perfectly rounded particles of granular soil exhibit inherent anisotropy due to initial fabric. Moreover, the microfabric studies confirmed that the particles may slide or move to adopt a stable structure and inter-particle contact becomes preferentially
oriented in the direction of deposition.

Changes in the magnitude and/or direction of stress exerts more anisotropy which is defined as induce anisotropy. Induced anisotropy is formed by plastic deformation due to anisotropic external loading [23]. This definition provides a convenient distinction between inherent and induced anisotropy, which was later modified [11, 28-30] to involve previous strain as a part of the initial anisotropy and consider previously induced anisotropy as the initial anisotropy for subsequent conditions [22]. Many researchers simulate some field loading situations, such as wave loading on seabed deposits, multidirectional earthquake loading in level ground, and lateral cyclic loading on the soil behind retaining structures, involve rotation of the principal stress directions during shear in the laboratory tests [21, 31, and 32]. The stress paths associated with these loading situations may be classified as non-proportional loading [18, 21, and 33]. However, the boundary conditions in these studies varied greatly, and the primary focus was on regenerating simple shear conditions rather than systematically investigating the effect of principal stress rotation and intermediate principal stress on the cyclic behaviour of sands. These studies showed that the rotation of the principal axis can cause liquefaction or strength reduction at a constant deviator stress.

Intermediate principal stress parameter, b, is not an influencing factor for the behaviour of saturated sands. Many studies have reported that, for a given value of α, the intermediate principal stress parameter, b, has little or no effect on the response of the sand (e.g. shear strength and internal friction angle) to monotonic and cyclic loading [29-40]. These studies and others have concluded that α has a significant influence on the response of the sands; however, the effect of b on liquefaction and pore water pressure build-up has also been reported. Yang et al. [33] observed that, under otherwise identical conditions, specimens tested at b = 0.0 exhibited greater resistance and the highest pore pressure build-up occurred at b = 1.0. In general, the effect of intermediate principal stress on soil behaviour is not
significant when compared with the major principal stress direction, especially for dense sand.

The effects of major principal stress direction on the soils behaviour were investigated in the tests with the fixed direction of principal stress [41-48]. The main purpose of such experiments were to clarify uncertainties in the determination of the shear strength (e.g. steady state or peak shear strength), internal friction angle, and the non-coaxiality which is defined as the no-coincidence between the principal stress axis and the principal strain rates axis. It has been found that, for the certain conditions of soil, the undrained behaviour of sands is influenced by the shearing mode. Most of these research works were focused on the behaviour of soils subjected to the monotonic loading. Therefore, more undrained tests should be performed to clarify the effects of principal stress direction on the response of soils under cyclic loading. To achieve this goal, a systematic program consisting of 20 undrained monotonic and cyclic torsional shear tests was performed on dense Babolsar and Toyoura sands under fixed major principal stress, $\alpha$. The specimens were prepared by wet tamping technique. The influences of principal stress direction on the soil responses were isolated while other parameters (e.g. construction technique, relative density, confining stress) were held constant. The applied torsional shear apparatus was able to control both the direction and magnitude of principal stresses. Using this device, which allows for the creation of three different principal stresses, it is possible to apply continuous controlled increments and/or rotations of principal stresses in the vertical plane of the hollow cylinder specimen. The stress-strain behaviour was measured carefully to make a reliable analysis of the soil behaviour. This paper aims to study the undrained anisotropic response of sands under a fixed direction of the major principal stress. The soil behaviour was investigated for a pattern of $\alpha$, varying from $10^\circ$ to $80^\circ$ (i.e. $\alpha=10^\circ$, 30°, 45°, 60°, and 80°). Based on the experimental results, the effects of shearing mode and sand type are discussed.
The notation used throughout the paper is given in Table 1.

