ISSN: 0128-7680
Pertanika J. Sci. & Technol. 7(2): 151-169 (1998)

Geotechnical Behaviour of a Malaysian Residual Granite Soil

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Received: 13 March 1998

ABSTRAK

Tanah baki granit tersebar dengan begitu meluas di Semenanjung Malaysia. Bentuk muka bumi yang curam, hujan tahunan yang lebat serta luluhawa tropika yang dalam merupakan sebab utama berlakunya banyak kegagalan cerun yang melibatkan tanah ini. Oleh itu keterangan mengenai kekuatan ricih tanah serta kelakuannya amat penting bagi merekabentuk struktur geoteknik yang selamat dan ekonomi. Untuk mengkaji ciri-ciri asas geoteknik tanah baki ini, ujian ricih terus bagi sampel yang terganggu dan tak terganggu telah dilakukan. Tanah telah diuji bagi keadaan direndam dan tak direndam untuk mengkaji kesan pembasahan. Tambahan daripada ini ujian paksi tiga CD dan CU juga telah dijalankan. Kelakuan tanah dalam ricihan terus dan paksi tiga menunjukkan tiada kekuatan puncak dalam pelotan tegasan-terikan. Dalam ujian ricih terus, kelakuan perubahan isipadu yang amat berbeza dilihat antara sampel tak direndam dan direndam. Tambahan lagi, kekuatan sampel yang direndam adalah jauh lebih rendah daripada sampel tak direndam. Dengan itu kesan pembasahan adalah jelas. Dalam ujian paksi tiga, parameter kekuatan (c', ϕ') bagi ujian CU dan CD adalah tidak sama. Kepentingan semua pemerhatian diperjelaskandalam kertas ini.

ABSTRACT

Granite soil is widely distributed in Peninsular Malaysia. Steep terrain, heavy seasonal rainfall and deep tropical weathering are the main causes of numerous slope failures in this formation. Information on the shear strength of soil and its behaviour is essential for safe and economic design geotechnical structures. In order to study the fundamental behaviour of this residual soil, direct shear tests were conducted on remoulded and undisturbed specimens. The soil was subjected to soaked and unsoaked conditions to study the effects of wetting. In addition, triaxial CD and CU tests were also conducted. The behaviour in direct and triaxial shear did not indicate peak strength in the stress-strain plots in all specimens. In direct shear tests, significant volume change behaviour was observed between unsoaked and soaked specimens. The shear strengths of soaked specimens were significantly lower than that of unsoaked specimens. Thus the effects of wetting is very obvious. In triaxial tests, the strength parameters (c', ϕ') for CD and CU tests were not found similar. All important observations are highligted in the paper.

Keywords: Residual soil, Malaysian soil, soil testing, direct shear test, triaxial shear test, undrained strength, drained strength

INTRODUCTION

In Malaysia, steep terrain, heavy seasonal rainfall and deep tropical weathering cause numerous slope failures every year. Intense hill slope development, associated with formation of large cuttings requires substantial resources be devoted to shear strength testing of residual soils for the purpose of safe and economic design. Well known major failures of geotechnical structures and formation in Malaysia in the past 2 years include Highland Tower Block Towers, Genting Highland debris flow and Pos Dipang (Komoo 1997). Coincidentally, these failures occurred in granite residual soil formation.

Traditionally, the preferred methods of testing are consolidated drained triaxial test and the consolidated undrained triaxial test with pore pressure measurements. These are conducted at low effective confining pressure which are relevant to the field conditions. However, the triaxial test is not only expensive and laborious but also time consuming. Direct shear tests are much simpler and quicker to perform, and appears to offer a solution to the concerns relating to triaxial test. Theoretical objections to the use of the test remain, such as rotation of principal stresses, progressive failure of the specimen, and inability to measure pore pressure. However, the field conditions of low overburden pressure, and wetting of the soil by infiltration and soaking under low head, can be simulated more easily in direct shear tests and the strain conditions are more representative for majority of slope failures observed in the field (Cheung *et al.* 1988). Furthermore, Ting and Ooi (1972) mentioned that shear box values resemble that of drained triaxial tests. However, differences in values may result as a consequence of differences in the shearing system. Therefore, extensive direct shear tests were carried out in this research programme to examine the shear strength behaviour of the granite residual soil. In addition, the triaxial tests were also conducted to further examine the characteristics of the soil.

Thus, the main aim of the research work is to perform a detailed investigation on factors that influence the shear strength characteristics of granite residual soil. The results will give us a better understanding of the shear strength behaviour of granite residual soil. This will improve our efforts in solving the geotechnical problems concerning different types of slope and foundation failures in granite residual soils of Malaysia.

MATERIALS AND METHODS

The soil used in this study was obtained from a residual granite soil formation just 8 km southeast of Kuala Lumpur, the capital city of Malaysia. The block sampling technique was adopted for soil sampling in which thirty 300 mm cube samples were hand cut, placed in wooden boxes, wax around the sides and stored in the laboratory until required for testing. Basic geotechnical tests such as the grain size distribution, moisture content, Atterberg's limits, etc. were conducted following test procedures mentioned in BS 1377 (1990).

The main geotechnical testing programme includes direct and triaxial shear tests. The direct shear tests were carried out for specimens in unsoaked and soaked conditions to investigate the effects of wetting. These tests were conducted on the soil encompassing both horizontal (H-specimens) and vertical (V-specimens) shear plane orientation with constant normal stress (*Fig. 1*) to analyse its anisotropic behaviour.

The normal stresses used were 50, 102 and 205 kPa. The dimensions of the specimens were 60 mm square by 20 mm in height. They were prepared by cutting the block samples using a knife and a cutter. The specimens were placed in the shear box. A normal stress of 5 kPa was applied and the specimens were flooded with water for saturation. The specimens were allowed to soak for 24 hours. After soaking, the specimens were consolidated to the required normal stress. When at least 98 percent consolidation had been achieved, the specimens were then sheared to failure. Tests were carried out under the strain control condition at the rate of 0.065 mm/min. Both undisturbed and compacted specimens were tested.

