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High Pressure Triaxial Testing of Desiccated Keswick Clay

W.S. Kaggwa
BSc.(Eng), MEngSc., PhD., MIEAust.
Senior Lecturer, The University of Adelaide, Australia
T.G. Ossa
B.Sc.Engg.
Postgraduate Student, University of Adelaide, Australia

Summary In many parts of Australia, clay soils occur in a moisture-deficient, or partially saturated, state caused by a deep groundwater table, where precipitation is less than evapotranspiration for a significant part of the year. These soils undergo seasonal vertical movements due to changes in moisture content, and when foundations are constructed, they are susceptible to seasonal movements. The shear strength and stiffness behaviour depend on a combination of changes in external loading as well as changes in moisture within the soil.

Laboratory tests on undisturbed desiccated Keswick Clay were undertaken using high pressure triaxial tests in order to determine the effective stress state by overcoming the high negative pore pressures within the soil. The test results, which included measurement of the salinity of the pore water, are described in this paper. The initial stress state of Keswick Clay is described in terms of equivalent effective stress state, which, when used as the basis of normalisation, gives normalised shear strength and stiffness values comparable to those obtained for saturated cohesive soils.

1. INTRODUCTION

Stiff clays are to be found where the clay soil has been subjected to external, or internal, stresses that are higher than the present stress state - that is, the soil is overconsolidated. External stresses arise from high overburden stresses that have since been removed. Internal stresses may be caused by desiccation, where the soil has been subjected to repeated cycles of wetting and drying. Under these conditions, the soil has high shear strength and stiffness when dry, but loses these upon saturation. It is therefore important to understand the fundamental cause of the measured high values because in some instances, significant reductions in strength or stiffness may occur when moisture and loading conditions change.

Semi-arid and arid regions characteristically have deep groundwater tables, and because of the high rates of evaporation, soils exist in an unsaturated state. During the dry season, evaporation at the ground surface sets up a moisture gradient in fine grained soils where moisture is drawn from the deeper, more moist soils to the near surface, drier soils. In the wet season, the gradient is reversed and moisture is drawn into the soil from the saturated zone near ground surface. Thus, changes in the moisture conditions at the ground surface induce cyclic wetting and drying, which can go down to 4 or 5 metres below the ground surface.

2. BACKGROUND

At the particulate level, clays are plate-like in shape and possess negative electrical charges on their faces and positive electrical charges along their edges. These electrical charges are due to isomorphous substitution of silicon and aluminium in the basic molecular unit, when replaced by atoms of lower valency.

The pore fluid within the inter-aggregate and intra-aggregate pore spaces consists of water molecules and dissolved salt ions, whereas air is present in the inter-aggregate pores. Adsorbed layers of salt cations form on the clay surfaces that are in a state of dynamic equilibrium with the negatively charged clay surface that attracts the cations present in the porewater. The like-charged anions are repelled. At the same time, the cations tend to move away from each other because they carry similar charges as well as their physical size when hydrated. The adsorbed layer is also known as the double layer, or dispersed layer.

The net effect is that the cations form an adsorbed layer adjacent to the clay particle, with a concentration that decreases with increasing distance from the clay surface until the concentration becomes equal to that in the free water in the void space. Quirk (1963) developed the equilibrium conditions in terms of the variation of cation and anion concentration with distance from the particle surface as shown in Figure 1. Thus, any change in the salt concentration of the porewater will result in a change in the thickness of the double layer. To achieve this, energy will be added or removed from the soil, and the measure of this energy constitutes solute suction.

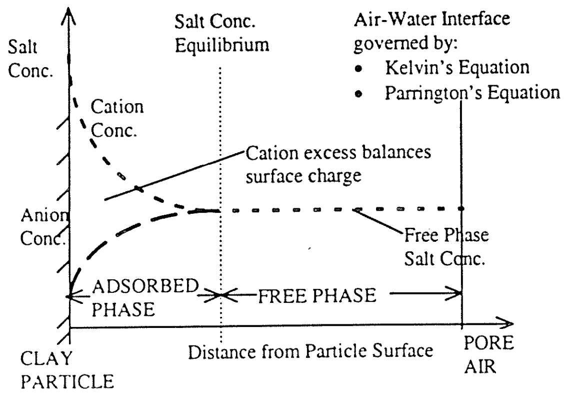


Figure 1. Equilibrium conditions for an expansive clay porewater system with salts present (after Quirk, 1963)

The presence of air in the voids, which is open to the atmosphere, means that the pressure of the porewater is negative, and is referred to as matrix suction. Total suction, which is a measure of energy relative to atmospheric pressure, thus, is the sum of matrix and solute suction.