2. Laboratory Testing Procedure

2.1. Torsion shear apparatus

The closed-loop control system of the dynamic hollow cylinder apparatus (manufactured by Wykeham Farrance International Company) used in this study has five main components: (1) hollow cylinder software, (2) high-speed data acquisition system (DAS), (3) servo valves, (4) vertical and horizontal actuators, and (5) load, pressure, and displacement transducers (Fig. 1). This system is computer controlled with five control channels and up to sixteen data acquisition channels with 20 bit closed-loop controlled and data acquisition system. The main specifications of the hollow cylinder specimens are as follows: 150mm triaxial cell with a 100mm outside diameter, 60mm inner diameter, and 200mm height. The Axial actuator can apply an axial load of +/- 10kN, with a stroke of +/- 25mm; and the torsional actuator can apply a torsional force of +/-200Nm, with 90 degrees of rotation. The transducer resolutions for axial and rotational measurement are: axial load $\leq$1N, axial displacement encoder $\leq$0.01mm, torque $\leq$0.01Nm, rotational encoder $\leq$1.5micron. These actuators are able to control up to a 5Hz single axis. The inner cell, outer cell, and back pressures are applied through digitally controlled air valves in a closed-loop with the inner and outer cell and back pressure transducers. This fully automated system allows applying frequencies up to 1 Hz on simultaneous 5-axis control, which is essential to apply magnitude and direction of the major and minor principal stresses. A compact self-contained unit provides all critical control, timing, and data acquisition functions for the test and the transducers.

2.2. Tested materials and specimen preparation

Two uniformly-graded sands, Babolsar and Toyoura, a standard Japanese sand, were selected
as test materials. Apart from the grading, the size and shape of the sand particles emerged as significant parameters that effect the response of sand in a remarkable manner. The selected materials are fine sand with relatively similar particle shapes. The Babolsar sand has slightly coarser particles than Toyoura sand; thus, the effect of particle size could be investigated throughout the tests. The particles of Babolsar sand are sub-rounded to sub-angular as illustrated in Fig. 2(a). The Babolsar sand was obtained from the South coast of the Caspian Sea. Toyoura sand is uniform fine material with mainly sub-angular particles shape (Fig. 2(b)). Particle size distribution curves of these two clean sands are shown in Fig. 3. Index tests were performed based on ASTM standards [49-52]. Physical properties of soils used in the experimental program are summarized in Table 2. According to the USCS definition, these two sands can be characterised as poorly-graded sands (SP).

The HCA specimens were prepared by wet tamping technique to minimize the degree of inherent anisotropy. In addition, the moist under compaction technique was used to obtain specimens with highly uniform density over its height [53]. This technique has been applied extensively both in research and consulting practice [53 and 54]. The relative density of the dense specimens after consolidation and prior to shearing was about 75% (71%-79%). For saturation, carbon dioxide was circulated through the specimen to displace any air from the soil pores. Full saturation of the soil specimens was achieved through application of relatively low back pressures. After a minimum acceptable B-value (Skempton’s parameter) of 0.96 was obtained, the consolidation procedure was initiated. The specimens were isotropically consolidated to a mean effective stress, $\sigma_{0m}$, of 150kPa. For more precise control of $\alpha$ and $b$, the internal and external cell pressures of dense specimens were kept constant and therefore $\alpha$ and $b$ were related by $b=\sin^2(\alpha)$. The axial load, horizontal torque, and back pressure were all kept constant. Following consolidation, the specimens were subjected to the monotonic or cyclic loading. Two set of tests were performed in this study:
monotonic tests were conducted in strain controlled manner by increasing deviator stress, \( q \), until failure (Fig. 4(a)). Monotonic shear loading was applied on the dense specimens which major principal stress inclination, \( \alpha \), was fixed at 10°, 30°, 45°, 60°, and 80° (Fig. 4(b)). The second type of tests were cyclic tests which the cyclic load was applied in stress controlled manner by a constant deviator stress ratio, \( q/\sigma'_0 \) (Fig. 4(a)). Cyclic shear load performed at fixed \( \alpha \) (\( i.e. \alpha=10°, 30°, 45°, 60°, \) and 80°). Stress paths of these tests are shown in Fig. 4(c).

The typical effective stress path, \( p'-q \), curves of some monotonic and cyclic tests conducted on the dense specimens are shown in Fig. 5. The mean normal effective stress is reduced to a critical value due to the undrained condition of loading (\( i.e. \) liquefaction in loose specimens and failure in the dense ones). The classic curves of \( p'-q \) shows good agreement in the monotonic and cyclic tests at different stress paths.