Consolidated drained and undrained triaxial tests were carried out with cell pressures of 400 - 600 kPa and back pressure 300 kPa. The tests were carried out with constant rate of axial strain of 0.065 %/min in both cases. Test data such as cell pressure, back pressure, pore water pressure, axial load, volume change and axial strain were monitored by computer controlled triaxial mechine.

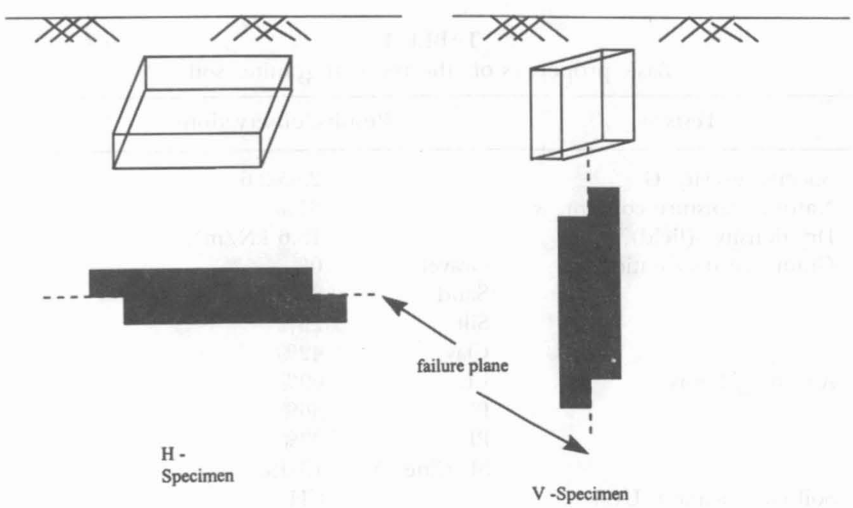

Fig 1. Failure planes of H and V-specimens

RESULTS AND DISCUSSION

Basic Soil Tests

The basic properties of the residual soil is given in Table 1. The mean dry unit weight from field tests was 13.6 kN/m^3 and the natural moisture content was about 31 %. The overall mean dry unit weight obtained from laboratory test was 13.52 kN/m^3. Initial moisture content agreed well with the natural moisture contents, indicating that there had been little drying out of the specimens. The specific gravity of this soil (2.55 to 2.6) shows a change from the specific gravity of fresh granites (2.65 to 2.68). This change in specific gravity of the decomposed granite soils from fresh rock indicates that a significant amount of clay minerals was formed during weathering. It is generally accepted that the void ratio increases as the granite is gradually weathered. The decomposed granite for which the porosity exceeds 40 % (natural void ratio $e_n > 0.7$) may be classified in the group of perfectly decomposed granite soils (JSSMFE 1979). Based on Unified Soil Classification System (UCS), the soil was grouped as "clay with high plasticity" (CH). It was also classified in the "A-7-6" group according to the AASHTO classification system.

In consolidation tests, the residual soil consolidated rapidly and almost 50 to 60 % primary consolidation was completed within 10 seconds of the beginning of loading. Coefficient of consolidation has a value of approximately 1.1 m^2/year. Optimum moisture content and maximum dry density are found to be 23 % and 1497.5 kg/m^3, respectively from compaction tests. The compaction values are within the range reported by Tan and Ong (1993).

TABLE 1
Basic properties of the residual granite soil

Tests		Results/observations
Specific gravity, G_s		2.55-2.6
Natural moisture content, w		31%
Dry density (field), γ_d		13.6 kN/m^3
Grain size distribution	Gravel	0%
	Sand	35%
	Silt	23%
	Clay	42%
Atterberg limits	LL	69%
	PL	36%
	PI	33%
	SL (linear)	13.9%
Soil classification: UCS		CH
AASHTO		A-7-6
pH		4.6
Organic carbon content, (C_{oc})		1.37%
Oedometer consolidation, C_v		1.1 m^2/year

Direct Shear Tests-undisturbed Specimens

Fig. 2 shows the relation between shearing stress and horizontal displacement for consolidated drained test of the undisturbed H-specimens in soaked and unsoaked conditions. Generally, there were no distinct peak points in stress-displacement curves and the curves for soaked condition located well below that of unsoaked specimens. For unsoaked condition, vertical expansion (dilation) was observed at low normal stress and contractions or settlements at higher normal stress (*Fig 3*). Vertical settlements dominated for specimens in soaked condition at all normal stress levels. Results of all other tests on undisturbed H-specimens in soaked and unsoaked conditions are summarised in Table 2.

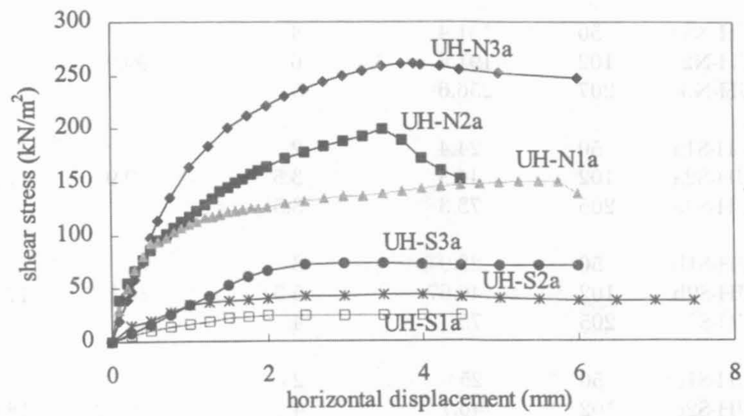

Fig 2. Relation between shear stress and horizontal displacement of H-specimens for CD test in soaked and unsoaked conditions