Application of external load influences the equilibrium state within the porewater, and causes a reduction in the thickness of the adsorbed layer, in addition to particle movements. Thus the observations at the macro level are influenced by the particulate-microstructural behaviour, particularly in the long term.

3. EXPERIMENTAL PROGRAMME

The experimental work involved laboratory testing of undisturbed soil samples of Keswick Clay recovered from Light Square in Adelaide, South Australia. Figure 2 shows a location map of Adelaide, with Light Square located to the North-West of Adelaide as marked on the figure. The soil samples were obtained by pushing thin sampling tubes, 70 mm internal diameter and 500 mm long. The sampling tube was pulled out and quickly sealed using standard rubber seals, and an additional plastic tube cover was screwed on to the ends. The tubes were stored in a controlled temperature and humidity room (21±0.5°C and 100% humidity) until the soil was required for testing.

Total suction tests were undertaken before and after triaxial tests in order to determine the equivalent effective stress state of the soil, the effect of using distilled water as the pore fluid, and the effects of changes in effective stress, (increasing the consolidation pressure).

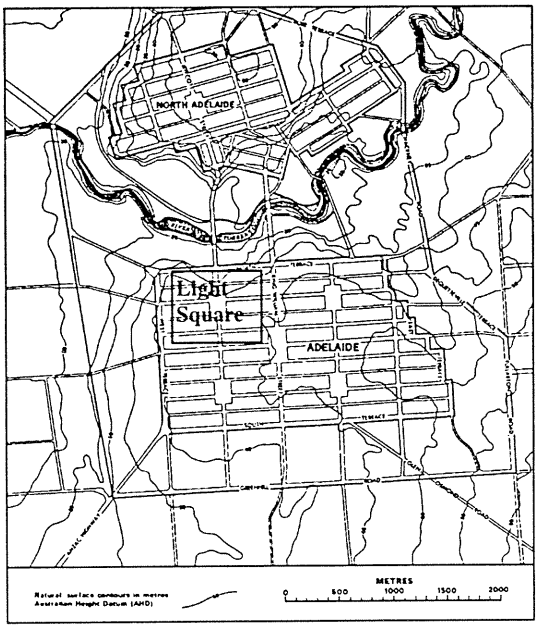


Figure 2. Location map of Adelaide, showing Light Square.

Triaxial tests were carried out on 50 mm diameter samples. These samples were obtained by pushing a 50 mm diameter thin wall tube while extruding the soil from the 70 mm sampling tube. Unconsolidated undrained (UU) triaxial tests were undertaken at "low" confining pressures (where the applied pressures were lower than the in situ total suction) as well as "high" confining pressures (with confining pressures higher than the in-situ total suction). Isotropically consolidated undrained (CIU) triaxial tests were also carried out at pressures that exceeded the equivalent effective stress state. The total suction of the soil was measured using a commercially available transistor psychrometer (SMI, 1995).

A total of 21 samples were tested using distilled water in the drainage line. Of these, 6 samples were tested in "low" pressure UU triaxial tests, 7 samples in "high" pressure UU tests, and 8 samples under CIU conditions. An additional 2 samples were tested using pore water that was collected at the end of the triaxial tests from the back pressure line. Table 1 summarises the confining pressures applied to the soil samples.

Table 1. Summary of triaxial test conditions.