Because loading frequency has little to no effect on the soil behaviour in liquefaction testing [55 and 56], all tests were performed at 0.2 Hz to ensure equilibration of pore water pressure throughout the specimen and thus provide more accurate pore pressure measurements.

Twenty tests under a controlled state of principal stress direction, \( \alpha \), and intermediate principal stress parameters, \( b \), were conducted to investigate the effect of stress direction on the undrained behaviour of sands. The tests program and specimens specifications are summarized in Table.3.

### 2.3. Calculation of stresses and strains

The average stresses and strains for thin-walled cylinders were determined from the following expressions [45, 7]:

**Vertical stress:**

\[
\sigma_z = \frac{F_v}{\pi (r_o^2 - r_i^2)} + \frac{p_o r_o^2 - p_i r_i^2}{r_o^2 - r_i^2}
\]  

(1)
In which $F_v$ is the vertical load, $p_o$ and $p_i$ are the outside and the inside pressures, and $r_o$ and $r_i$ are the outside and the inside radii of the hollow cylinder.

Radial stress:

$$\sigma_r = \frac{p_o r_o + p_i r_i}{r_o + r_i} \quad (2)$$

Tangential stress:

$$\sigma_\theta = \frac{p_o r_o - p_i r_i}{r_o - r_i} \quad (3)$$

Shear stress:

$$\tau_{z\theta} = \frac{3 T}{2\pi (r_o^3 - r_i^3)} \quad (4)$$

in which $T$ is the torque applied to twist the hollow cylinder. Corrections were applied to the data after the tests were performed due to vertical piston uplift, membrane strength, etc.

Vertical strain:

$$\varepsilon_z = \frac{\Delta H}{H_0} \quad (5)$$

in which $H_0$ is the initial height of specimens and $\Delta H$ is the change in height of the hollow cylinder specimen.

Radial strain:

$$\varepsilon_r = \frac{l_o - l_i}{r_o - r_i} \quad (6)$$

Tangential strain:

$$\varepsilon_\theta = \frac{l_o + l_i}{r_o + r_i} \quad (7)$$

in which $l_o$ and $l_i$ are the changes in outside and inside radii determined from the following expressions:
\[ l_o = \sqrt{\frac{\pi (H_o r_o^2) + \Delta I_{vol} + \Delta V}{\pi h}} - r_o \]  
(8)

\[ l_i = \sqrt{\frac{\pi (H_o r_i^2) + \Delta I_{vol}}{\pi h}} - r_i \]  
(9)

in which \( \Delta I_{vol} \) and \( \Delta V \) are the changes in volumes of the inner cell and the specimen, respectively.

Shear strain:

\[ \varepsilon_{\theta} = \frac{\Delta \theta (r_o^3 - r_i^3)}{3H (r_o^2 - r_i^2)} \]  
(10)

in which:

\[ \Delta \theta = \frac{\Delta H_{LVDT}}{r_{measurement \, plate}} \]  
(11)

where \( \Delta H_{LVDT} \) is the recorded change in the horizontal LVDT reading and \( r_{measurement \, plate} \) is the distance from the center to the radio wire cord of the pie-shaped measurement plate.

Major principal stress direction to the vertical axis, \( \alpha \):

\[ \alpha = \frac{1}{2} \tan^{-1}\left(\frac{\tau_{\theta \theta}}{\sigma_z - \sigma_{\theta \theta}}\right) \]  
(12)

Major principal strain increment direction to the vertical axis, \( \alpha_{dc} \):

\[ \alpha_{dc} = \frac{1}{2} \tan^{-1}\left(\frac{d\gamma_{\theta \theta}}{d\varepsilon_z - d\varepsilon_{\theta \theta}}\right) \]  
(13)

Non-coaxiality between the principal stress and strain increment directions, \( \zeta \):

\[ \zeta = \alpha_{dc} - \alpha \]  
(14)

