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The Strength Anisotropy of a Residual Soil in Singapore

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ABSTRACT: The undrained shear strength of soil is one of the most important parameters required for geotechnical design. Depending on the design situations, the undrained shear strength of soil may have to be determined by different tests. In this paper, some testing data on the determination of the undrained shear strength of a residual soil in Singapore are presented. Large blocks of undisturbed residual soil samples were taken from a construction site of the Deep Tunnel Sewerage System Project in Singapore. To study the inherent strength anisotropy, specimens cut in both vertical and horizontal directions were tests. The anisotropy behaviour of intact residual soil was also investigated under different major principal stress directions. The tests conducted included K_0 consolidated undrained triaxial compression (CK_0UC) and extension (CK_0UE) tests, and K_0 consolidated undrained direct simple shear (CK_0UDSS) tests. Based on the experimental results, the c_u/σ_{10}' versus OCR relationships of each type of tests are established for practical applications. The failure envelopes and friction angles determined from different types of tests are also compared.

1. INTRODUCTION

Residual soils are widely distributed in tropical regions, which are formed in place by the following processes, e.g. incorporation of humus (decaying vegetation), physical and chemical weathering, leaching of insoluble materials, accumulation of insoluble residues, downward movement of fine particles (lessivage) and disturbance by root penetrations, animal burrowing, free fall and desiccation (Fookes, 1997). In tropical regions, weathering of the parent rocks of residual soils is more intense and occurs to greater depths than elsewhere. This is led by the tropical climatic conditions, such as high temperatures and precipitation. The engineering behaviour of residual soils is significantly influenced by the progress of weathering process, the presence of bonded structures and fabric, and also by the depth of penetration of which will be influenced by the discontinuities in the parent rock (Hight & Leroueil, 2002; Viana da Fonseca, 2003).

Residual soil, as a common but important material in tropical regions, has been studied intensively for decades. Researches were carried out to characterize the engineering properties of residual soils (Winn et al., 2001; Viana da Fonseca, 2003; Hight & Leroueil, 2002; Rahardjo et al., 2004; Zhang et al., 2007), to study the weathering effects during soil formation (Vaughan & Kwan, 1984), to index the engineering properties of residual soils (Vaughan et al., 1988), to investigate the effects of soil structure on the engineering properties (Leroueil & Vaughan, 1990; Nagaraj et al., 1998), to analyze the compressibility behaviour (Nagaraj et al., 1998) and also to evaluate the collapse behaviour of it (Rao & Revanasiddappa, 2006; Huat et al., 2008). It should be pointed out that the majority of laboratory studies on residual soils were carried out on small scale of intact residual soils or compacted residual soils, and tested under triaxial or oedometer conditions. The studies on strength anisotropy of large blocks intact residual soils were seldom. Relevant investigation is worth to be carried out.

Anisotropy refers to the variation in material properties with direction. When soil deposits, its particles have strong tendency to align themselves in a direction normal to the direction of deposition. Therefore, natural clays exhibit significantly different structure and strengths in the vertical and horizontal directions.

Civil engineering construction almost invariably involves changes to the stress state in the ground, resulting in movements and requiring stability under the new stress regime to be considered (Leroueil & Hight, 2002). The construction process of earth structures will also lead to principal stress rotation.

Figure 1 shows an example of stress changes beneath a vertically loaded circular footing (Leroueil & Hight, 2002). The stress state changes in terms of deviator stress are presented in Figure 1(a), as contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p$ associated with a pressure, p applied on the circular footing. $\Delta\sigma_1$ and $\Delta\sigma_3$ are the changes in major and minor principal stresses respectively. The changes of stresses beneath a circular footing could also be presented in terms of major principal stress direction or changes in the relative magnitude of the intermediate principal stress (Δb) (as shown in Figure 1(b) and (c)). The direction of major principal stress was defined as an angle, α to the vertical, and the Δb value is calculated by Leroueil & Hight (2002) as:

$$\Delta b = \frac{\Delta\sigma_2 - \Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \qquad (1)$$

It is clearly observed that both magnitude and direction of principal stresses were affected due to the loading imposed on circular footing.

Another example is given in Figure 2 for the stress conditions along failure surface beneath embankment. If the soil is anisotropic, the available undrained strength will vary along the potential failure surface. The c_u changes with σ_1 along the slip surface as shown in Figure 2 (Bjerrum & Aitchison, 1973; Graham, 1979; Zdravkovic et al, 2002). The rotation of principal stresses was also obtained in excavation project (Figure 3) (after Clough & Hansen, 1981).

The effects of the principal stress rotation can be attributed to the anisotropy of soils, including inherent anisotropy and induced anisotropy. The inherent anisotropy refers a physical characteristic inherent in the material and entirely independent of the applied stress, whereas the induced anisotropy refers a physical characteristic due exclusively to the strain associated with applied stress (Casagrande & Carrillo, 1944).

Since sedimentary soils are inherently anisotropic, their response to loading will depend on the directions of principal stresses in relation to the deposition direction. Rotation of the principal axes of stress occurs in practice under most types of loading. Even the loading imposed by the simplest foundation induces a rotation of axes at all points, except under the center of the foundation (or under the centerline if it is a strip footing) (Joer et al, 1998).

For residual soils, the inherent anisotropy is due to the development of an anisotropic macrostructure and microstructure after deposition of particulates through air or water, when compacted, which is a product of weathering processes (Bica et. al., 2008). Inherent anisotropy of the soil is resulting from the structure of the material, i.e. its fabric and bonding at all levels. Whatever the soil condition, microstructured or not, induced anisotropy derives mainly from volumetric strains imposed on the soil during shearing (Leroueil & Hight, 2002). When the microstructure is preserved during loading, inherent anisotropy is dominant (Bica et. al., 2008). Anisotropy at small strain bears no relation to anisotropy at large strains, since anisotropy induced by strains can modify inheren anisotropy (Hight, 2001).

As for induced anisotropy, Broms and Casbarian (1965) proposed that its effects on clay may be attributed to the reorientation of the individual clay particles. It has been observed (Lambe, 1958; Hvorslev, 1960) that flat clay particles have a tendency to orient themselves perpendicular to the direction of the major principal stress. The tendency of the individual clay particles to align themselves parallel with the final failure plane will increase with increasing rotation of the principal stress directions.