Sample No.	Initial Total Suction (kPa)	Confining or Consolidation Pressure (kPa)
"Low" Pressure Unconsolidated (UU) tests		
D5-1	1785	80
D5-2	1870	1000
D5-3	1870	600
D6-1	1420	54
D6-2	2030	100
D6-3	1665	1000
"High" Pressure Unconsolidated (UU) tests		
C5-2	1785	1790
C5-3	1745	3000
C5-4	2005	2020
C5-5	1915	4000
C6-6	1785	5000
C5-7	1870	1890
C7-8	1610	4500
Consolidated Undrained (CIU) tests		
B5-0	1325	1045
B5-1	1325	2045
B5-2	1745	2045
B5-3	1870	3045
B6-4	1520	4060
B6-5	1520	3550
B6-6	1400	1015
B3-7	1825	765

4. RESULTS

4.1 Index Properties of Keswick Clay

The results of index tests (Liquid Limit, LL; Plasticity Index, PI; specific gravity of solids, G_s; and clay content) are summarised in Table 2. The results are comparable to those reported in earlier investigations (eg. Cox, 1970; Islam, 1994), and within the ranges reported by Jaksa and Kaggwa (1992).

Table 2. Summary of results of index tests on Keswick Clay.

Study	*LL*	*PI*	G_s	% of Clay
Cox (1970)	75-115	55-80	2.70	65
Islam (1994)	76.5	49.5	2.73	-
This Study	75	51	2.72	70

4.2 Results of Total Suction Tests

The in-situ moisture content varied between 29% and 35% depending on the depth of the sample and the time of sampling. For each total suction measurement, three samples were used and the average value determined. The total suction varied between 1200 kPa and 2100 kPa, again depending on the depth of the sample and the time of sampling. Figure 3 shows the suction profiles along three boreholes, BH 5, 6 and 7, taken within an area 5 metres in diameter, with BH 6 in the centre.

The total suction measurements are summarised in Table 3. There were slight variations in the range of values of total suction measured from in situ samples as well as in tests after consolidated undrained triaxial tests, within the precision of the transistor psychrometer used. However, the scatter of results from UU tests was comparatively high as seen in Table 3. This suggests that solute suction gradients were still present although there were no further significant changes in the pore pressure readings. Thus, the suction measurements at the end of UU tests should be treated with caution.

Table 3. Summary of total suction measurements.

Sample No.	Initial Total Suction	Final Total Suction	
	Average (kPa)	Average (kPa)	Range (kPa)
"Low" Pressure Unconsolidated (UU) tests			
D5-1	1785	1665	1385 - 1915
D5-2	1870	1555	1485 - 1630
D5-3	1870	1485	1355 - 1745
D6-1	1420	1325	1150 - 1450
D6-2	2030	1630	1485 - 1915
D6-3	1665	1520	1355-1915
"High" Pressure Unconsolidated (UU) tests			
C5-2	1785	1450	1320 - 1705
C5-3	1745	2245	2145 - 2410
C5-4	2005	1870	1705 - 2145
C5-5	1915	1955	1555 - 2095
C6-6	1785	2050	1825 - 2465
C5-7	1870	1555	1385 - 1785
C7-8	1610	1560	1455 - 1725
Consolidated Undrained (CIU) tests			
B5-0	1325	1030	935 - 1150
B5-1	1325	2100	1870 - 2100
B5-2	1745	2700	2300 - 2900
B5-3	1870	3030	2830 - 3175
B6-4	1520	3730	3325 - 4200
B6-5	1520	3400	3325 - 3400
B6-6	1400	1955	1825 - 2000
B3-7	1825	1915	1485 - 2250

4.3 Results of Unconsolidated Undrained Triaxial Tests

The variations of the deviator stress with axial strain are shown in Figure 4 for UU tests. It can be seen from Figure 4 that there was no clear peak-residual behaviour except for one sample.

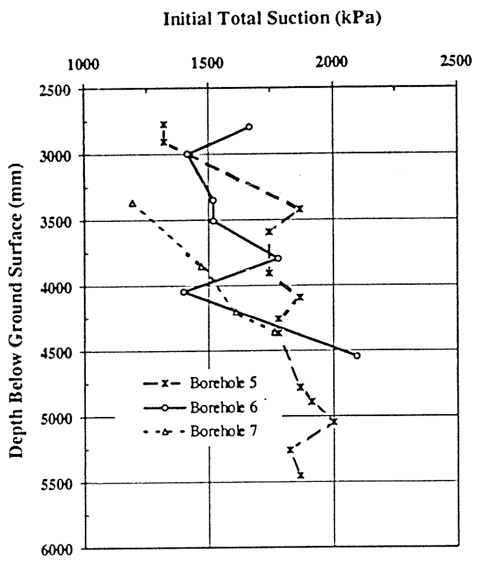


Figure 3. Variation of total suction with depth.