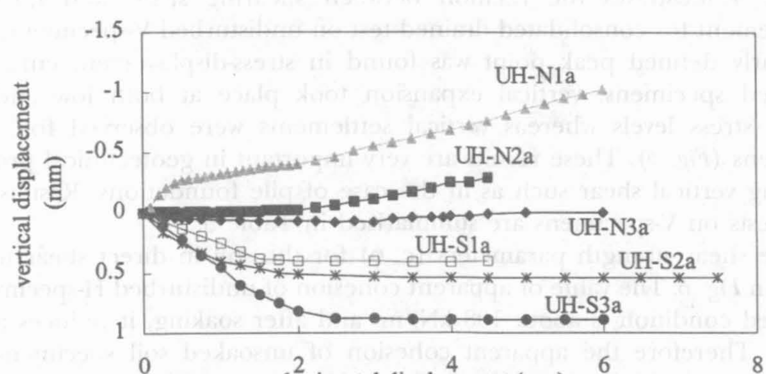

Fig 3. Relation between horizontal and vertical displacements of H-specimens for CD test in soaked and unsoaked conditions

TABLE 2
Results of direct shear test on undisturbed H-specimens in soaked and unsoaked conditions

Set No	Specimen No	Normal Stress kN/m²	Maximum Shear Stress kN/m²	Displacement at τ_{max} mm	Cohesion c' kN/m²	Angle ϕ' degree
	UH-N1a	50	150.5	4		
a	UN-N2a	102	199.4	4	120.7	34.6
	UH-N3a	207	261	3.5		
	UH-N1b	50	127	3.5		
b	UH-N2b	102	197.2	5	103.9	35.9
	UH-N3b	207	246.6	4		
	UH-N1c	50	131.4	4		
c	UH-N2c	102	191.6	6	99.9	37.9
	UH-N3c	207	256.6	5		
	UH-S1a	50	24.4	2		
a	UH-S2a	102	44.3	3.5	9.9	17.3
	UH-S3a	205	73.3	3.5		
	UH-S1b	50	23.9	3		
b	UH-S2b	102	46.67	5.5	10.1	17.8
	UH-S3b	205	75	4		
	UH-S1c	50	25	2		
c	UH-S2c	102	46.7	4	9.62	18.6
	UH-S3c	205	75	4.5		

Fig. 4 illustrates the relation between shearing stress and horizontal displacement for consolidated drained test on undisturbed V-specimens. Again no clearly defined peak point was found in stress-displacement curves. For unsoaked specimens, vertical expansion took place at both low and high normal stress levels whereas vertical settlements were observed for soaked specimens (*Fig. 5*). These results are very important in geotechnical problems involving vertical shear such as in the case of pile foundations. Results of all other tests on V-specimens are summarised in Table 3.

The shear strength parameters (c, ϕ) for the soil in direct shear test are shown in *Fig. 6*. The value of apparent cohesion of undisturbed H-specimens in unsoaked condition is about 108 kN/m² and after soaking, it reduces to 10.5 kN/m². Therefore the apparent cohesion of unsoaked soil specimens were about ten times higher than that of soaked specimens. This is possibly due to the effect of soil suction and less lubricating effect in unsoaked condition that prevents soil slippage and movement. Similar observations were obtained for

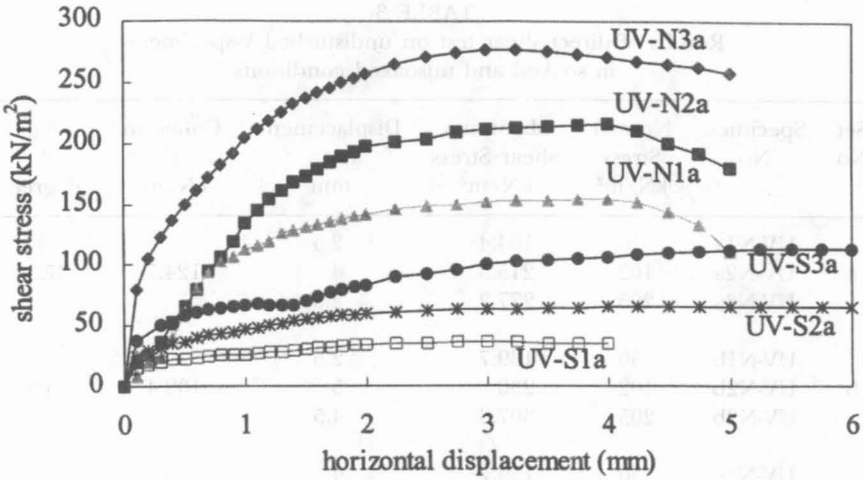

Fig 4. Relation between shear stress and horizontal displacement of V-specimens in soaked and unsoaked conditions

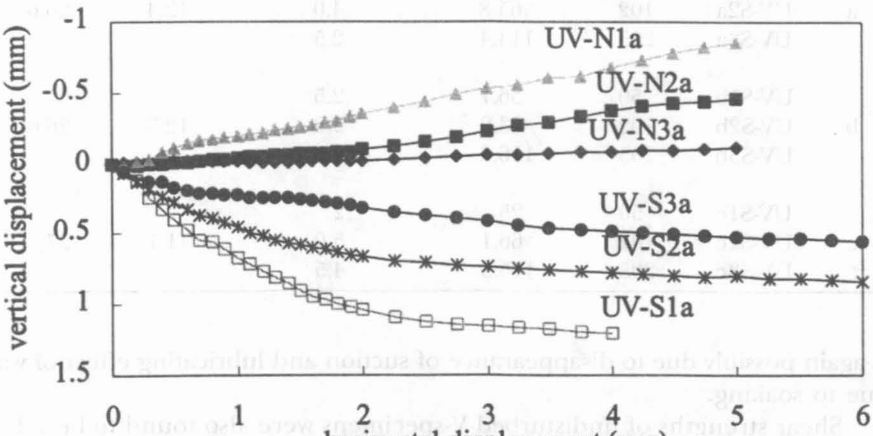

Fig 5. Relation between horizontal and vertical displacements of V- specimens for CD test in soaked and unsoaked conditions

undisturbed V-specimens. In unsoaked condition the value was about 110 kN/m^2 and after soaking, it reduced to 12 kN/m^2.