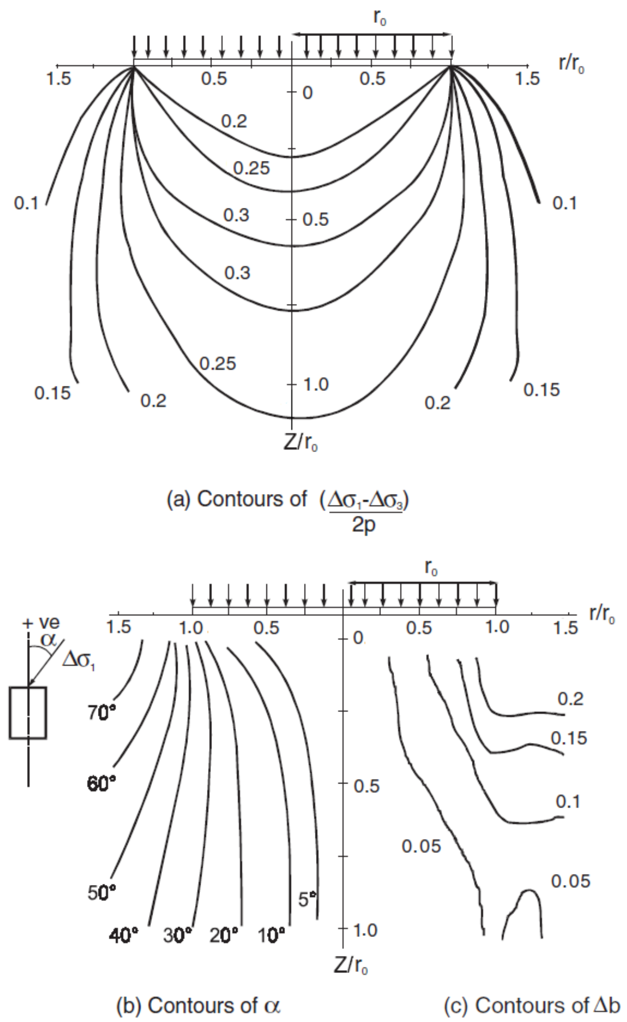

Figure 1. Stress Changes Beneath Vertically Loaded Circular Footing on Isotropic Linear Elastic Foundation (ν = 0.45): (a) Contours of $(\Delta\sigma_1 - \Delta\sigma_3)/2p$; (b) Contours of α; (c) Contours of Δb (after Leroueil & Hight, 2002)

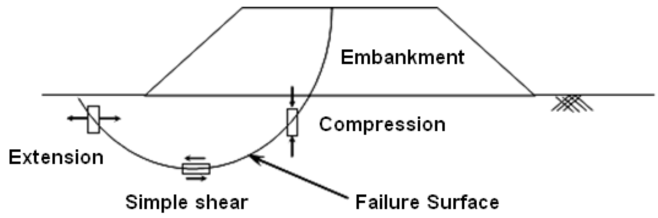

Figure 2. Laboratory Experiments that Resemble Stress Conditions along the Failure Surface beneath an Embankment (after Bjerrum & Aitchison, 1973)

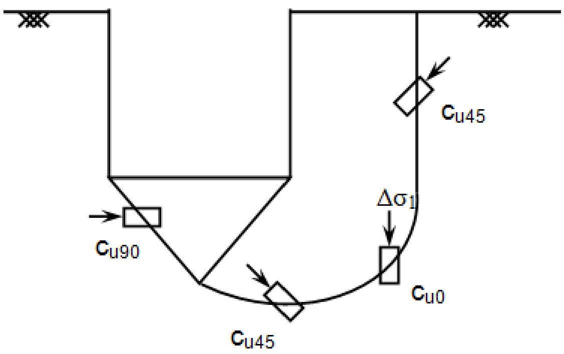

Figure 3. Rotations of Principal Stresses in Excavation Project (After Clough & Hansen, 1981)

2. MATERIAL AND TESTING SYSTEMS

In relation to a tunnelling construction project in Singapore, the strength anisotropy of an intact residual soil was investigated through laboratory tests. Large blocks of intact residual soil samples were taken from a construction site of the Deep Tunnel Sewerage System project in Singapore. A series of tests including K_0 consolidated undrained triaxial compression (CK_0UC) tests, K_0 consolidated undrained triaxial extension (CK_0UE) tests and K_0 consolidated undrained direct simple shear (CK_0UDSS) tests were conducted to investigate the induced strength anisotropy behaviour. The inherent strength anisotropy was studied by conducting laboratory tests on intact residual soils cut in different directions (as shown in Figure 4). Part of the testing data was reported by Meng et al. (2007), Meng et al. (2008) and Meng & Chu (2011).

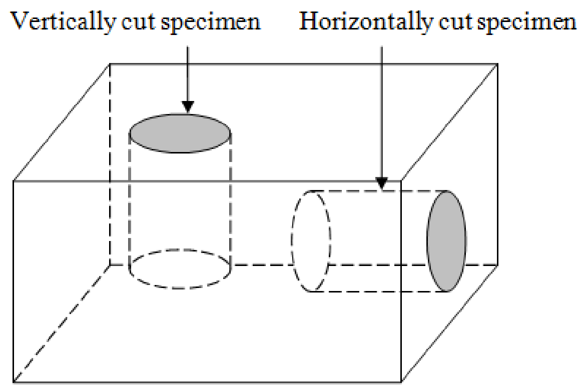

Figure 4. Cutting Orientation of Undisturbed Soil Specimen

The Bukit Timah Granite, as one of the oldest formations in Singapore, widely distributed in the central and northern parts of Singapore Island. Large blocks of soil sample were retrieved 19.4m from the residual soil layer that belongs to the formation of weathered Bukit Timah granite by carefully cutting the soil using a cutter. This soil layer belongs to completed weathered “fine grained soil”, as indicated by Winn et al. (2001). The degree of weathering of the granite decreases with depth. Field observations indicate that the weathering of the Bukit Timah Granite has been rapid and is primarily due to the chemical decomposition under the humid tropical climate of Singapore.

The ground water table was at 2.1 m below the ground level. The in-situ effective stress was estimated to be 195kPa, in both vertical and horizontal direction. A pre-consolidation pressure (σ_p') of 300kPa was determined from the K_0 consolidation tests using a triaxial cell (Meng & Chu, 2011). It should be noted that the term "pre-consolidation stress" was used in the generalized term to mark a yielding point. The basic soil properties are listed in Table 1. The grain size distribution curve is presented in Figure 5. The microstructure of intact residual soil of Bukit Timah granite was studied with the aid of a scanning electron microscope (SEM). An example of SEM images was given in Figure 6. With the scale of 2.0μm provided by SEM, laminated micro-structure was observed.

The testing systems have been described in detail in Meng & Chu (2011). For triaxial tests (CK_0UC & CK_0UE tests), the specimen was anisotropically (K_0) consolidated under the condition of $d\varepsilon_v/d\varepsilon_1 = 1$ by increasing the vertical load gradually. This technique was proposed by Chu (1991) and Lo & Chu (1991).

Table 1. Basic Properties of Intact Residual Soil of Bukit Timah Granite

Soil Properties	Value
Bulk Density	1.8-1.93 Mg/m^3
Liquid Limit	43 %
Plastic Limit	24.51 %
Plastic Index	18.35 %
Fines Content	50%
Specific Gravity	2.693
Water Content	31%
In-situ OCR	1.54
Permeability	10^{-9} m/s
SPT N Value	31