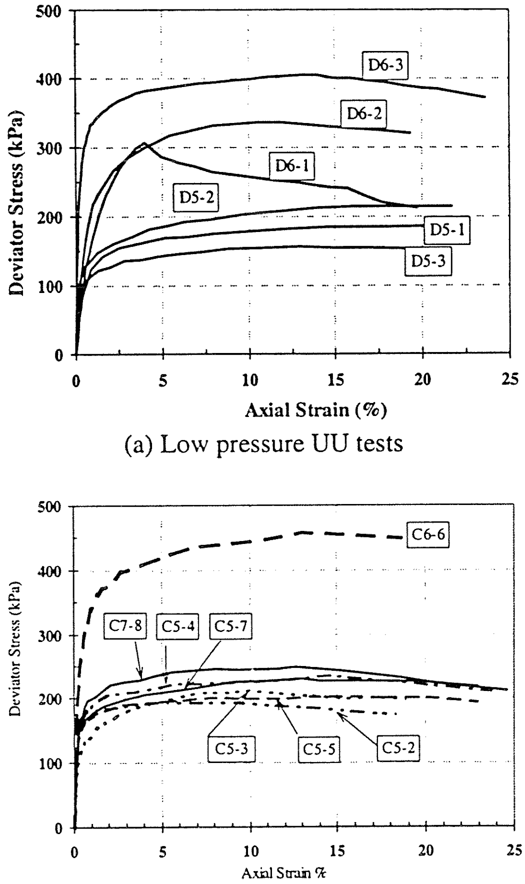
For "low" pressure tests, the peak and residual deviator stress varied between 150 kPa and 400 kPa. For "high" pressure tests, the peak deviator stress was 180 to 250 kPa for most of the samples, with only one sample having a peak deviator stress greater than 300 kPa. These results are comparable to the test results of triaxial tests under normal pressures that were reported by Kaggwa and Jaksa (1992).

It would appear that the level of confining pressure had little effect on the undrained shear strength of Keswick Clay. This is true because the degree of saturation was high (higher than 96%).

Figure 5 shows the variation of undrained shear strength (from UU tests) with depth. It can be seen that the undrained shear strength decreases, up to a depth of 4 metres, followed by a slight increase with depth. The results fall within the range reported by Kaggwa and Jaksa (1992) and Jaksa (1996).

This trend is characteristic of high overconsolidation ratio near the ground surface, gradually decreasing to normal consolidation conditions at depth. The trend is consistent with the effects of desiccation, being more pronounced near the ground surface than at depth.

The use of high confining pressures in UU tests allowed determination of effective stress paths, and these are shown in Figure 6, in normalised form. The shapes of the stress paths are what one would expect for overconsolidated samples, and they are similar irrespective of whether low or high confining pressures were used.



(a) Low pressure UU tests

(b) High pressure UU tests

Figure 4. Deviator stress versus axial strain in UU tests.

