Laboratory and micromechanical investigation of soil anisotropy

L.-T. Yang i), D. Wanatowski ii), X. Li iii), H.-S. Yu iv) and Y. Cai v)

i) Research Associate, Nottingham Centre for Geomechanics, University of Nottingham, Nottingham, United Kingdom.
ii) Associate Professor, Department of Civil Engineering, University of Nottingham, Ningbo, China.
iii) Lecturer, Nottingham Centre for Geomechanics, University of Nottingham, Nottingham, United Kingdom.
iv) Professor, Nottingham Centre for Geomechanics, University of Nottingham, Nottingham, United Kingdom.
v) Lecturer, Department of Civil Engineering, Huqiao University, Xiamen, China.

ABSTRACT

This paper presents an experimental investigation revisiting the anisotropic behavior of geomaterials in drained monotonic shear using Hollow Cylinder Apparatus. The test program has been designed to cover the effect of material anisotropy, pre-shearing and material density on the behavior of Leighton Buzzard sand. Experiments have also been performed on glass beads to understand the effect of particle shape. The test results demonstrate that the stress-strain-strength behavior of the specimen shows strong dependence on the principal stress direction. The pre-shearing history, material density and particle shape are found to be influential. The deformation non-coaxiality, i.e., non-coincidence between the principal stress direction and the principal strain rate direction is observed. In particular, it was found that non-coaxiality is more significant in presheared specimens. This paper also explains the phenomenological observations based on the recently acquired understanding in micromechanics, with attention focused on strength and deformation non-coaxiality.

Keywords: anisotropy, laboratory tests, micromechanical, sand (soil type)

1 INTRODUCTION

Shear strength is a fundamental soil property on which many geotechnical engineering design is based, and thus it must be determined with reasonable accuracy. However, the stress-strain-strength behavior of most sedimentary deposits is anisotropic. Soil strength is generally lower when the loading is further away from the deposition direction. Hence, soil anisotropy has attracted long-lasting interest of researchers and practitioners (e.g. Arthur and Menzies 1972; Oda et al. 1978; Miura and Toki 1984; Gutierrez et al. 1991; Cai et al. 2013).

Extensive phenomenological observations on soil strength and loading path dependence have been made. Clear evidence of material deformation non-coaxiality, an interesting phenomenon firstly reported by Roscoe et al. (1967) has been obtained. During the last few decades, researchers have been exploring the micromechanics of soil anisotropy through multi-scale investigations (e.g. Oda et al. 1985; Rothenburg and Bathurst 1989; Takemura et al. 2007). It is now generally recognized that the material anisotropy is originated from particle scale as a consequence of particle spatial arrangement, also known as the internal structure. More recently, Li and Yu (2009) presented 2D DEM simulation results that gave insight into strength anisotropy and deformation non-coaxiality. Li and Yu (2013) explained the micromechanics with the aid of the established Stress-Force-Fabric (SSF) relationship of granular materials (Li and Yu 2014). The material strength and the degree of non-coaxiality were determined analytically in terms of fabric tensor characterizing the anisotropy of material internal structure and contact force distributions characterizing the anisotropy of particle interactions (Li and Yu 2013).

However, it is worth pointing out that laboratory testing remains an irreplaceable approach for studying fundamental behavior of real geomaterials. Idealization of particle shapes, limitation of sample size and use of simple contact models are often inevitable shortcomings in multi-scale studies. Micromechanically established theories have to be carefully validated by laboratory testing before applying to problems involving real geomaterials. Based on this context, a comprehensive experimental investigation has been carried out in this study by means of HCA to revisit the anisotropic behavior of geomaterials. In addition to the phenomenological observations, attention has been placed on applying more recently acquired micromechanical theories to understanding the strength anisotropy and deformation non-coaxiality observed in real geomaterials.
2 TESTING ARRANGEMENT AND PROCEDURE

2.1 Hollow cylinder apparatus

In this study, the hollow cylinder apparatus, developed by GDS Instruments Ltd, was used. For the details of the testing system, see Yang (2013). The cell contains the hollow cylindrical specimen with inner radius of 30 mm, outer radius of 50 mm and height of 200 mm. In monotonic shear, the application of axial load \( W \), torque \( M_r \), inner cell pressure \( p_i \) and outer cell pressure \( p_o \) enables the control of four stress components, axial stress \( \sigma_z \), radial stress \( \sigma_r \), circumferential stress \( \sigma_{\theta \theta} \) and shear stress \( \sigma_{\theta z} \), on an element in the wall of the hollow cylindrical specimen. The radial strain \( \varepsilon_r \), circumferential strain \( \varepsilon_\theta \) and shear strain \( \gamma_{\theta \theta} \) were measured indirectly from the changes of inner and outer radii of the specimen. The radius changes were computed from the changes of the volume in the inner chamber and the specimen measured by the two digital pressure/volume controllers. The stresses and strains are calculated following the formulations of Hight et al. (1983).