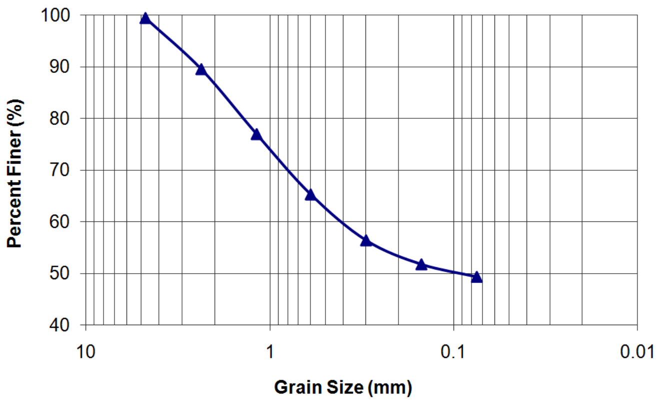

Figure 5. Grain Size Distribution Curve

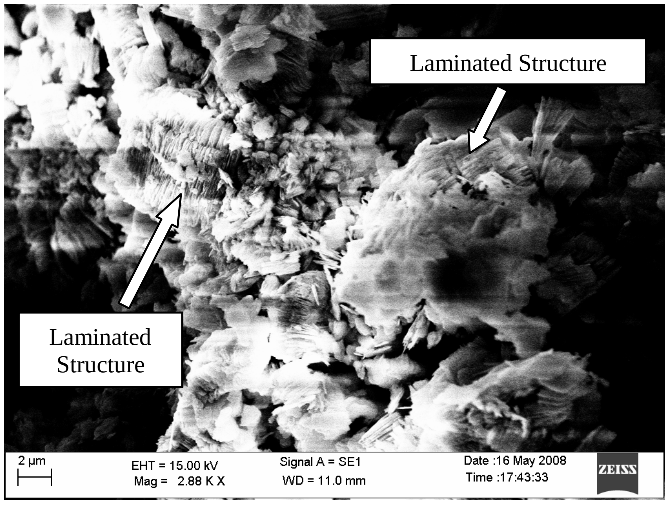

Figure 6. SEM Image of Intact Residual Soil of Bukit Timah Granite

3. TEST RESULTS

3.1 CK_0UC tests

The CK_0UC tests conducted on both vertically cut and horizontally cut intact residual soil specimens are presented in Figure 7 & Figure 8. In all the tests, the specimens were K_0 consolidated before being sheared undrained. To achieve a normally consolidated (NC) state, the specimen was consolidated to a certain value of effective vertical stress, σ_1', which was far away from the estimated pre-consolidation pressure (e.g. 900kPa for specimen C-V1). To achieve a certain overconsolidation ratio (OCR), such as OCR = 2 in test C-V3, the specimen was consolidated to an effective vertical stress (σ_1') of 390kPa, which was far beyond the pre-consolidation stress (σ_p' = 300kPa), and then unloaded to 195kPa (in-situ effective vertical stress). The same procedure was adopted for the specimen with OCR value of 4 (C-V4). The specimen was consolidated to 780kPa and then unloaded to 195kPa. It should be mentioned that, specimen C-V2 was consolidated directly to the in-situ effective vertical stress of 195kPa, which was lower than σ_p' (300kPa). The OCR value of specimen C-V2 was calculated as $OCR = \sigma_p'/\sigma_{10}' = 300/195 = 1.54$. The stress path followed by K_0 consolidation of specimen C-V2 is shown in Figure 7(a) for reference. The effective vertical stress at end of K_0 consolidation (e.g. at beginning of shearing), σ_{10}' are indicated on stress-strain curves.

Figure 7(a) and (b) present the effective stress paths and stress-strain curves determined from CK_0UC tests conducted on vertically cut intact residual soil specimens. The failure envelope of the vertically cut intact residual soil is shown in Figure 7(a). It was determined as an envelope to all the effective stress paths, and passes through the points of peak effective stress ratio, $(q/p')_{peak}$. It is straight within the tested stress range, with a slope of 1.2. It is assumed that the true cohesion of the intact residual soil is small enough to be ignored, since the intact residual soil disintegrated immediately after soaking into water. Thus, the failure envelope within the low stress range is assumed to pass through the origin, as shown in Figure 7(a).

The stress-strain behaviour of the vertically cut intact residual soil specimens determined from CK_0UC tests is shown in Figure 7(b). It is observed that the peak deviator stress (q_{peak}) was attained at a very small axial strain (1-2%) for all the tests, and the peak indicated the failure of the specimens. A significant amount of strain softening was noted after the q_{peak} for NC clay. For OC specimens, the q_{peak} increased with OCR.

The horizontally cut intact residual soil samples behaved similarly to the vertically cut samples, as presented in Figure 8. A failure envelope with a slope of 1.2 was also determined for horizontally cut specimens. q_{peak} was attained at a very small axial strain, which increased with OCR for specimens consolidated to the same stress level.

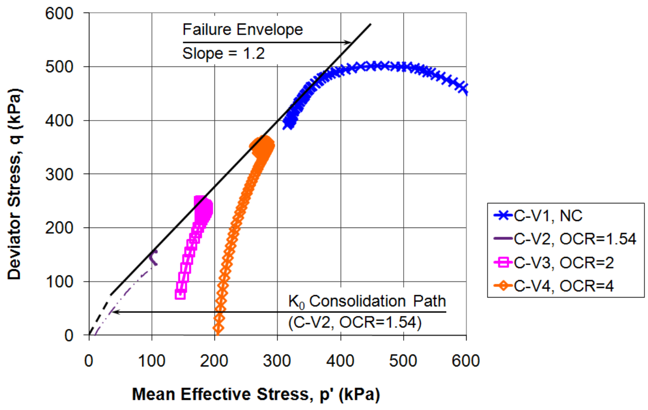

Figure 7 (a)

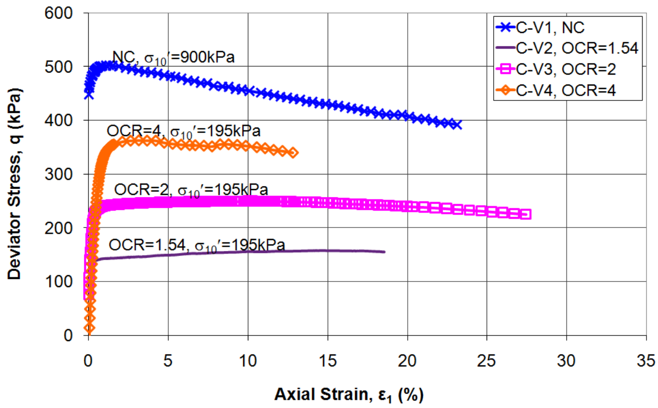

Figure 7 (b)

Figure 7. Undrained Behaviour of Vertically Cut Intact Residual Soil during CK$_0$UC Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves

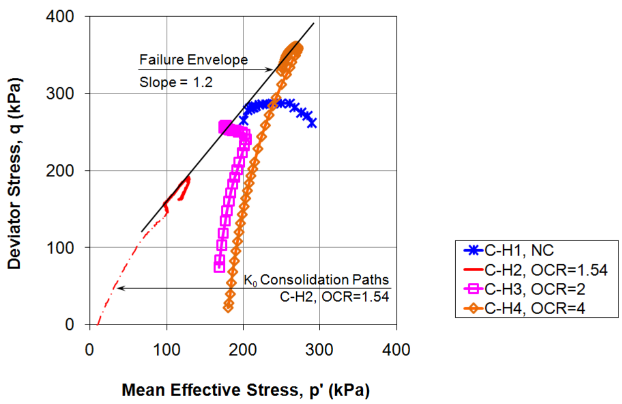

Figure 8 (a)

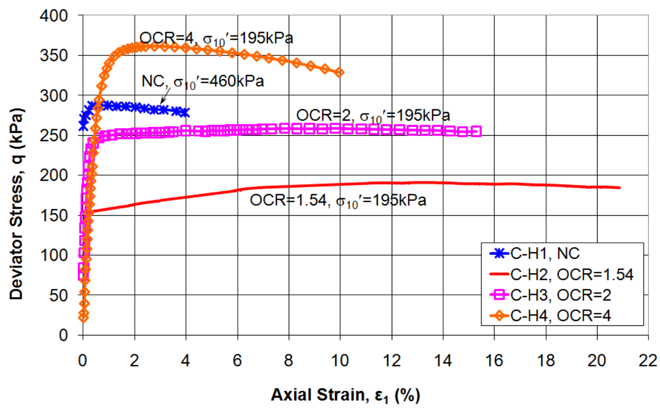

Figure 8 (b)

Figure 8. Undrained Behaviour of Horizontally Cut Intact Residual Soil during CK$_0$UC Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves

3.2 CK$_0$UE tests

A series of CK$_0$UE tests was conducted on both vertically cut and horizontally cut specimens. The stress behaviour resulted from CK$_0$UE tests are presented in Figure 9 and Figure 10. Same K_0 consolidation method as described in Section 1 was adopted to achieve different OCR values.