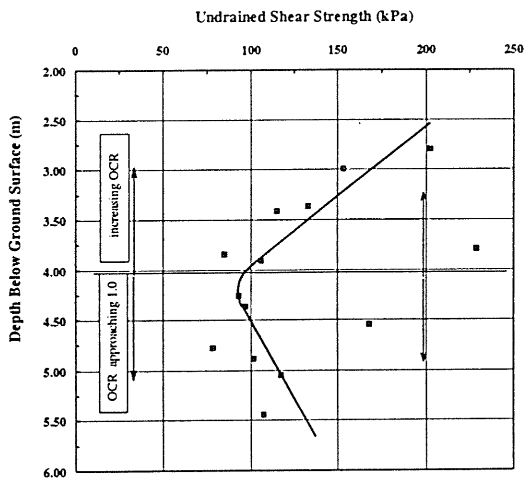
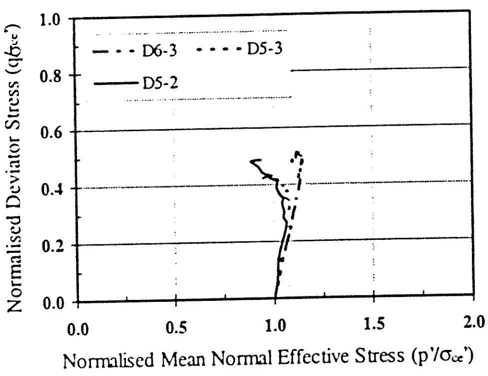
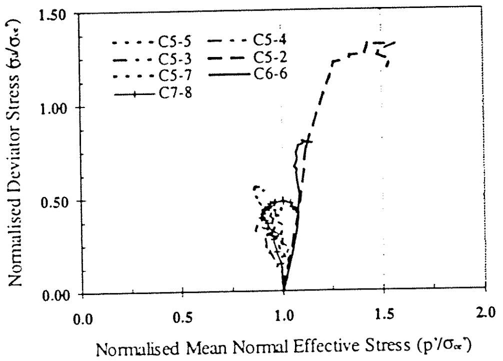


Figure 5. Undrained shear strength profile.



(a) Low pressure UU tests



(b) "High" pressure UU tests

Figure 6. Normalised effective stress paths from UU tests.

In the majority of samples, there was no gain in shear strength once the failure surface was reached. For the soil samples that showed high shear strength (greater than 300 kPa), this is associated with a tendency for the samples to dilate (decrease in excess pore pressure results in right side shift of the stress path).

4.4 Results of Isotropically Consolidated Undrained Triaxial Tests

The results of consolidated undrained triaxial tests on Keswick Clay are summarised in Figure 7.

Figure 7 shows that some samples exhibited clear peak-residual response. There was an increase in the undrained shear strength and stiffness as the consolidation pressure increases. Figure 7 is replotted in Figure 8 with the deviator stress normalised by the consolidation pressure.

Figure 8 shows that, similar to other clays, the stiffness of Keswick Clay is directly proportional to the consolidation pressure for the normally consolidated samples tested under CIU conditions. There also appear to be two distinctive values of normalised residual shear strength, implying variations in the macro-fabric structure of Keswick Clay.

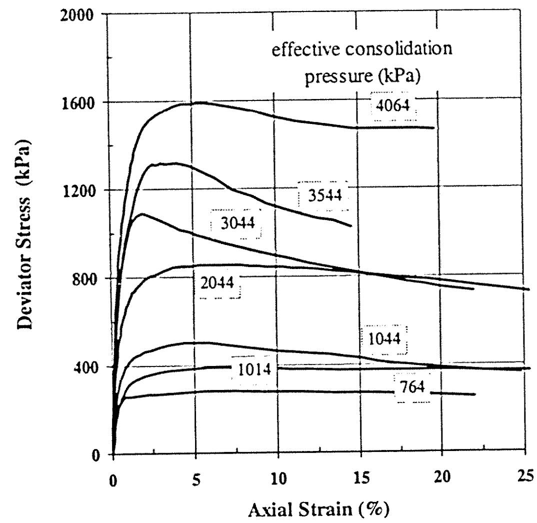


Figure 7. Deviator stress versus axial strain response in CIU tests.

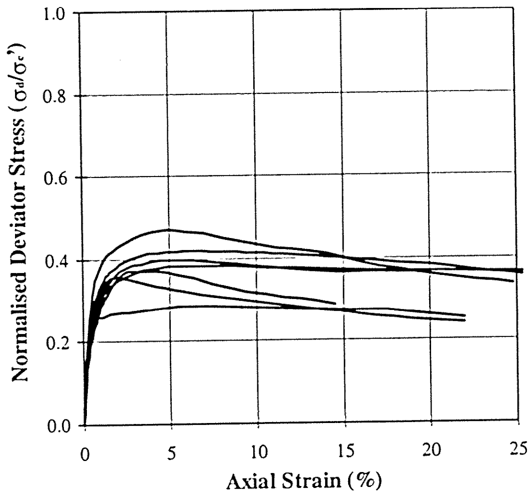


Figure 8. Normalised deviator stress versus axial strain from CIU tests.