3. Test Results and Discussion

3.1. Effect of stress direction on internal friction angle at steady state
The shear strength of granular material generally results from internal friction. Many researchers have investigated the parameters affecting the internal friction angle \([57-64]\). Sadrekarimi and Olson conducted a comprehensive review of the factors affecting the internal friction angle \([63]\). They reported that internal friction originates from interparticle sliding friction and the geometrical interface. The friction mobilized by the sliding of two adjacent particles is referred to as interparticle sliding friction \(\phi'_m\), and is mainly affected by particle surface roughness \([57]\). Confining stress has no significant effect on interparticle sliding friction. Geometrical interface friction \(\phi'_g\) is affected by the geometrical characteristics of the particles, and their movement mechanisms. Geometrical interface resistance can be divided to two major parts: (a) resistance mobilized by the interface of particles, which arises from particles pushing against, climbing over, and damaging adjacent particles \(\phi'_d\); and (b) resistance mobilized by particle rearrangement, and damage \(\phi'_p\) \([57]\). The geometrical interface angle \(\phi'_d\) range increases from 0º (at high effective confining pressures) to 30º (at low effective confining pressures), and depends on the initial density, effective confining stress, and surface roughness of the particles \([60-62]\).

The mobilized friction angle \(\phi'_\text{mob}\) can be written as:

\[
\phi'_\text{mob} = \phi'_m + \phi'_g = \phi'_m + \phi'_d + \phi'_p \tag{4}
\]

Determination of the steady state friction angle for the dense sand contains uncertainties. Moreover, the effects of loading mode and particle crushing (at high effective stress) on the steady state line is questionable. In addition to the above uncertainties, determination of steady state condition is somewhat difficult. In this study, the failure condition is presumed to be steady state condition, although it may not be exactly so. The failure of dense sand was defined as the first occurrence of either 6% double amplitude (DA) or 6% single amplitude (SA) shear strain. The stress paths of deviator stress and mean normal effective stress of Babolsar and Toyoura sands are shown in Fig.6. The monotonic tests were applied for
locating the failure line more accurately. As illustrated in this figure, changes of $\alpha$ have no significant effect on the internal friction angle at steady state ($\varphi_{ss}$) or phase transformation ($\varphi_{PT}$) in dense Babolsar and Toyoura sands. Many studies have reported that the direction and rotation of the principal stress greatly influence the internal friction angle at steady state [65-68]. Towhata and Ishihara [69] and Arthur et al. [2] reported, however, the angle of shearing resistance is not affected by anisotropy induced by the principal stress direction. In the current study, the effect of stress anisotropy on the internal friction angle was investigated at high levels of strain. The results were in agreement with the observations of Towhata and Ishihara [69] and Arthur et al. [2]. It should be noted that the uniqueness of the steady state at a given void ratio is vague. Some test results support the idea of a unique steady state, while others indicate that the strain rate, the deformation mode, the loading path (compression or extension), and the deposition mode might affect the steady state line [70]. The dense condition of specimens made the contribution of rearrangement ($\varphi'_p$) to decrease and geometrical interference to be dominated by dilation ($\varphi'_d$). Also, $\varphi'_d$ tends to become zero at steady state. Conversely, restriction of particles against rearrangement (as a result of the dense condition of specimens) reduced the effect of loading direction on the geometrical interference at failure. The degree of anisotropy is generally associated with the initial fabric of the sand. An increase in strain will destroy particle interlock and the initial fabric. This will decrease the degree of anisotropy as the strain increases and the sand behaviour will become isotropic. Thus, the effect of stress anisotropy on the internal friction angle will decrease due to the destruction of the soil fabric.

The results revealed that for a particular void ratio, confining stress and laboratory procedure, the friction angle at steady state remained unchanged in dense sand and was not affected by the stress direction (Fig. 6).
3.3. Effect of stress direction on non-coaxiality

Non-coaxiality can be defined as the non-coincidence of the principal stress axis and the principal strain rate axis. Conventional elastoplasticity theory states that, for isotropic materials subjected to proportional loading, the principal directions of strain increment should always be the same as those of the principal stress tensor. Strong experimental and micromechanics-based evidence suggest, however, the coaxiality cannot be satisfied in granular materials [71].