The effective stress paths of vertically cut samples resulted from CK$_0$UE tests are presented in Figure 9(a). The failure envelope was determined based on the peak effective stress ratio ($(q/p')_{peak}$). For vertically cut intact residual soil specimens, the slope of the failure envelope is 1.23 and with a friction angle of 30.70°.

Figure 9(b) presents the stress-strain curves obtained from the same tests. The deviator stress reached the peak value at relatively large strain (3-7%). For OC specimens consolidated to the same stress level, the higher the OCR value, the higher the peak deviator stress, q_{peak}.

Four CK$_0$UE tests were conducted on horizontally cut intact residual soil specimens with different OCRs. The stress paths and stress-strain curves are presented in Figure 10. The failure envelope is assumed to pass through the origin as CK$_0$UC tests (Figure 10(a)). For horizontally cut intact residual soil, strain softening was observed for both NC and OC specimen (Figure 10(b)).

3.3 CK$_0$UDSS tests

The stress behaviour of vertically cut and horizontally cut intact residual soil under direct simple shear testing condition is presented in Figure 11 and Figure 12 respectively. A modified NGI type of direct simple shear (DSS) apparatus was used in this study. The reinforced membrane used by Bjerrum & Landva (1966) was replaced by a stack of slip metal rings. The 1-D (or K_0) condition was achieved by using metal rings to restrain lateral deformation during consolidation stage. Same as CK$_0$UC and CK$_0$UE tests, the CK$_0$UDSS specimens were also K_0 consolidated to OCR of 1, 1.54, 2 and 4.

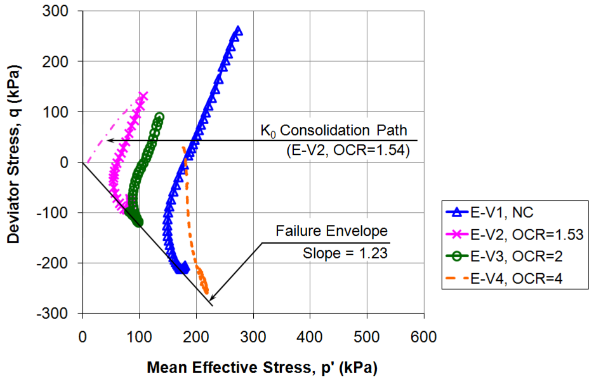

Figure 9 (a)

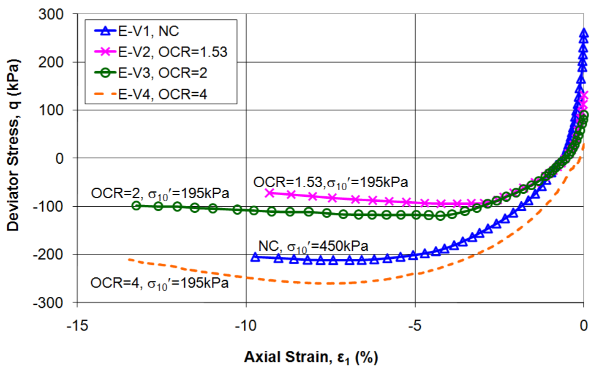

Figure 9 (b)

Figure 9. Undrained Behaviour of Vertically Cut Intact Residual Soil during CK_0UE Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves

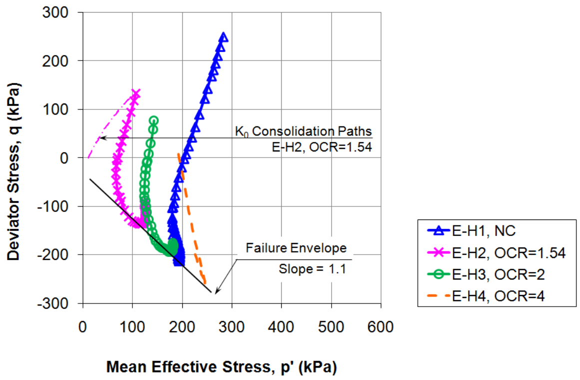

Figure 10 (a)

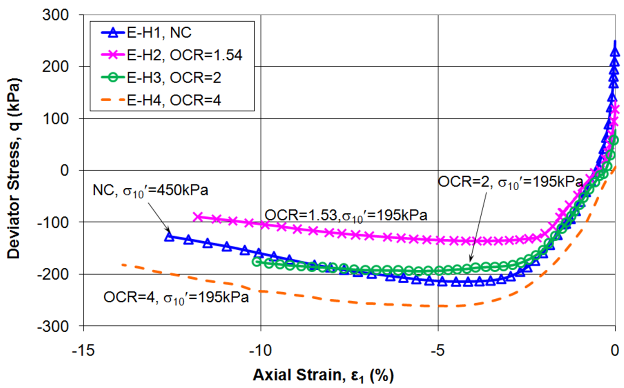

Figure 10 (b)

Figure 10. Undrained Behaviour of Horizontally Cut Intact Residual Soil during CK_0UE Tests: (a) Effective Stress Paths; (b) Stress-Strain Curves

The shear stress (τ) versus normal stress (σ_n) curves are presented in Figure 11(a) and Figure 12(a) based on the specimen cutting orientation. Failure envelopes with slope of 0.58 and 0.56 were obtained for vertically cut and horizontally cut specimens respectively. The failure envelope was determined from the peak stress ratio, $(\tau/\sigma_n)_{peak}$.

For vertically cut specimens, it is clearly shown that the shear resistant increased with OCR for OC specimen. The shear stresses reached the peak at relatively small strain (3%), then followed by a large amount of strain softening (Figure 11(b)). Similar observation was made on horizontally cut specimens, as presented in Figure 12(b).

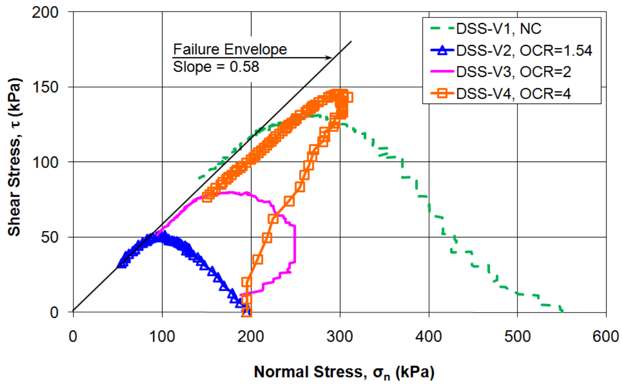

Figure 11 (a)

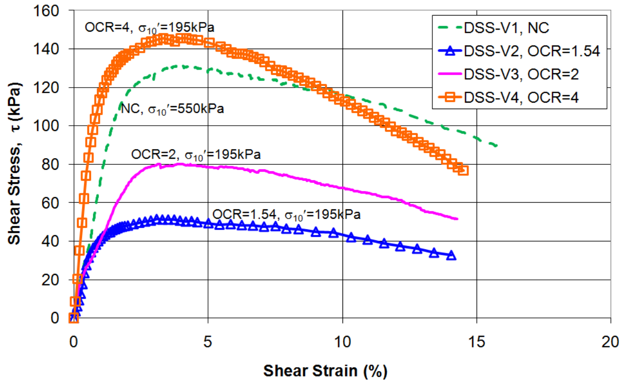

Figure 11 (b)