5. ANALYSIS OF HIGH PRESSURE TRIAXIAL TEST RESULTS

5.1 Equivalent Effective Stress Based on Total Suction

The equivalent effective stress, p_e', is defined here as the difference between the applied confining pressure and the equilibrium pore pressure developed within the soil sample. In the present context the equivalent effective stress state is influenced by the in situ soil conditions, particularly the degree of saturation and total suction. The equivalent effective stress is plotted against the initial total suction, ψ_i, in Figure 9. It can be seen that there is wide scatter in the results, with no definite trend. The scatter is possibly due to the inherent variability of the soil, degree of saturation, and the overconsolidated state of the stiff soil.

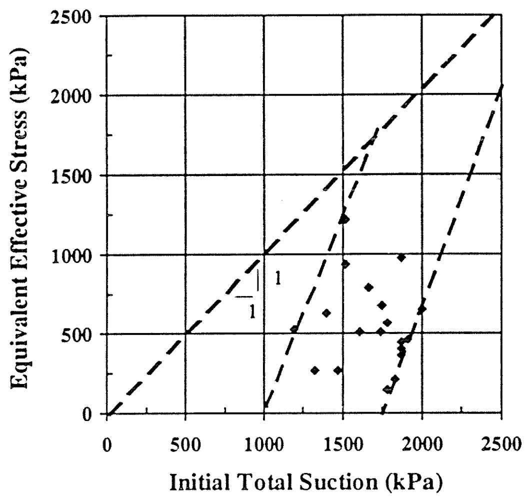


Figure 9. Variation of equivalent effective stress with initial total suction.

Figure 10 shows the variations, with depth below ground surface, of the equivalent effective stress, p_e' and initial total suction, φ_i. It can be seen that there is a wide scatter for depths between 2.5 and 4 m. For depths below 4 metres, an increase in total suction is matched by a corresponding increase in equivalent effective stress. However, no definite conclusions can be made from the available data.

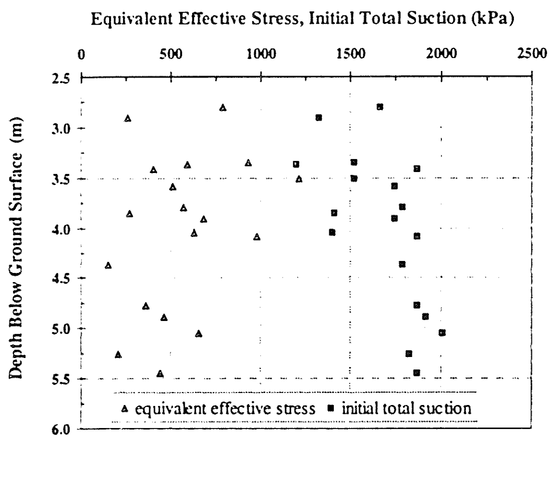


Figure 10. Variation of equivalent effective stress and initial total suction with depth.

The variation of total suction with consolidation pressure is shown in Figure 11. A linear increase in total suction with increasing consolidation pressure is apparent. This is to be expected because consolidation results in expulsion of pore water from the sample (reduction in void ratio). Thus, measurement of total suction after the test reflects the lower void ratio.