2.2 Tested material and sample preparation

Leighton Buzzard (Fraction B) sand and Ballotini glass beads were tested in this study. Leighton Buzzard sand is standard sand consisting mainly of sub-rounded quartz particles with some carbonate materials. The Ballotini glass beads are made of high quality pure soda-lime glass. The index properties of the two materials are summarized in Table 1.

| Property               | Leighton Buzzard sand | Ballotini glass beads |
|------------------------|-----------------------|-----------------------|
| Mean grain size \( D_{50} \): mm | 0.62                  | 1.35                  |
| Effective grain size \( D_{10} \): mm | 0.45                  | 1.15                  |
| Uniformity coefficient: \( D_{50}/D_{10} \) | 1.56                  | 1.18                  |
| Specific gravity \( G_s \) | 2.65                  | 2.50                  |
| Minimum void ratio \( e_{\min} \) | 0.52                  | 0.52                  |
| Maximum void ratio \( e_{\max} \) | 0.79                  | 0.68                  |

The water sedimentation method was used to prepare all the samples. After saturation, with Skempton’s B value greater than 0.96, specimens were consolidated isotropically under an effective confining stress of \( p' = 200 \) kPa.

2.3 Experimental program

Each series of drained monotonic shear tests with various loading directions carried out in this study are summarized in Table 2. The first two series of tests were performed on dense and medium dense Leighton Buzzard sand in order to generate a basic understanding of the anisotropic behavior of granular geomaterials. The third series of tests were performed on presheared sand specimens in order to investigate the impacts of preshearing on the response of sand to subsequent loading. In this series of tests, a presheared specimen was obtained by shearing the isotropically consolidated specimen in the vertical direction \((\alpha = 0^\circ)\) up to the peak and unloading it to a stress state with deviatoric stress \( q = 20 \) kPa. The fourth series of tests was performed on glass beads in order to study the effect of particle shape on the behavior of granular materials.

Table 2. Initial conditions for monotonic shear tests.

| Description | Test ID | \( D_e (%) \) | \( b \) | \( \alpha (^\circ) \) |
|-------------|---------|---------------|------|-----------------|
| Dense sand  | LBD     | 76            | 0.5  | 0, 15, 30, 60, 75, 90 |
| Medium sand | LBM     | 43            | 0.5  | 0, 15, 30, 60, 75, 90 |
| Presheared sand | LBD-PL | 73            | 0.5  | 0, 15, 30, 60, 75, 90 |
| Glass beads | GBD     | 90            | 0.5  | 0, 15, 30, 60, 75, 90 |

\( D_{ec} \): relative density after consolidation, \( a \): principal stress direction, \( b \): intermediate principal stress parameter.

Fig. 1 illustrates the stress paths in the \( X-Y \) stress space for monotonic loading tests with different inclinations of the major principal stress \((\alpha = 0^\circ, 15^\circ, 30^\circ, 60^\circ, 75^\circ, \) and \( 90^\circ)\). During the test, monotonic loading was applied in HCA strain-controlled mode in terms of the axial displacement under a drained condition. To ensure full discharge of water from the specimen, the axial strain was increased at a slow rate of 0.05%/min. In all the tests the magnitudes of the mean effective stress \( p' \) and the intermediate principal stress parameter \( b \) were maintained constant. It should also be pointed out that since the calculations of stresses and strains in HCA testing are based on global measuring system, the post-peak stress-strain curves could be subject to considerable error due to severe changes in sample thickness and curvature along the sample height. Nonetheless, the post-peak stress-strain behavior remains very useful for qualitative assessment of soil behavior and thus is included in all the plots.