Figure 11. Undrained Behaviour of Vertically Cut Intact Residual Soil during CK_0UDSS Tests: (a) Shear Stress vs. Normal Stress Curves; (b) Shear Stress-Strain Curves

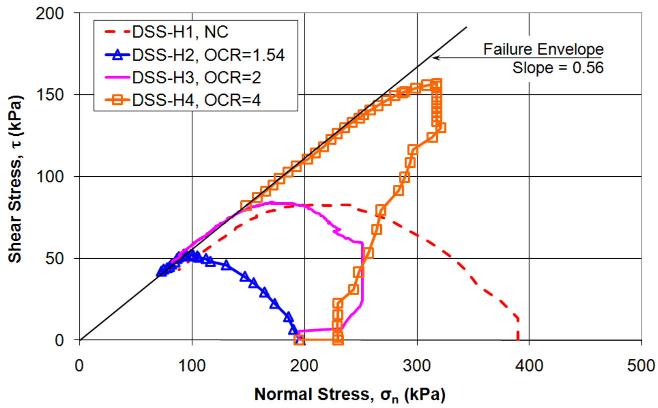

Figure 12 (a)

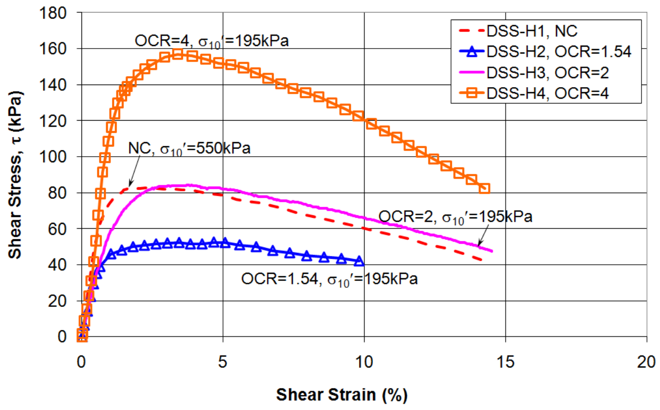

Figure 12 (b)

Figure 12. Undrained Behaviour of Horizontally Cut Intact Residual Soil during CK_0UDSS Tests: (a) Shear Stress vs. Normal Stress Curves; (b) Shear Stress-Strain Curves

4. DISCUSSION

4.1 Inherent strength anisotropy

The inherent strength anisotropy of the intact residual soil of Bukit Timah granite was studied by conducting tests on specimens cut in different orientations. The failure points determined from CK_0UC tests of vertically cut and horizontally cut specimens are compared in Figure 13(a). The failure point was determined from the points of peak effective stress ratio, $(q/p')_{peak}$. As shown in Figure 13(a), both vertically cut and horizontally cut intact residual soil specimens approached to a unique failure envelope. The solid line in Figure 13(a) represents the failure envelope within the tested stress range, which has a slope of 1.2. The dash lines are the assumed failure envelopes passing through the origin. It is clearly observed that the failure points determined for vertically cut and horizontally cut specimens are highly consistent. It is indicative that the effective failure envelopes are not affected by specimen orientation.

Figure 13(b) presents the comparison of c_u/σ_{10}' versus OCR relationships. The undrained shear strength c_u refers to the peak strength, which was calculated as $c_u = q_{peak}/2$. The data for both vertically cut and horizontally cut intact residual soil specimens fell into a consistent relationship. No observation of stress anisotropy was made for the intact residual soil. This indicated that the c_u/σ_{10}' versus OCR relationships was independent of the specimen orientation. An average $(c_u/\sigma_{10}')_{NC}$ value of 0.297 was taken from the data of normally consolidated vertically cut and horizontally cut specimens.

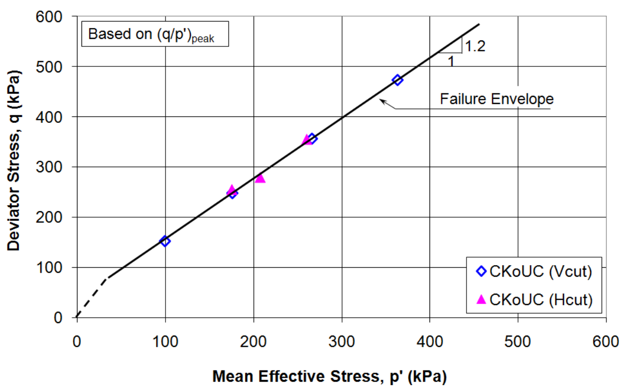

Figure 13 (a)

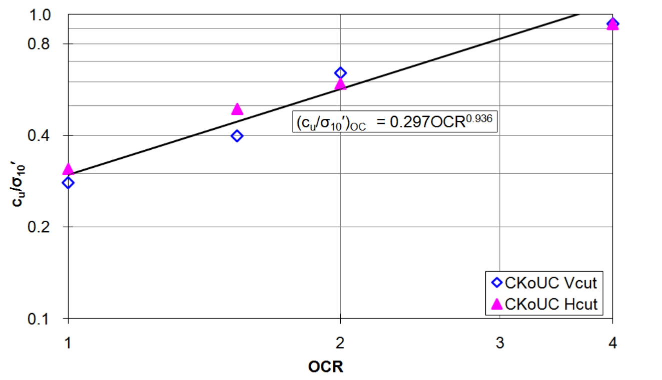

Figure 13 (b)

Figure 13. Comparison of CK_0UC Test Results: (a) Failure Envelope; (b) c_u/σ_{10}' vs. OCR Relationship

The failure points determined from CK_0UE tests were compared in Figure 14(a). It is noted that all the points fell into a straight line. Similar observation has been made from CK_0UC tests. It is an indication that the failure envelope is not greatly affected by the specimen orientation.

Figure 14(b) presents the comparison of c_u/σ_{10}' versus OCR relationships. There are some differences. However, as the data at ORC =1 and ORC =4 are fairly close, we may assume they follow the same relationship as shown in Figure 14(b). This is consistent with the results obtained from CK_0UC tests. The reasons for the small differences in the undrained shear strength behaviour between vertically cut and horizontally cut specimens are possibly: (1) the use of vertically and horizontally cut specimens is not a good testing method to study the anisotropic strength behaviour, (2) the samples at in-situ were probably consolidated under a stress condition close to the isotropic state as implied by the oedometer tests on vertically and horizontally cut specimens. Therefore, the inherent anisotropy is not evident.

Similar comparison has been made on CK_0UDSS test results in Figure 15. The results of vertically cut and horizontally cut specimens are highly consistent. Inherent strength anisotropy was not observed in terms of failure envelope and τ/σ_{n0} versus OCR relationship.