The angle of shearing resistance of undisturbed H and V-specimens in unsoaked condition are 36.3° and 43.3°, respectively and after soaking it reduces to 17.5° and 26.6°. This translates into a reduction in the range of 40 to 50 % of angle of shearing resistance. Such phenomenon was also found by Radwan (1988). He mentioned that angle of friction reduced noticeably and the cohesion was almost lost due to soaking of decomposed granite soil. This

TABLE 3
Results of direct shear test on undisturbed V-specimens in soaked and unsoaked conditions

Set No	Specimen No	Normal Stress kN/m²	Maximum Shear Stress kN/m²	Displacement at τ_{max} mm	Cohesion c' kN/m²	Angle ϕ' degree
	UV-N1a	50	154.4	2.5		
a	UV-N2a	102	215.5	4	124.7	37.3
	UV-N3a	205	277.2	2.5		
	UV-N1b	50	139.7	2.5		
b	UV-N2b	102	230	5	102.4	45.9
	UV-N3b	205	307.8	4.5		
	UV-N1c	50	139.7	4		
c	UV-N2c	102	230	5	134.3	40.4
	UV-N3c	205	307.8	4.5		
	UV-S1a	50	36.1	2.5		
a	UV-S2a	102	65.8	4.0	12.4	26.6
	UV-S3a	205	114.4	2.5		
	UV-S1b	50	36.7	2.5		
b	UV-S2b	102	63.9	5.0	12.3	26.6
	UV-S3b	205	116.3	4.5		
	UV-S1c	50	25	2		
c	UV-S2c	102	66.1	5.0	11.1	27
	UV-S3c	205	115.5	4.5		

is again possibly due to disappearance of suction and lubricating effect of water due to soaking.

Shear strengths of undisturbed V-specimens were also found to be 1.1~1.2 and 1.3~1.6 times of H-specimens strength for unsoaked and soaked conditions, respectively. The difference between the shear strength of V and H-specimens in both soaked and unsoaked conditions is small at low normal stresses and increases with increasing normal stresses. These observations show that anisotropy does exist but it is not very pronounced in granite residual soils as indicative by its mode of formation. Sedimentary deposits on the other hand are formed layer by layer in the vertical direction, thus resulting in significant anisotropy. Similar observations were made by Onitsuka *et al.* (1985) in which for undisturbed granite soil, shear strength of V-specimens was greater i.e. 1.1~1.5 times that of H-specimen. This strength anisotropy may be caused by an anisotropic earth pressure at rest before sampling and the anisotropy of granite before weathering.

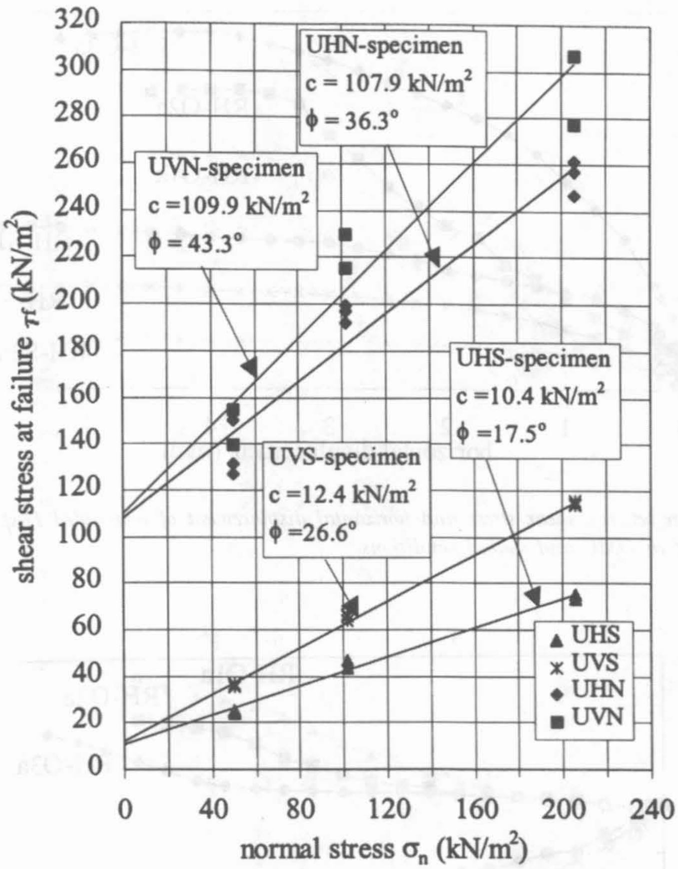

Fig 6. Shear vs normal stresses for undisturbed H and V-specimens in both soaked and unsoaked conditions

Direct shear tests-remoulded specimens

Fig. 7 shows the relation between shearing stress and horizontal displacement for remoulded H-specimens at optimum moisture content (OMC) and soaked conditions. There were no distinct peak points in stress-displacement curves. For soil at OMC, settlement initially took place upon shearing, then expansion of soil specimens prevails (*Fig. 8*). However, soaked soil experienced settlement throughout the shearing stage. Results of all other tests on remoulded H-specimens at OMC and soaked conditions are summarised in Table 4.

The results for V-specimens are shown in *Figs. 9* and *10*. Again no clearly defined peak points were found in stress-displacement curves. Their trends of behaviour during shearing are also similar to those of H-specimens mentioned above. Results of all other tests on V-specimen in OMC and soaked condition are summarised in Table 5.