The direction of major principal stress and major principal strain increments for dense Babolsar and Toyoura sands in shown in Fig.7 and Fig.8. Variation of major principal stress/strain increments directions was plotted with \( N/N_f \) in cyclic tests and \( \gamma_{oct} \) in monotonic ones. The N is the number of cycles, the \( N_f \) is the number of cycles to the failure of specimens, and the \( \gamma_{oct} \) is the octahedral shear strain.

In both monotonic and cyclic tests, the major strain increment directions for \( \alpha=10^\circ \) and \( 30^\circ \) are slightly higher than the stress directions. For \( \alpha=45^\circ \), \( 60^\circ \) and \( 90^\circ \), the major principal strain increment directions are slightly lower than the major principal stress directions which is due to the cross anisotropic nature of the specimen (i.e. the bedding planes are horizontal). At \( \alpha=10^\circ \), \( 30^\circ \), the horizontal bedding planes will make the strain increment direction to become more horizontal and consequently slightly higher \( \alpha_{de} \) than \( \alpha \). At \( \alpha=45^\circ \), in which the plane of maximum shear stress was aligned to the horizontal plane, this also occurs but to a lesser extent. However, at \( \alpha=60^\circ \) and \( 80^\circ \), the strain increment direction moves over to the other side of the principal stress direction. These results are in a good agreement with the results of the laboratory tests reported by Miura et al. [66] and Cai et al. [72].

As shown in Figs 7 and 8, the initial response resulted from fabric anisotropy creates an initial deviation of major strain increment direction from the major stress direction. By increasing the shear strains, major strain increment direction aligns with the major stress
direction and the non-coaxiality value decreases. This is owing to the diverse role of elastic and plastic parts of the strain. Elastic component of strain is considerable at the early stages of loading and causes higher values of non-coaxiality. In higher shear strains, the contribution of elastic strains decreases and the plastic strains will be dominant, and consequently the non-coaxiality decreases. For cyclic tests, the initial response is highlighted for the first 7 or 8 cycles. In the monotonic tests, the effects of initial anisotropy drastically reduce after 0.5% to 1% octahedral shear strain. The outstanding fact is that the peak shear strength of these specimens occurs at 0.5%-1% shear strain. In other words, the effect of initial anisotropy on the sand response is mainly before the peak shear strength. When specimens approach failure, the sand behaves as an isotropic material. Cai et al. [72] and Rodriguez and Lade [10] observed similar initial responses and higher values of non-coaxiality before the peak shear strength.

In addition to the initial anisotropy, the stress anisotropy has significant effect on the value of non-coaxiality. For the tests with $\alpha=45^\circ$, where the specimens are subjected to torsional loading, the non-coaxiality was minimum (i.e., less than 6$^\circ$). In this test condition, the major principal stress is oriented such that the horizontal bedding plane (i.e., the weakest plane) direction is close to the direction of the maximum shear stress ratio, $q/\sigma_{0m}$. The largest deviations between $\alpha$ and $\alpha_{de}$ occurred in the tests with $\alpha=10^\circ$ and $\alpha=80^\circ$, where the specimens were respectively subjected to compression and extension loading. In these loading conditions the non-coaxiality increased more than 10$^\circ$ for both Babolsar and Toyoura sands. For a specific value of $\alpha$, the non-coaxiality values of Babolsar sand were slightly higher (i.e., these were about 1$^\circ$ -2$^\circ$) than the Toyoura sand. Apart from grain size, the morphological characteristics of these two type of sand particles are relatively similar. The behaviour could thus be associated with the coarser particles of Babolsar sand.
3.4. Effect of stress direction on the pore water pressure

Excess pore water pressure during cyclic loading has two components. The first is the transient excess pore water pressure, \( u_t \), which denotes the change in applied mean normal stress for saturated sand. Because the variation in the transient excess pore water pressure is associated with total mean normal stress, it has little influence on soil effective stress. The second component is residual excess pore water pressure, \( u_r \), which increases with the progressive plastic deformation of the soil skeleton. The residual excess pore water pressure is thus directly related to the shear strength and stiffness of soil [56].