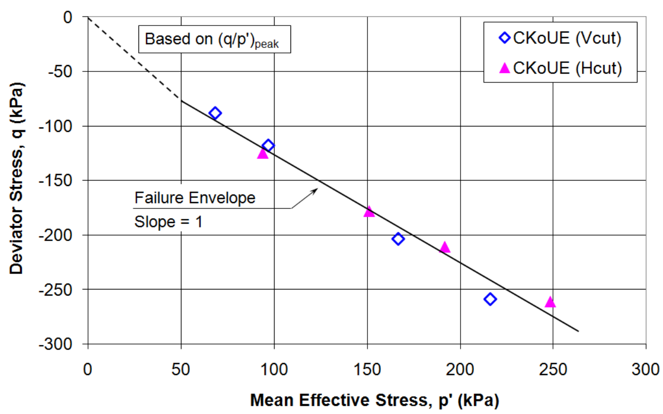

Figure 14 (a)

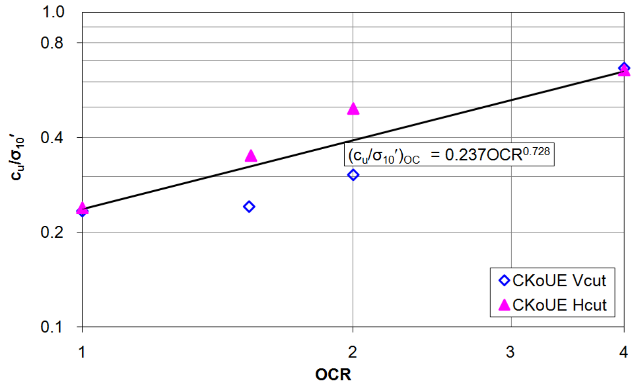

Figure 14 (b)

Figure 14. Comparison of CK_0UE Test Results: (a) Failure Envelope; (b) c_u/σ_{10}' vs. OCR Relationship

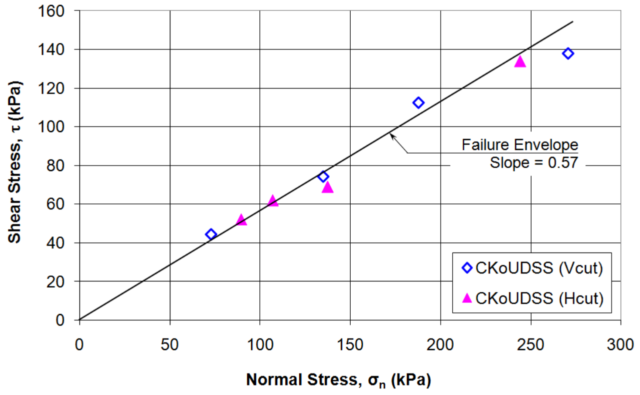

Figure 15 (a)

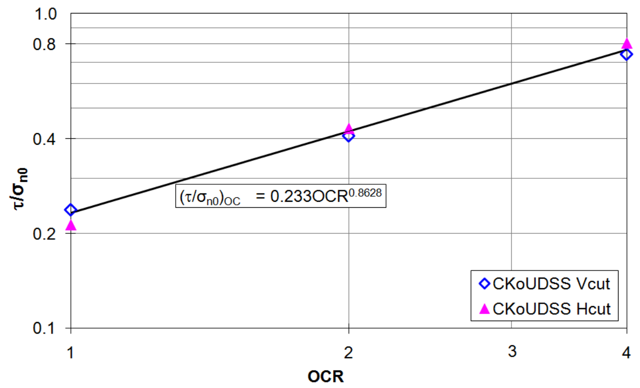

Figure 15 (b)

Figure 15. Comparison of CK_0UDSS Test Results: (a) Failure Envelope; (b) τ/σ_{n0} vs. OCR Relationship

4.2 Induced strength anisotropy

The strength anisotropy induced by rotation of major principal stress direction was investigated by conducting laboratory compression, extension tests, and direct simple shear tests. The c_u/σ_{10}' versus OCR relationships obtained from CK_0UC, CK_0UE, and CK_0UDSS tests are compared in Figure 16. As presented in earlier section, the c_u/σ_{10}' versus OCR relationships were independent of the specimen orientations. However, the relationships were influenced by the major principal stress direction. As shown in Figure 16, at a given OCR, the c_u/σ_{10}' ratio obtained from the CK_0UC tests was the highest. The c_u/σ_{10}' ratio obtained from the CK_0UE tests was the lowest. The one determined from CK_0UDSS was somewhere in between. This is consistent with the observations made from other soils (Ladd, 1986; Chu et al. 1999). It should be also pointed that CK_0UDSS tests were conducted under plane-strain condition, although relative magnitude of intermediate stress, $b = (\sigma_2 - \sigma_3)/(\sigma_1 - \sigma_3)$ cannot be measured directly. The induced anisotropy between TC, TE and DSS tests, is actually an reflection of the combined effects of b and α.

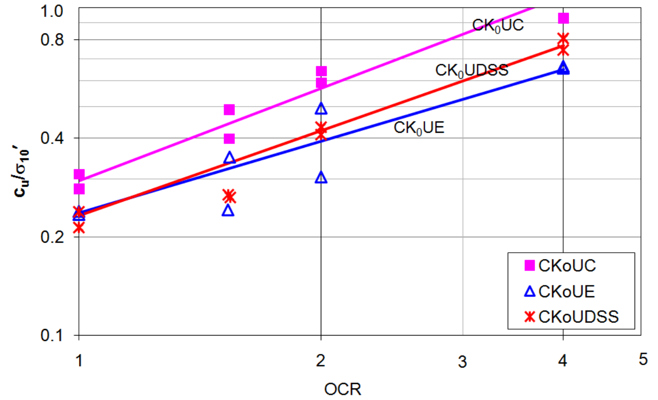

Figure 16. c_u/σ_{10}' vs. OCR Relationships Determined from Different Types of Tests

A comparison of the secant friction angle obtained from different type of tests is shown in Figure 17. Here secant friction angle, ϕ_s', is defined as:

$$\sin\phi_s' = (\sigma_1' - \sigma_3') / (\sigma_1' + \sigma_3') \qquad (2)$$

irrespective of whether the effective cohesion is zero or not. It is observed that the secant friction angle is affected by consolidation stress. This is caused by whether bonds in the intact residual could be preserved or not which is related to whether the stress level is above or below the pre-consolidation stress. Consequently, the soil behaviour was partially controlled by the soil bonds left behind. For all types of tests, the secant friction angles obtained from tests conducted on the specimen, which consolidated to the in-situ stress was the highest. This is due to the preservation of soil bonds at low consolidation stress. On the other hand, NC, OCR2, OCR3 and OCR4 specimens had been consolidated to a stress level much more higher than the pre-consolidation stress and the structure of the residual soil specimens have been destroyed during consolidation process as pointed out by Smith et al. (1992).

For vertically cut intact residual soil specimens, CK_0UC tests gave the highest ϕ_s', followed by CK_0UE tests. The lowest ϕ_s' was calculated from CK_0UDSS tests.

Different from the CK_0UDSS test results on vertically cut specimen, no significant reduction in the ϕ_s' value was observed for horizontally cut specimens. Hence, at high OC state (OCR > 2), value of ϕ_s' determined from CK_0UDSS tests was higher than the one obtained from CK_0UE tests.

By comparing Figure 17(a) and (b), it should also point out that the ϕ_s' values of horizontally cut intact residual soil were higher than the values of vertically cut specimen. The ϕ_s' values were affected by specimen orientation.

The c_u of soil is also affected by the rotation of principal stress direction. The effect of principal stress rotation can be evaluated by plotting the ratio of $c_{u/\alpha}/c_{u/CK_0UC}$ versus the angle of major principal stress rotation to the vertical, α. α = 0° represents the normal compression testing condition, with the major principal stress at failure, σ_{1f}' acting vertically. α = 90° represents the major principal stress direction to be horizontal, as in CK_0UE condition.

$\alpha = 45°$ is assumed for the principal stress direction in a CK_0UDSS testing condition for illustration purpose.