3 RESULTS AND DISCUSSION

3.1 Material anisotropy

3.1.1 Stress-strain behavior

The first series of tests, performed on dense Leighton Buzzard sand, is shown in Fig. 2(a). For volumetric strain on the figures, a positive value along the vertical axis indicates contraction and the negative indicates dilation. The effect of anisotropy produced...
during sample preparation is apparent in both stress ratio and volumetric strain responses. The shear strength reduces and the volumetric compressibility increases with increasing values of $\alpha$. The highest peak was obtained when the major principal stress direction was vertical and it was reduced dramatically as the direction of the major principal stress was changed from $\alpha = 30^\circ$ to $\alpha = 60^\circ$. It can be seen from Fig. 2(b) that preshearing history to the peak stress has a significant effect on the subsequent stress-strain response of sand. Generally, a softer response in the stress-strain relationship, larger initial contraction and larger strain to reach the peak stress ratio were observed at each loading direction for presheared specimens.

Fig. 2. Stress-strain behaviour at different loading directions for: (a) dense sand; (b) presheared sand.

3.1.2 Strength anisotropy

The peak stress ratios $\eta_p$ at different major principal stress direction $\alpha$ obtained from test series LBD and LBD-PL are compared in Fig. 3. It can be observed that the variation of the peak stress ratios with principal stress direction show similar trend patterns for the two series of tests. The highest peak stress ratio was obtained when the major principal stress direction was parallel to the deposition direction (i.e. $\alpha = 0^\circ$) and the lowest value was obtained at $\alpha = 60^\circ$. The specimen strength reverted slightly from $\alpha = 60^\circ$ to $90^\circ$.

Using a 2D discrete element code PFC2D, Li and Yu (2009) prepared and tested anisotropic specimens consisting of non-spherical particles under monotonic loading with different fixed strain increment directions. The pre-failure stress ratio (corresponding to 2% of axial strain) with different loading directions obtained from the initially anisotropic samples and preloaded samples were analyzed thoroughly by Li and Yu (2009). A qualitative analysis of the DEM simulations and the HCA test results from the current study shows a similar variation of the pre-failure stress ratios with different loading directions. Based on the established Stress-Force-Fabric (SFF) relationship (Rothenburg and Bathurst 1989, Li and Yu 2014), Li and Yu (2014) explained when material approaches the peak stress ratio, the direction of the force anisotropy and fabric anisotropy were generally coaxial with loading direction. Therefore, the magnitude of peak stress ratio was dependent on the developed degrees of force anisotropy and fabric anisotropy. As the loading directions varied from $\alpha = 0^\circ$ to $60^\circ$, the force anisotropy and fabric anisotropy decreased, leading to decreasing stress ratio. Upon further increase of $\alpha$ to $90^\circ$, the fabric anisotropy decreased continuously while the force anisotropy increased, resulting in slight increase of stress ratio. The micromechanical explanation is supported by the experimental data with the qualitative agreement with DEM simulation results.

3.1.3 Deformation non-coaxiality

The numerical study carried out by Li and Yu (2009) shows that a preloading history may have significant effects on the anisotropic behavior of granular materials to subsequent loading. Although deformation non-coaxiality is negligible for initially anisotropic samples, it could be significant once the samples have been presheared. The major directions of stress and strain increment obtained from test series LBD and LBD-PL are plotted against the stress ratio in Fig. 4(a) and 4(b), respectively. During shearing, the direction of major principal stress $\alpha$ was fixed as indicated by solid lines. The calculated strain increment
directions are indicated by dashed lines with open circle symbols. The results obtained from test series LBD and LBD-PL provide confirmation of the observations made by Li and Yu (2009). The degree of non-coaxiality observed in tests conducted on presheared specimens is significantly different from that obtained from non-presheared specimens. Significant non-coincidence between the stress and strain increment directions was observed at $\alpha = 15^\circ$, $30^\circ$, $60^\circ$ and $75^\circ$ for presheared samples. The deviations were especially large when $\alpha$ was equal to $30^\circ$ and $60^\circ$, where the degree of non-coaxiality reached about $22^\circ$ in both cases.