The residual excess pore water pressure in one cycle of a stress-controlled test equals the excess pore water pressure at zero deviator stress. The pore water pressure is generally normalized using the initial effective stress, known as the excess pore water pressure ratio \( i.e. \, r_{ut}=u_t/\sigma'_0m \, \text{and} \, r_{ur}=u_r/\sigma'_0m \) [56].

Fig.9 shows the transient excess pore water pressures of Babolsar and Toyoura specimens tested at different principal stress directions. As it can be observed from this figure, the value of transient excess pore water pressure for \( \alpha=10^\circ \) and \( \alpha=80^\circ \) tests is slightly greater than the tests carried out at \( \alpha=45^\circ \). The transient excess pore water pressure resulted from the increase and decrease of loads in cycles. Stiffer soil skeleton results in higher transient pore water pressure. At the same time, lower shear strains will lead to lower residual pore water pressure. The residual excess pore water pressure generated in the cyclic tests of Babolsar and Toyoura sands is indicated in Fig.10. For both Babolsar and Toyoura sands, more residual excess pore pressure was observed when the principal stress directions are inclined at \( 30^\circ, \, 45^\circ \) and \( 60^\circ \) in which the specimens have lower cyclic resistance. Lower cyclic shear resistance will lead to development of higher plastic strain as well as residual excess pore water pressure. These results were in a good agreement with the result of HCA tests performed by Logeswaran [73] and Vipulanantham [74]. They observed that the lowest
cyclic resistance (i.e. minimum soil stiffness) occurs when \( \alpha \) is around 30° to 60° (i.e. the minimum value was observed at \( \alpha=45^\circ \)), in which the plane of maximum shear stresses are inclined to the bedding plane.

It was observed that for a given initial fabric and confining pressure, the transient and residual excess pore water pressure were influenced by the direction of the major principal stress. Furthermore, these effects could be amplified by the particles size. The effect of major principal stress on the residual excess pore water pressure generation was more noticeable in the Babolsar sand. Finer particles size, more uniformity and more roundness of Toyoura sand particles decrease the degree of fabric anisotropy and thus the direction of the major principal stress had little effect on its residual excess pore water pressure generation.

4. Conclusions

In order to investigate the effect of stress inclination on the monotonic and cyclic behaviour of dense sands, a series of undrained torsional shear tests were carried out on hollow cylinder specimens. Twenty monotonic and cyclic tests have been conducted on the dense Toyoura and Babolsar sands. The test were performed under controlled condition of the major principal stress direction, \( \alpha \), initial confining stress, \( \sigma'_0 \), and initial relative density, Dr. According to the results of the experiments, some of the main findings are presented as follows:

1. The direction of major principal stress, \( \alpha \), had no significant effect on the internal friction angle at failure (\( \varphi_f \)) or phase transformation (\( \varphi_{PT} \)) in both Babolsar and Toyoura sands (Fig. 6). The dense condition of the specimens caused the contribution of rearrangement (\( \varphi'_p \)) to decrease and the geometrical interference to be dominated by dilation (\( \varphi'_d \)), whereas the dilation angle tended to fall to zero at failure. Moreover, sand anisotropy is mainly affected by the initial fabric. An increase in strain destroyed
the particle interlock and the initial fabric. Thus, the degree of anisotropy decreased with an increase in strain, and the sand behaviour became more isotropic. At high levels of strain, the stress anisotropy had a minimum effect on the soil internal friction angle.

2. Although the specimen preparation method employed in this study (*i.e.* wet tamping technique) reduces the fabric anisotropy, but still the cross anisotropy resulted from the bedding plane made the strain increment direction to be greater than the direction of the major principal stress for specimens tested at \( \alpha < 45^\circ \). As it can be seen in Fig. 8, the plane of cross anisotropy had reverse effect on the specimens tested at \( \alpha = 45^\circ, 60^\circ, \) and \( 80^\circ \), while the major principal stress direction was greater than the strain increment direction.

3. The maximum non-coaxiality between the directions of the principal stress and principal strain increments occurred around the peak shear strength (*i.e.* \( \gamma_{oct} = 0.5\% - 1\% \)), when the main particles interlock began to be destroyed.