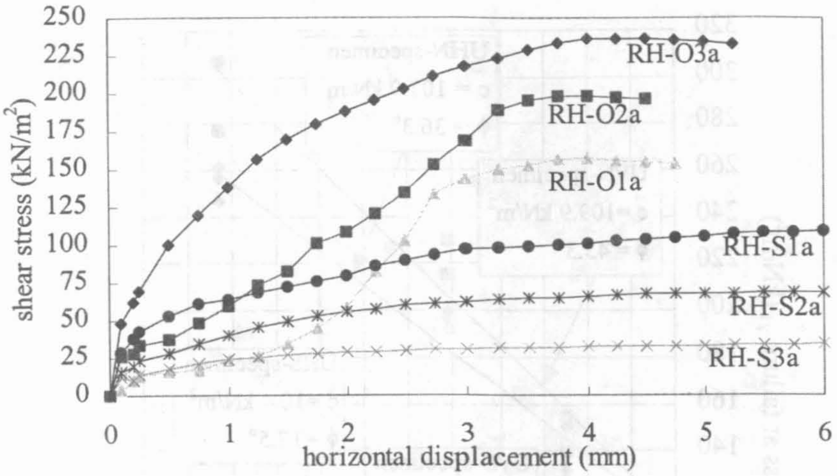

Fig 7. Relation between shear stress and horizontal displacement of remoulded H-specimens or CD test in OMC and soaked conditions

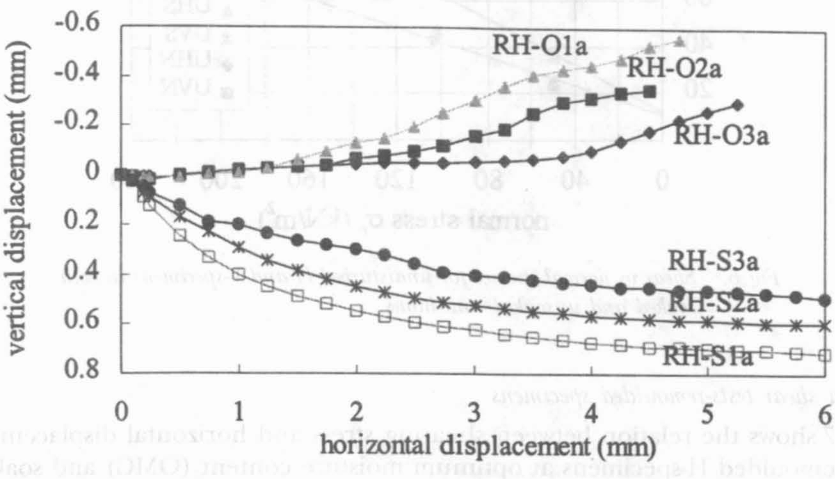

Fig 8. Relation between horizontal and vertical displacement of remoulded H-specimens for CD test in OMC and soaked conditions

Shear strength parameters (c', ϕ') for consolidated drained shear test on remoulded H and V-specimens at OMC and soaked conditions are shown in *Fig. 11.* The apparent cohesion of H-specimens at OMC condition is about 132 kN/m^2 and after soaking, it reduces to 15 kN/m^2. The remoulded V-specimens at OMC has a cohesion of about 155 kN/m^2 and after soaking it reduces to 10 kN/m2. The angles of shearing resistance of remoulded H and V-specimens in unsoaked condition were 27.3° and 30°, after soaking they reduced to 26.5° and

TABLE 4
Results of direct shear test on remoulded H-specimens in OMC and soaked conditions

Set No	Specimen No	Normal Stress kN/m^2	Maximum Shear Stress kN/m^2	Displacement at τ_{max} mm	Cohesion c' kN/m^2	Angle ϕ' degree
	RH-O1a	50	157.2	4		
a	RH-O2a	102	198.3	4	138.8	26.1
	RH-O3a	205	236.4	4.25		
	RH-O1b	50	166.1	4.25		
b	RH-O2b	102	201.6	3.75	144.3	26.8
	RH-O3b	205	246	3.75		
	RH-O1c	50	144.2	4.5		
c	RH-O2c	102	192.8	3.75	113.3	35
	RH-O3c	205	256.6	4.5		
	RH-O1a	50	35.4	4		
a	RH-O2a	102	68.6	4	12.9	26.7
	RH-O3a	207	115	3.5		
	RH-O1b	50	38.05	3.5		
b	RH-O2b	102	69.6	5	103	26.4
	RH-O3b	207	116.5	4		
	RH-O1c	50	38.9	4		
c	RH-O2c	102	69.6	6	15.9	26.3
	RH-O3c'	207	116.9	5		

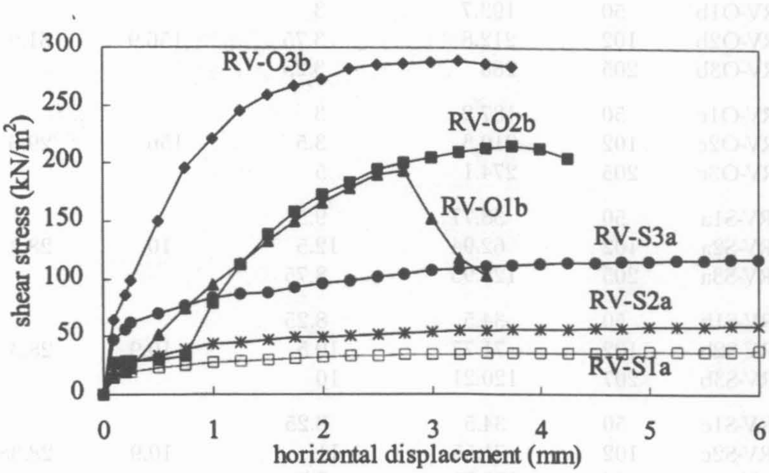

Fig 9. Relation between shear stress and horizontal displacement of remoulded V-specimens for CD test in OMC and soaked conditions