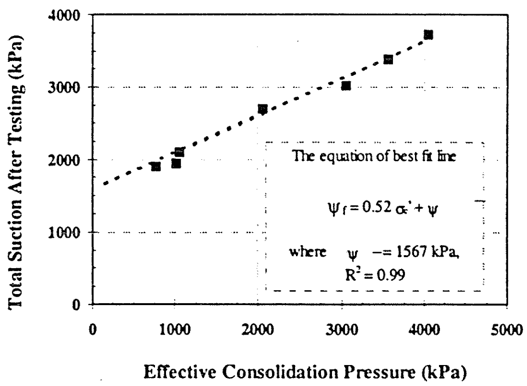


Figure 11. Variation of total suction with effective consolidation pressure

5.2 Shear Strength Characteristics at High Pressure

The variation of undrained shear strength with consolidation pressure, for CIU tests, is shown in Figure 12. The consolidation pressures were such that the samples were normally consolidated. The undrained shear strength ratio, (s_u/σ'_c) = 0.18 to 0.20. At failure, the deviator stress, q, is related to the mean normal effective stress, p', by:

$$q = Mp' \qquad (1)$$

where M is the slope of the line of best fit through the origin in the p' - q stress space. M = 0.34 to 0.42, corresponding to an effective friction angle, ϕ' in the range 9° to 11°. This range should be compared to $\phi' = 8^\circ$ reported by Islam (1994) for normally consolidated, remoulded Keswick Clay.

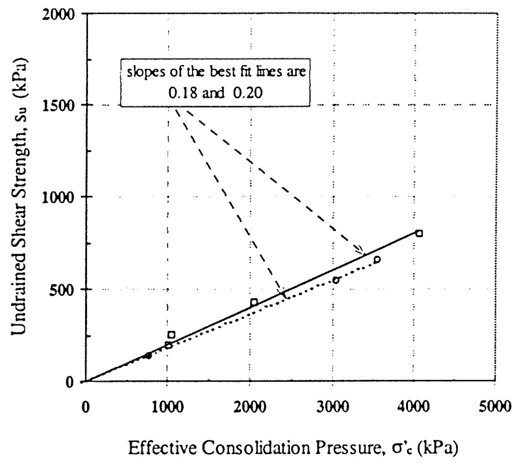


Figure 12. Variation of undrained shear strength with consolidation pressure.

The friction angle is significantly lower than the typical ranges reported elsewhere. Based on Wroth (1984) in his Rankine lecture, the friction angle would be expected to be close to 20° on the basis of the undrained shear strength ratio (if one assumes soil parameters r and λ are 2 and 0.8, respectively). The fact that the soil parameters are different for other clays (those suggested by Wroth were derived from tests on remoulded Kaolin) appears not to be appreciated by many researchers. It is possibly due to differences in moisture conditions (and clay mineralogy) in the northern hemisphere where the water table is at shallow depth compared to Australia's drier environment where clays have been subjected to desiccation.

5.3 Compressibility of Keswick Clay at High Pressure

The undrained Young's modulus corresponding to 25% stress level (25% of peak deviator stress), increased from E_{u25} = 100 MPa to 250 MPa (the ratio E_{u25}/σ'_c = 65 to 100). At the 50% deviator stress level, E_{u50} = 80 MPa to 200 MPa (the ratio E_{u50}/σ'_c = 50 to 80). In comparison, Islam reported values of the ratio $E_{u25}/\sigma'_c = E_{u50}/\sigma'_c = 50$ based on results of triaxial tests on normally consolidated, remoulded Keswick Clay.

The values obtained from UU tests range from 40 MPa to 110 MPa for E_{u25}, (at 25% of peak deviator stress) and 25 MPa to 80 MPa, for E_{u50}. These values are comparable to the values reported by Kaggwa and Jaksa (1992).

These normalised values are comparable to the data obtained by Islam (1994) for overconsolidated samples of remoulded Keswick Clay· However, the values reported by Islam were higher than the ratios reported in the literature for other clays (eg. Ladd et al. 1977).

6. SUMMARY AND CONCLUSIONS

Considering available results, the following conclusions can be made with respect to the effectiveness of high pressure testing.

- The use of high pressures in triaxial tests, to raise the porewater pressure and permit the determination of effective stress parameters, does not appear to significantly alter the behaviour of Keswick Clay.
- High pressure triaxial tests, where the confining pressure exceeds the in situ total suction, are the only way of determining the effective stress parameters for the undisturbed Keswick Clay.
- The measurement of total suction, before and after triaxial testing, allows for the examination of the effects of the test conditions, and provides a better understanding of the relationship between consolidation stress and total suction.

7. ACKNOWLEDGMENTS

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