4. Babolsar and Toyoura sand particles have relatively similar morphological characteristics; however, the Babolsar sand had coarser particles that increased the degree of anisotropy. For a specific value of \( \alpha \), the non-coaxiality of the Babolsar sand was slightly greater (1\(^\circ\)-2\(^\circ\)) than that of the Toyoura sand.

5. For specimens tested at \( \alpha = 30^\circ, 45^\circ, \) and \( 60^\circ \), the direction of maximum shear stress was aligned with the weakest plane (*i.e.* horizontal plane), and therefore, the specimens showed softer responses to the loading. It resulted in higher residual excess pore water pressure and lower transient excess pore water pressure.

6. Apart from the morphological characteristics of the sand particles, particle size may have influenced by the soil anisotropic behaviour. The results showed that the major principal stress direction had more influence on the response of Babolsar specimens to
monotonic and cyclic loading. This may relate to the coarser particles in this type of sand.

5. References


19


[40] Shibuya, S., Hight, D. W., Jardine, R. J. “Local boundary surfaces of a loose sand


[70] Riemer, M. F. & Seed, R. B. “Factors affecting the apparent position of steady state


List of Figures

Fig. 1. Closed-loop control system of hollow cylinder apparatus
Fig. 2. Particle image of (a) Babolsar sand and (b) Toyoura sands
**Fig. 3.** Gradation curves of tested soils

<table>
<thead>
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<th>ASTM D422-63</th>
<th>Babolsar</th>
<th>Toyoura</th>
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<tr>
<td>$D_{10}$</td>
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<tr>
<td>$C_c$</td>
<td>1.32</td>
<td>0.95</td>
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- **Silt**
- **Sand**
  - Fine
  - Medium
  - Coarse
- **Gravel**
Fig. 4. Stress paths for: (a) monotonic and cyclic tests at $\tau_{\alpha\beta}$ and $(\sigma_z-\sigma_\theta)$ space; (b) monotonic loading; (c) cyclic loading
**Fig. 5.** Example of effective stress paths for monotonic and cyclic tests: (a) dense Babolsar; (b) dense Toyoura
Fig. 6. Stress paths of deviator stress with mean normal effective stress in Babolsar and Toyoura Sands.
Fig. 7. Directions of principal strain increments versus shear strain for monotonic torsional shear tests of: (a) Babolsar sand; (b) Toyoura sand
Fig. 8. Directions of principal strain increments versus cycle ratio for cyclic torsional shear tests of:

(a) Babolsar sand; (b) Toyoura sand
Fig. 9. Transient excess pore water pressure generated in cyclic tests of: (a) dense Babolsar sand; (b) dense Toyoura Sand.
Fig. 10. Residual excess pore water pressure generated in cyclic tests of: (a) dense Babolsar sand; (b) dense Toyoura Sand.
### Table 1. List of notation