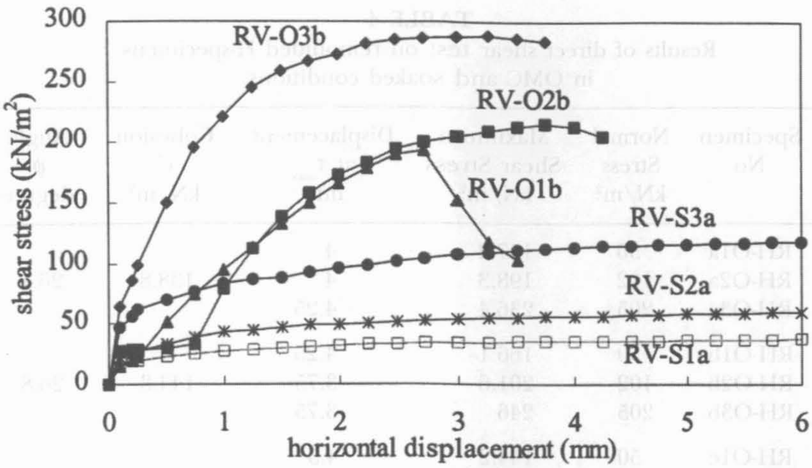

Fig 10. Relation between vertical and horizontal displacement of remoulded V-specimens for CD test in OMC and soaked conditions

TABLE 5
Results of direct shear test on remoulded V-specimen in OMC and soaked conditions

Set No	Specimen No	Normal Stress kN/m^2	Maximum Shear Stress kN/m^2	Displacement at τ_{max} mm	Cohesion c' kN/m^2	Angle ϕ' degree
	RV-O1a	50	197.8	2		
a	RV-O2a	102	205.3	4.25	154.3	33.1
	RV-O3a	205	294.1	2.75		
	RV-O1b	50	193.7	3		
b	RV-O2b	102	212.8	3.75	156.9	31.9
	RV-O3b	205	288	3.25		
	RV-O1c	50	187.2	3		
c	RV-O2c	102	210.3	3.5	156	29.6
	RV-O3c	205	274.1	5		
	RV-S1a	50	38.77	9.5		
a	RV-S2a	102	62.94	12.5	10	28.4
	RV-S3a	205	121.93	8.75		
	RV-S1b	50	34.5	8.25		
b	RV-S2b	102	71.77	10.5	10.9	28.3
	RV-S3b	207	120.21	10		
	RV-S1c	50	34.5	8.25		
c	RV-S2c	102	71.55	11	10.9	28.38
	RV-S3c	205	120.21	9.5		

27°, respectively. Angles of shearing resistance for remoulded H and V-specimens decrease 10% due to soaking. This is in line with Onitsuka *et al.* (1985) who pointed out that the angles of shearing resistance for compacted residual granite soil reduce 20% due to soaking. The main observation here is that significant reduction of cohesion results upon wetting.

Fig. 11 also shows the shear strength envelopes of remoulded H and V-specimens in both soaked and unsoaked conditions. For remoulded H-specimens, the corresponding strength anisotropy ratio, i.e. the ratio of shear stress at failure for V- specimens, $(\tau_f)_V$ to the shear stress at failure for H-specimens, $(t_f)_H$, was found to be between 1.1 to 1.2 for OMC condition and 1.0 to 1.1 for soaked condition. For remoulded soils, Onitsuka *et al.* (1985) also found that the corresponding strength anisotropy ratio varied between 1.1~1.2.

Effect of Shear Rate on Shear Strength

Tests were also carried out to evaluate the effects of shear rate on shear strength. In these tests the specimens were subjected to five shear rates. The time of failure

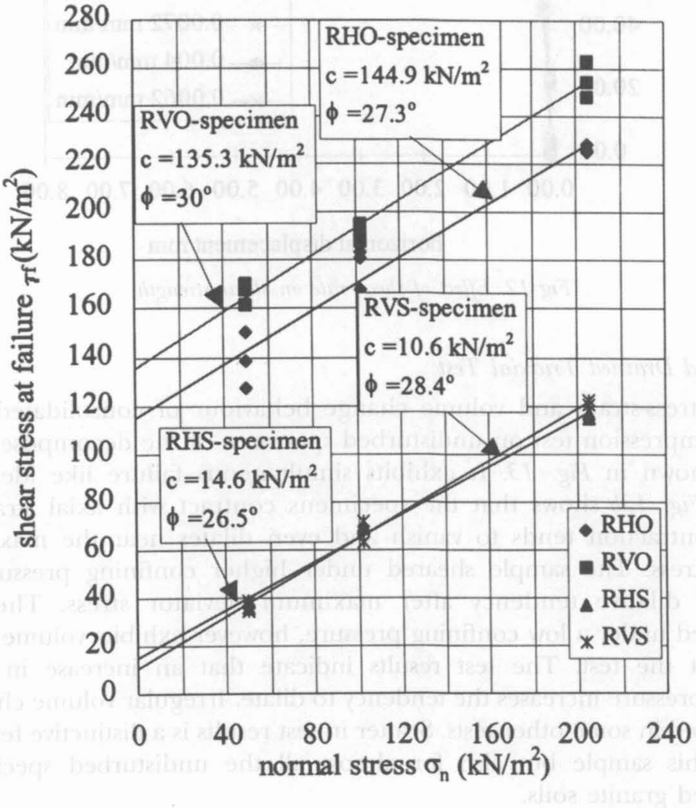

Fig 11. Shearing envelope of remoulded H and V-specimens in OMC and soaked condition

for fast shear rate (~0.6 mm/min) was less than half an hour, three hours for medium rate (~0.06 mm/min) and slow tests (~0.0072 mm/min) took 20 to 25 hours. No significant differences in the measured shear strengths were noted when the shear rate varied from 0.0072 to 0.4095 mm/min (*Fig. 12*). These results are similar to the results obtained by Cheung *et al.* (1988).