The results of the three tests, after being normalized with the undrained shear strength for $\alpha = 0°$ are presented in Figure 18(a) and (b) for vertically cut and horizontally cut intact residual soil specimens, respectively. It can be seen that the undrained shear strength is significantly affected by rotation of major principal stress, regardless of specimen cutting orientations. This is consistent with previous studies carried out by O'Rourke et al. (1976). It is indicated that the strength anisotropy could be investigated better by conducting tests with different major principal stress directions.

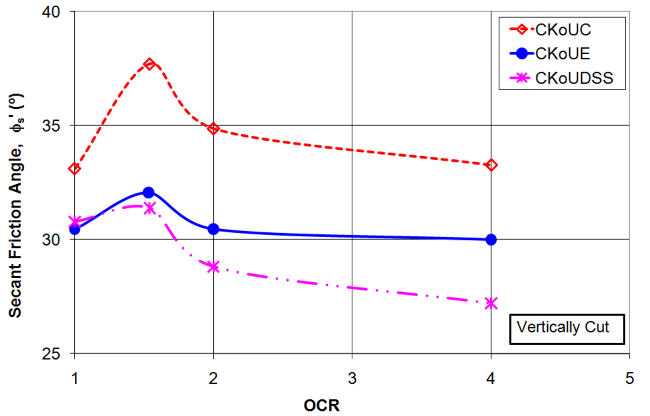

Figure 17 (a)

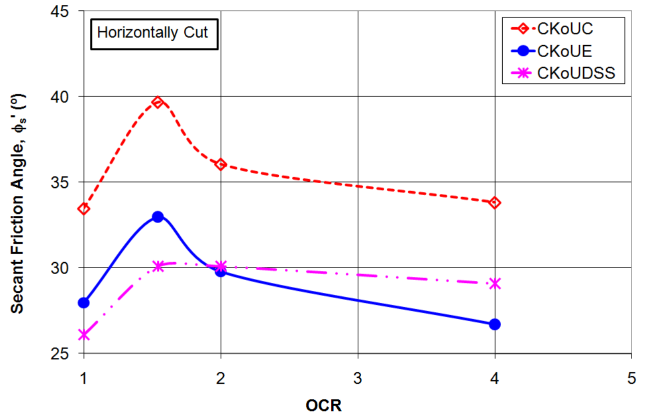

Figure 17 (b)

Figure 17. Secant Friction Angles Determined from Different Types of Tests: (a) Vertically Cut Specimens; (b) Horizontally Cut Specimens

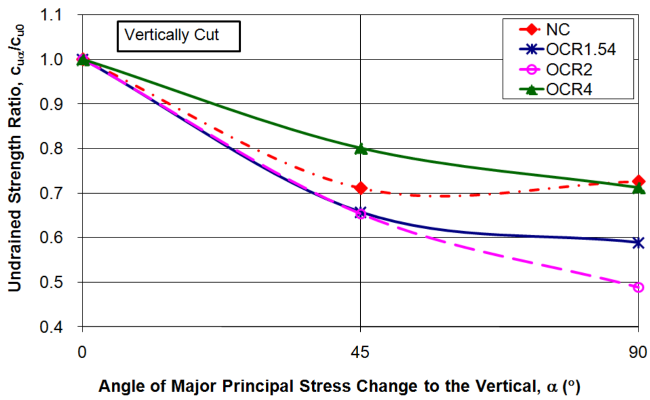

Figure 18 (a)

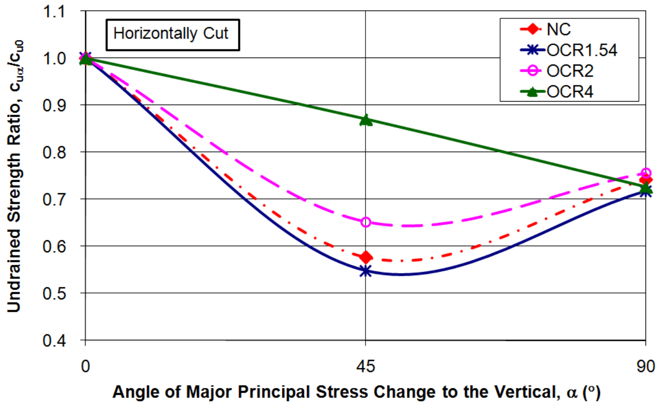

Figure 18 (b)

Figure 18. Variation of Undrained Strength Ratio with the Angle of Stress Reorientation: (a) Vertically Cut Specimens; (b) Horizontally Cut Specimens

5. CONCLUSIONS

The undrained shear strength and stress-strain behaviour of the intact residual soil of Bukit Timah granite have been presented with reference to the results of CK_0UC, CK_0UE and CK_0UDSS tests. Following conclusions can be made from these results:

1. As pointed out by many researchers (Ladd, 1986; Kulhawy & Mayne, 1990; Chu et al., 1999), the undrained shear strength, c_u, is not a constant, but varies with many factors, including effective overburden stress, σ_{10}', OCR, consolidation path (isotropic or K_0 consolidation), stress states and stress anisotropy. From the test results presented in this paper, it is found that the undrained behaviour of the intact residual soil is affected greatly by the consolidation stress. As presented in Figure 17, an excessively high secant friction angle was obtained at low stress level (i.e. below pre-consolidation stress). It is believed that this is caused by the soil bonds preserved when consolidated below pre-consolidation stress.
2. The failure envelope of the intact residual soil was not greatly affected by the cutting orientations of soil specimens. This is observed in different types of tests including CK_0UC, CK_0UE and CK_0UDSS tests conducted on specimens cut with different orientations. This could be because the influence of inherent anisotropy on shear strength, if any, could be diminished during to the shearing effect.
3. The c_u/σ_{10}' versus OCR relationships were established for each type of tests. The tests conducted on vertically and horizontally cut specimens did not show a strong strength anisotropy in terms of c_u/σ_{10}' ratio. However, pronounced strength anisotropy both in terms of effective friction angle and undrained shear strength was observed from the tests with major principal stress rotation. It is observed from the testing results that, in general, under a given OCR the c_u/σ_{v0}' obtained from the CK_0UC tests is the highest. The CK_0UE tests give the lowest one, whereas the value obtained from CK_0UDSS tests is somewhere in between. The strength anisotropy was induced by different degree of bond breakdown due to different volumetric responses with major principal stress rotation and effect of intermediate stress. This is consistent with the observations of Ladd & Lambe (1963), Ladd et al., (1980), Malandraki and Toll (2000), Wanatowski & Chu (2008) and Mayne et al. (2009).

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7. REFERENCES

Bica, A.V.D., Bressani, L.A., Vendramin, D., Martins, F.B., Ferreira, P.M.V. and Gobbi, F. (2008) "Anisotropic shear strength of a residual soil of sandstone", Canadian Geotechnical Journal, 45, No 3. Pp. 367-376.

Bjerrum, L. and Aitchison, G. D. (1973) "Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others)", Proceedings of the International Conference on Soil Mechanics and Foundation Engineering = Comptes Rendus du Congres International de Mecanique des Sols et des Travaux de Fondations, 3, No. 8, pp. 111-190.

Bjerrum, L. and Landva, A. (1966) "Direct simple-shear tests on a norwegian quick clay", Geotechnique, 16, Issue 1, pp1-20.

Broms, B. B. and Casbarian, A. O. (1965) "Effect of Rotation of the Principal Stress Axes and of the Intermediate Principal Stress on the Shear Strength", Proceedings of 6th International Conference on Soil Mechanics and Foundation Engineering, 1, pp. 179-183.