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<th>Symbol</th>
<th>Physical quantity</th>
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<tr>
<td>( b )</td>
<td>intermediate principal stress parameter ( b=(\sigma_2-\sigma_3)/(\sigma_1-\sigma_3) )</td>
</tr>
<tr>
<td>( \alpha )</td>
<td>inclination of major principal stress to vertical</td>
</tr>
<tr>
<td>( \alpha_{dc} )</td>
<td>inclination of major principal strain increment to vertical</td>
</tr>
<tr>
<td>( H )</td>
<td>hollow cylinder specimen length</td>
</tr>
<tr>
<td>HCA</td>
<td>Hollow Cylinder Apparatus</td>
</tr>
<tr>
<td>( \sigma'_{0m} )</td>
<td>initial effective mean normal stress</td>
</tr>
<tr>
<td>( \zeta )</td>
<td>Non-coaxiality which is defined as non-coincidence between the principal stress and strain increment directions, ( \zeta=\alpha-\alpha_{dc} )</td>
</tr>
<tr>
<td>( q )</td>
<td>deviator stress</td>
</tr>
<tr>
<td>( \varepsilon_z, \varepsilon_\theta, \varepsilon_r )</td>
<td>Vertical, tangential and radial normal strains</td>
</tr>
<tr>
<td>( \varepsilon_1 )</td>
<td>Major principal strain</td>
</tr>
<tr>
<td>( \tau_{\theta z}, \tau_{z \theta} )</td>
<td>circumferential shear stresses</td>
</tr>
<tr>
<td>( \sigma_a, \sigma_r, \sigma_0 )</td>
<td>axial, radial and circumferential normal stresses</td>
</tr>
<tr>
<td>( \sigma_1, \sigma_2, \sigma_3 )</td>
<td>major, intermediate and minor total principal stresses</td>
</tr>
<tr>
<td>( \sigma'_1, \sigma'_2, \sigma'_3 )</td>
<td>effective major, intermediate and minor principal stresses</td>
</tr>
<tr>
<td>( p_i ) and ( p_o )</td>
<td>inner and outer confining pressures; respectively</td>
</tr>
<tr>
<td>( r_i ) and ( r_o )</td>
<td>inner and outer specimen radii; respectively</td>
</tr>
<tr>
<td>( F_v )</td>
<td>vertical load</td>
</tr>
<tr>
<td>( T )</td>
<td>torque applied to twist the hollow cylinder</td>
</tr>
<tr>
<td>( l_o ) and ( l_i )</td>
<td>changes in outside and inside radii; respectively</td>
</tr>
<tr>
<td>( \Delta V ) and ( \Delta V )</td>
<td>changes in volumes of the inner cell and the specimen; respectively</td>
</tr>
<tr>
<td>( \Delta H_{LVDT} )</td>
<td>recorded change in the horizontal LVDT reading</td>
</tr>
<tr>
<td>( r_{\text{measurement plate}} )</td>
<td>distance from the center to the radio wire cord of the pie-shaped measurement plate</td>
</tr>
<tr>
<td>( \varphi'_\mu )</td>
<td>interparticles sliding friction angle</td>
</tr>
<tr>
<td>( \varphi'_g )</td>
<td>geometrical interface friction angle</td>
</tr>
<tr>
<td>( \varphi'_d )</td>
<td>dilation angle</td>
</tr>
<tr>
<td>( \varphi'_p )</td>
<td>particle rearrangement or damage angle</td>
</tr>
<tr>
<td>( \varphi'_{\text{mob}} )</td>
<td>mobilized friction angle</td>
</tr>
<tr>
<td>( \varphi'_{f} )</td>
<td>friction angle at failure</td>
</tr>
<tr>
<td>( \varphi'_{\text{PT}} )</td>
<td>friction angle at phase transformation</td>
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Table 2. Physical properties of test materials

<table>
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<tr>
<th>Soil type</th>
<th>Specific gravity</th>
<th>$e_{\text{max}}$</th>
<th>$e_{\text{min}}$</th>
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<tr>
<td>Babolsar sand</td>
<td>2.75</td>
<td>0.79</td>
<td>0.53</td>
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<tr>
<td>Toyoura sand</td>
<td>2.65</td>
<td>0.97</td>
<td>0.61</td>
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<tr>
<td>Standard designation</td>
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<td>ASTM D4254-00</td>
<td>ASTM D4253-00</td>
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Table 3. Characterisation of the specimens and their loading condition

<table>
<thead>
<tr>
<th>No.</th>
<th>Sand Type</th>
<th>Loading condition</th>
<th>$\alpha$ (degree)</th>
<th>Dr (%)</th>
<th>$e$</th>
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<td>45</td>
<td>77</td>
<td>0.59</td>
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<td>Babolsar</td>
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<td>74</td>
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</tbody>
</table>
Fardin Jafarzadeh received his PhD degree in Civil Engineering from Tohoku University, Japan, in 1995, and is currently Associate Professor of Civil Engineering at Sharif University of Technology, Tehran, Iran. He is Vice-Chairman of the Iranian Geotechnical Society, and a member of ISSMGE, JSCE and several other professional associations. His research interests include a broad area of topics in Geotechnical and Geotechnical Earthquake Engineering with a special focus on the earth and rockfill dams, ground improvement, physical modeling and constitutive laws.

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