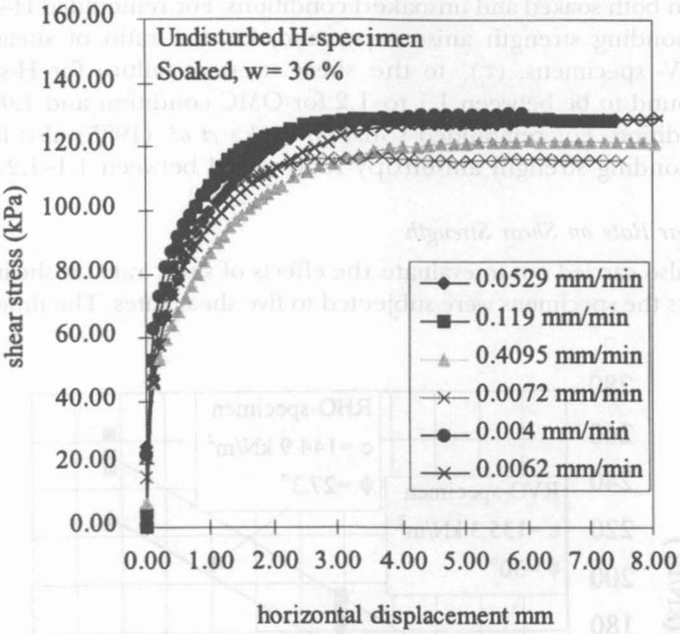

Fig 12. Effect of shear rate on shear strength

Consolidated Drained Triaxial Test

Deviator stress-strain and volume change behaviour of consolidated drained triaxial compression test on undisturbed specimen of the decomposed granite soil are shown in *Fig. 13*. It exhibits simultaneous failure like ideal plastic material. *Fig. 13b* shows that the specimens contract with axial strains. The volume contraction tends to vanish and even dilates near the maximum of deviator stress. The sample sheared under higher confining pressure shows significant dilative tendency after maximum deviator stress. The sample consolidated under a low confining pressure, however exhibits volume decrease throughout the test. The test results indicate that an increase in effective confining pressure increases the tendency to dilate. Irregular volume change was also observed in some other tests. Scatter in test results is a distinctive feature not only for this sample but also for almost all the undisturbed specimens of decomposed granite soils.

The Mohr stress circles at failure and the strength parameters for consolidated drained test of undisturbed decomposed granite soil are shown

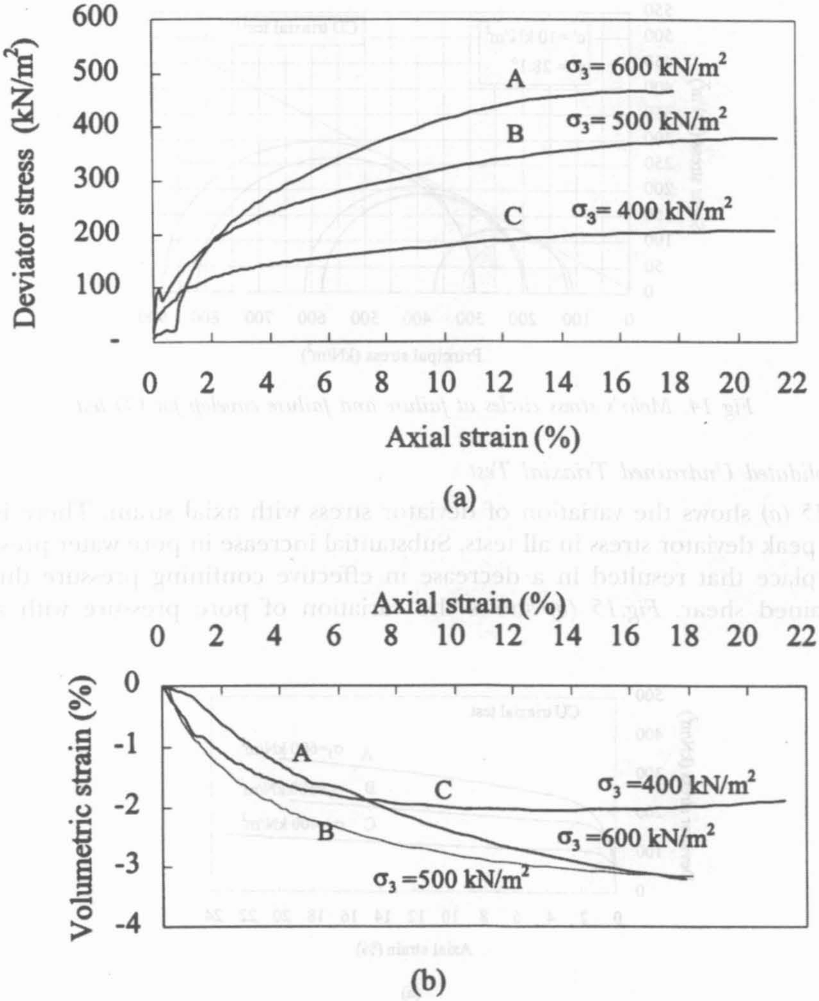

Fig 13. Results of consolidated drained triaxial test on undisturbed sample: (a) Deviator stress vs. axial strain (b) Volumetric strain vs. axial strain

in *Fig. 14.* The failure envelope can be best represented by a straight line. The cohesion intercept is 10 kPa and the internal frictional angle is 28.1°. Studies conducted by Ting and Ooi (1976) revealed a similar angle of internal friction but much higher cohesion intercept. It is well established that strength behaviour is highly dependent on the previous strain history of the soil structure, mode of imposed shear and the drainage condition (Terzaghi *et al.* 1996). The latter two factors could be said to be the same, thus leaving the soil history as the major difference between the current study and that of Ting and Ooi (1976). It is possible that the differences were due to differences in sampling location and depth, and also the heterogeneity of the residual soil.