Casagrande, A. and Carrillo, N. (1944) "Shear failure of anisotropic materials", Boston Society of Civil Engineers -- Journal, 31, No. 2, pp. 74-87.

Chu, J. (1991) "Strain Softening Behaviour of Granular Soils under Strain Path Testing", PhD Thesis, Department of Civil and Marine Engineering, The University of New South Wales, Australian Defense Force Academy, Australia.

Chu, J., Choa, V. and Bo, M. W. (1999) "Determination of Undrained Shear Strength of Clay by Direct Simple Shear Tests", Proceedings of 11th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Hong et al. (Eds), pp. 49-52.

Clough, G. W. and Hansen, L. A. (1981) "Clay anisotropy and braced wall behavior", Journal of the Geotechnical Engineering Division, 107, No. GT7, pp. 893-913.

Fookes, P. G. (1997) Tropical residual soils: a Geological Society Engineering Group Working Party revised report Geological Society from The Geotechnical society Publishing House.

Graham, J. (1979) "Embankment stability on anisotropic soft clays", Canadian Geotechnical Journal = Revue Canadienne de Geotechnique, 16, No. 2, pp. 295-308.

Hight, D.W. (2001) "Sampling effects in soft clay: an update on Ladd and Lambe (1963) ", Ladd Retirement Symposium, MIT, Cambridge, USA

Huat, B. B. K., Aziz, A. A., Ali, F. H. and Azmi, N. A. (2008) "Effect of wetting on the collapsibility and shear strength of tropical residual soils", Electronic Journal of Geotechnical Engineering, Vol.13 G.

Hvorslev, M. J. (1960) "Physical components of the shear strength of saturated clays", ASCE Res. Conf. on the Shear Strength of Cohesive Soils, Boulder, Colo., USA, pp. 169-274.

Joer, H. A., Lanier, J. and Fahey, M. (1998) "Deformation of granular materials due to rotation of principal axes", Geotechnique, 48, No. 5, pp. 605-619.

Kulhawy, F. H. and Mayne, P. W. (1990) "Manual on Estimating Soil Properties for Foundation Design", Report EL-6800, Electric Power Research institute, Palo Alto, 360.

Ladd, C. C. (1986) "Stability Evaluation during staged construction", Journal of Geotechnical engineering, ASCE, 117, No. 4, pp. 540-615.

Ladd, C. C. and Lambe, L. W. (1963) "The Strength of Undisturbed Clay Determined from Undrained Tests.", Laboratory shear Testing of soils, Vol.STP361, ASTM, West Conshohocken/PA, pp. 342-371.

Ladd, C. C., Germaine, J. T., Baligh, M. M. and Lacasse, S. M. (1980) "Evaluation of self-boring pressuremeter tests in Boston blue clay". Report FHWA/RD-80/052, US Federal Highway Administration, Washington, D.C.

Lambe, T. W. (1958) "Clay structure and engineering behavior", ASCE -- Proceedings -- Journal of the Soil Mechanics and Foundations Division, 84, No. SM2, Part 1.

Leroueil, S. and Hight, D. W. (2002) "Behaviour and properties of natural soils and soft rocks", Characterisation and Engineering Properties of Natural Soils, A.A. Balkema, Publishers, United States, pp. 29-254.

Leroueil, S. and Vaughan, P. R. (1990) "The general and congruent effects of structure in natural soils and weak rocks", Geotechnique, 40, No. 3, pp. 467-488.

Lo, S. C. R. and Chu, J. (1991) "Discussion of "Instability of Granular Materials with Nonassociated Flow"", Journal of Engineering Mechanics, ASCE, 117, No. 4, pp. 930-933.

Malandraki, V. and Toll, D. G. (2000) “Drained Probing Triaxial Tests on a Weakly Bonded Artificial Soil”, Geotechnique, 50, No. 2, pp. 141 – 151.

Mayne, P. W., Coop, M. R., Springman, S. M., Huang., A.-B. and Zornberg, J. G. (2009) "Geomaterial Behavior and Testing (SAO report)", Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, alexandria, Egypt.

Meng, G. H. and Chu, J. (2011) “Shear Strength Properties of a Residual Soil in Singapore”, Soils and Foundations, 51, No. 4, pp. 565-573.

Meng, G. H., Chu J., and Klotz C. (2007) Undrained Shear Strength of a Residual Soil Measured by Triaxial and Direct Simple Shear Tests, Proceedings of the 16th South-East Asian Geotechnical Conference, Kuala Lumpur, Malaysia, 375-380.

Meng, G. H., Chu J., and Klotz U. (2008) Engineering Properties of Bukit Timah Residual Soil at a Tunneling Project Site in Singapore, Proceedings of the International Conference on Deep Excavations, 2008, Singapore.

Nagaraj, T. S., Prasad, K. N., Reddy, V. M. C. and Reddy, N. G. (1998) "Analysis of residual tropical cemented soil behaviour", The geotechnics of hard soils - soft rocks. Proceedings of the second international symopsium on hard soils-soft rocks, Naples, Italy, A.A.Balkema, 2, pp. 715-723.

O’Rourke, T. D., Cording E, J. and Boscardin, M. (Aug, 1976) "The Ground Movements Related to Braced Excavations and Their Influence on Adjacent Buildings", Report No. DOT-TST-761-23, United States Department of Transportation.

Rahardjo, H., Aung, K. K., Leong, E. C. and Rezaur, R. B. (2004) "Characteristics of residual soils in Singapore as formed by weathering", Engineering Geology, 73, No. 1-2, pp. 157-169.

Rao, S. M. and Revanasiddappa, K. (2006) "Influence of cyclic wetting drying on collapse behaviour of compacted residual soil", Geotechnical and Geological Engineering, 24, No. 3, pp. 725-734.

Smith, P. R., Jardine, R. J. and Hight, D. W. (1992) "The yielding of Bothkennar clay", Geotechnique, 42, No. 2, pp. 257-274.

Vaughan, P. R. (1988) "Characterising the Mechanical Properties of the In-Situ Residual Soil", Proceedings of the 2nd International Conference on Geomechanics in Tropical Soils, Singapore, Balkema, Rotterdam, 2, pp. 469-487.

Vaughan, P. R. and Kwan, C. W. (1984) "Weathering, structure and in situ stress in residual soils", Geotechnique, 34, No. 1, pp. 43-59.

Viana da Fonseca, A. (2003) "Characterising and Deriving Engineering Properties of a Saprolitic Soil from Granite, inn Porto", Characterisation and Engineering Properties of Natural Soils, 2, pp. 1341-1378.

Wanatowski, D. and Chu, J. (2008) "Undrained behaviour of Changi sand in triaxial and plane-strain compression", Geomechanics and Geoengineering, 3, No. 2, pp. 85-96.

Winn, K., Rahardjo, H. and Peng, S. C. (2001) "Characterization of residual soils in Singapore", Journal of the Southeast Asian Geotechnical Society, 32, No. 1, pp. 1-13.

Zdravkovic, L., Potts, D. M. and Hight, D. W. (2002) "The effect of strength anisotropy on the behaviour of embankments on soft ground", Geotechnique, 52, No. 6, pp. 447-457.

Zhang, G., Whittle, A. J., Germaine, J. T. and Nikolinakou, M. A. (2007) Characterization and engineering properties of the Old Alluvium in Puerto Rico_Taylor & Francis Group London, pp.2557-2588.