Based on the SFF relationship, Li and Yu (2013) reported that non-coaxiality was quantitatively dependent on the relative direction, as well as the relative magnitude of the fabric anisotropy (Fig. 5(a)) and the contact force anisotropy (Fig. 5(b)). As the direction of force anisotropy is almost coaxial with the loading direction during shearing, non-coaxiality is hence the result of the principal directions of fabric anisotropy deviating from the loading direction. Microscopically, it was found that for simulation with loading direction parallel to the deposition direction ($\alpha = 90^\circ$), the principal direction of fabric anisotropy was coincident with loading direction throughout the shearing. For test with loading direction perpendicular to the preloading direction ($\alpha = 0^\circ$), the principal directions of fabric anisotropy quickly approached to the loading direction at the initial stage of shearing. Hence, the material behaved almost coaxially when the samples were loaded in the direction of major principal stress parallel or perpendicular to the deposition direction. However, for simulations with loading direction fixed at $\alpha = 75^\circ$, $60^\circ$, $45^\circ$, $30^\circ$ and $15^\circ$ as shown in Fig. 5(a), the principal directions of fabric anisotropy were gradually rotated in such a manner that they finally pointed in the loading direction at large strain levels, thus it was observed that non-coaxiality degree decreased with increasing stress ratio and material were nearly coaxial close to failure. As for the presheared specimens, the magnitude of fabric anisotropy was found to be larger than the initially anisotropic sample prepared by deposition. Accordingly, more significant deformation non-coaxiality was observed. Therefore, DEM simulations reported by Li and Yu (2013) provide a plausible explanation for the observations on Leighton Buzzard sand, shown in Fig 4.

Fig. 4. Stress and strain increment directions at different loading directions for: (a) dense sand; (b) presheared sand.

Fig. 5. Principal directions of: (a) fabric anisotropy and (b) contact force anisotropy during monotonic shear in the initially anisotropic sample (after Li and Yu 2009).
3.3 Effects of material density and particle shape

Fig. 6 presents the comparison of the stress-strain curves obtained at three representative loading directions between LBD and LBM. For a comparison purpose, the results from LBM are plotted with solid lines while the corresponding results of LBD are shown with dashed lines. It can be observed that regardless of the loading direction, the medium dense sand tends to exhibit lower shear strength and more contractive volumetric strain than those of the dense sand. Moreover, larger deviatoric strain was required for the medium dense sand to reach the peak state.

The comparison of the LBD and GBD results is shown in Fig. 7 with solid lines representing GBD and dashed lines representing LBD. It can be observed that the glass beads tend to have softer stress-strain response, lower shear strength and larger volume compressibility, even though the beads have higher relative density than the sand. It is also obvious that spherical glass beads exhibit more severe fluctuations in the stress-strain curves than angular sands (also known as stick-slip phenomenon, Adjémian (2005)).

Comparison of the peak stress ratios obtained at different loading directions from the test series LBD, LBM and GBD are shown in Fig 8. It can be observed that the general trend of the variation of peak stress ratios with increasing values of \( \alpha \) is similar for the three series of tests. However, the results obtained from test series LBD and LBM show that the maximum difference between the peak stress ratios obtained from both series is only 0.06 and it was obtained at \( \alpha = 0^\circ \), even though the difference between the relative densities of the two samples is about 34%. The difference between results obtained from test series LBD and GBD, was more significant. As shown in Fig. 8, a large reduction in the material strength was observed when the angular sand was changed to the spherical glass beads even though the relative density of the glass beads was 14% higher than the dense sand (see Table 2).

Fig. 9 shows the comparison of the calculated strain increment directions obtained at different loading directions from test series LBD, LBM and GBD. It can be seen that the magnitudes of the directions of strain increments were very similar at each loading direction for test series LBD and LBM. Therefore, the experimental results suggest that the effect of relative density on the non-coaxial behavior of sand in monotonic shear is not significant. As indicated in Fig. 9, despite the slightly smaller degree of non-coaxiality in test series GBD, the margin by which the non-coaxiality of LBD exceeded that of the GBD was limited to 3°. Hence, the effect of particle shape on the non-coaxial behavior of sand in monotonic shear is also not significant.
4 CONCLUSIONS

This paper presents an experimental investigation revisiting anisotropic stress-strain-strength behavior of geomaterials in drained monotonic shear using Hollow Cylinder Apparatus. The test program has been designed to cover the effect of material anisotropy, preshearing and material density on the behavior of Leighton Buzzard sand. Experiments have also been performed on glass beads to understand the effect of particle shape. An attempt has also been made to explain the phenomenological observations of strength anisotropy and deformation non-coaxiality based on the recently acquired understanding in micromechanics. The major findings and conclusions can be summarized as follows:

- The effect of anisotropy produced during sample preparation is apparent in both deviatoric strain and volumetric strain responses of sand. The sand specimens subjected to preshearing history to the peak stress were found to be softer and contracted more in the subsequent responses. For a given loading direction, the peak shear strength is relatively unaffected by preloading to the peak stress. However, the preshearing history does have a significant effect on the non-coaxiality of sand specimens.