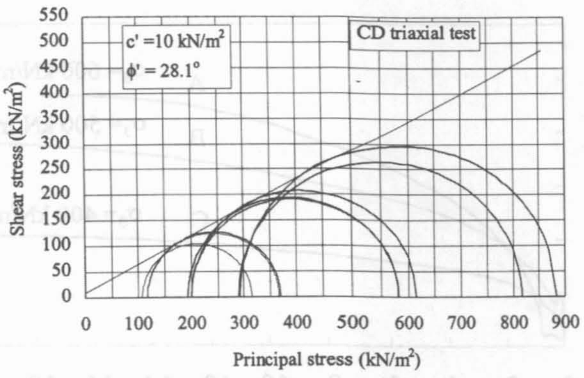

Fig 14. Mohr's stress circles at failure and failure envelop for CD test

Consolidated Undrained Triaxial Test

Fig. 15 (a) shows the variation of deviator stress with axial strain. There is no clear peak deviator stress in all tests. Substantial increase in pore water pressure took place that resulted in a decrease in effective confining pressure during undrained shear. *Fig.15 (b)* shows the variation of pore pressure with axial

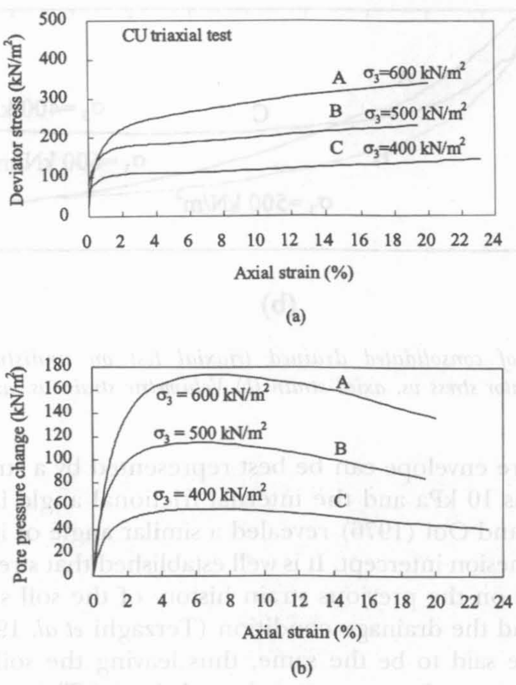

Fig 15. Results of consolidated drained triaxial tests on undisturbed sample: (a) Deviator stress vs. axial strain (b) Volumetric strain vs. axial strain

strain. The Mohr's circle at failure and the strength parameters for CU test are shown in *Fig. 16*. The failure envelope is assumed to be straight line. The undisturbed sample has a cohesion intercept of 15 kPa and an internal friction angle of 30.9°.

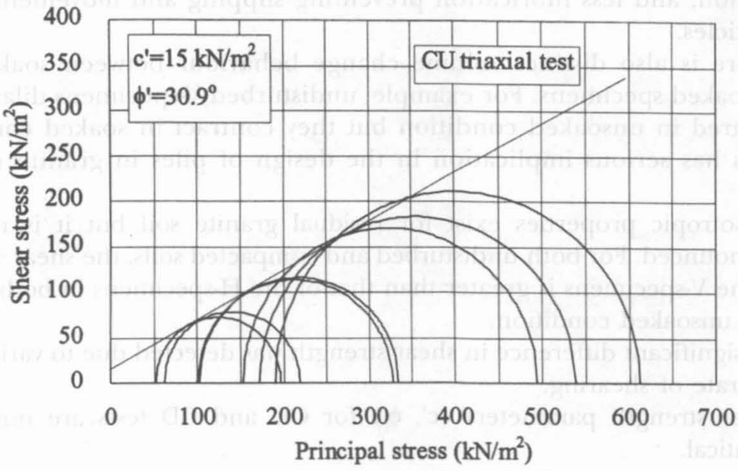

Fig 16. Mohr's stress circles at failure and failure envelope for CU test

Theoretically, the effective shear strength parameters (c', ϕ') of normally consolidated clays obtained from consolidated undrained tests with pore pressure measurement and from drained tests are identical. On the other hand, for heavily over-consolidated clays and dense sand, the drained test will lead to slightly higher values of the shear strength parameters due to work done by the increase in volume of the sample during shear and to the smaller strain at failure (Bishop and Henkel, 1976). Practically, the shear strength parameters (c', ϕ') for CD and CU tests are not identical because of the different nature of the two types of test. Similar behaviour was also shown by Ting and Ooi (1976) for a residual granite soil.

CONCLUSION

The shear strength characteristics and the factors influencing strength parameters of residual granite soil were investigated and presented in this paper. In order to study the anisotropic strength properties of residual granite soil, direct shear test on H and V-specimens were carried out in both soaked and unsoaked conditions. The H and V- specimens differ in terms of the orientation of sample extruding and it also represents perpendicular failure plane to one another. Direct shear tests were conducted on undisturbed and compacted specimens. Drained and undrained triaxial tests were also included in the testing programme.

Amongst the main conclusions that may be derived from this study are listed as follows:

- No peak behaviour was observed in both direct shear and triaxial tests.
- The strength of unsoaked soil is approximately 10 times greater than that of soaked soil specimens. This may be attributable to the lack of suction, and less lubrication preventing slipping and movement of soil particles.
- There is also distinct volume change behaviour between soaked and unsoaked specimens. For example, undisturbed V-specimens dilate when sheared in unsoaked condition but they contract in soaked condition. This has serious implication in the design of piles in granite residual soil.
- Anisotropic properties exist for residual granite soil but it is not very pronounced. For both undisturbed and compacted soils, the shear strength of the V-specimens is greater than that of the H-specimens in both soaked and unsoaked condition.
- No significant difference in shear strength was detected due to variation in the rate of shearing.
- Shear strength parameters (c', ϕ') for CU and CD tests are not found identical.

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