- The observations shown that lower relative density and rounder particle shape of the assembly of granular materials tend to produce softer response, severer initial contraction and lower shear strength in monotonic shear. However, the effects of the particle shape and relative density on the non-coaxial behavior of granular materials under monotonic shear were found to be less significant.

- The phenomenological observations of strength anisotropy and deformation non-coaxiality was explained by recently acquired micromechanical theories. Based on the established Stress-Force-Fabric relationship, the strength anisotropy of granular materials was mainly due to the differences in the variation of the degrees of fabric anisotropy and force anisotropy at different loading directions. The degree of non-coaxiality was dependent on the relative direction, as well as the relative magnitude of the fabric anisotropy and the contact force anisotropy. As in monotonic shearing, the direction of force anisotropy is coaxial with the loading direction. The deformation non-coaxiality is hence the result of the principal directions of fabric anisotropy being deviated from the loading direction.

REFERENCES

1) Arthur, R. F. and Menzies, B. K. (1972): Inherent anisotropy in sand. Geotechnique, 22(1), 115-131.
2) Adjémian, F. and Evesque P. (2005): Experimental stick-slip behaviour in triaxial test on granular matter. Powders and Grains, 12, 115-121.
3) Cai, Y. Y., Yu, H. S., Wanatowski, D., and Li, X. (2013): Non-coaxial behavior of sand under various stress paths. J. Geotech. Geoenviron. Eng., 139(8), 1381–1395.
4) Gutierrez, M., Ishihara, K. and Towhata, I. (1991): Flow theory for sand during rotation of principal stress direction. Soils and Foundations, 31(4), 121–132.
5) Hight, D. W., Gens, A., Symes, M. J. (1983): The development of a new hollow cylinder apparatus for investigating the effects of principal stress rotation in soil. Geotechnique, 33(4), 355-383.
6) Li, X. and Yu, H. S. (2009): Influence of loading direction on the behaviour of anisotropic granular materials. International Journal of Engineering Science, 47, 1284-1296.
7) Li, X. and Yu, H. S. (2013): Particle scale insight into deformation non-coaxiality of granular materials. International Journal of Geomechanics, (doi: 10.1061/(ASCE)GM.1943-5622.0000338).
8) Li, X. and Yu, H. S. (2014): Fabric, force and strength Anisotropies in granular materials: A micromechanical insight. Acta Mechanica, 225(8), 2345-2362.
9) Miura, S. and Toki, S. (1984): Anisotropy in mechanical properties and its simulation of sands sampled from natural deposits. Soils and Foundations, 24(3), 69-84.
10) Oda, M., Isao, K. and Toshio, H. (1978): Experimental study of anisotropic shear strength of sand by plane strain test. Soils and Foundations, 18(1), 25-38.
11) Oda, M., Nemat-Nasser, S., and Konishi, J. (1985): Stress-induced anisotropy in granular masses. Soils and Foundations, 25(3), 85-97.
12) Roscoe, K. H., Bassett, R. H., and Cole, E. R. L. (1967): Principal axes observed during simple shear of a sand. Proc. 4th Eur. Conf. Soil Mech. Found. Eng. Oslo, 231-237.
13) Rothenburg, L. and Bathurst, R. J. (1989): Analytical study of induced anisotropy in idealized granular materials. Géotechnique, 39(4), 601-614.
14) Takemura, T., Takahashi, M., Oda, M., Hirai, H., Murakoshi, A. and Miura, M. (2007): Three-dimensional fabric analysis for anisotropic material using multi-directional scanning line—application to X-ray CT image. Materials Transactions, 48(6), 1173-1178.
15) Yang, L. T. (2013): Experimental study of soil anisotropy using Hollow Cylinder testing. Doctoral dissertation, University of Nottingham, United Kingdom.