Normalised Shear Strength and Compressibility Characteristics of Adelaide Expansive Clay

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SUMMARY The available data on the undrained shear strength and compressibility of Adelaide clays is presented, particularly for the Keswick Clay. Analyses which take into account the soil suction in the soil mass (in-situ negative pore pressures) are used to relate the undrained shear strength and Young's modulus determined from field and laboratory tests to the effective stress state in the soil. It is shown that in spite of the wide variations in the values of undrained shear strength and compressibility characteristics of Keswick Clay, which are attributable to the variable effective stress state within the soil mass and also the inherent variability of the clay, the normalised values fall within ranges typical of over-consolidated clays acted upon by positive pore pressures.

1. INTRODUCTION

Expansive clay soils occur extensively in the Adelaide metropolitan area and surrounding suburbs, as shown in Fig.1. The clays are typically stiff and appear to be over-consolidated, with high values of total suction in situ. Extensive data is available on the index properties of the clays, natural moisture content including seasonal variations, soil suction and their shrink-swell characteristics. However, less data is available on the shear strength and compressibility characteristics of these clays. In addition, this data is mainly for undrained or short-term conditions.

The authors have assembled a large data base on the shear strength and compressibility of the expansive Keswick and Hindmarsh Clays from the Adelaide metropolitan area, the in situ moisture content, dry density, and total soil suction. This paper analyses the data for Keswick Clay, determined from field and laboratory tests, in order to derive common variations with depth and the influence of the effective stress state. The data base is, however, not exhaustive, and the data for the sand layer is not included in this paper.

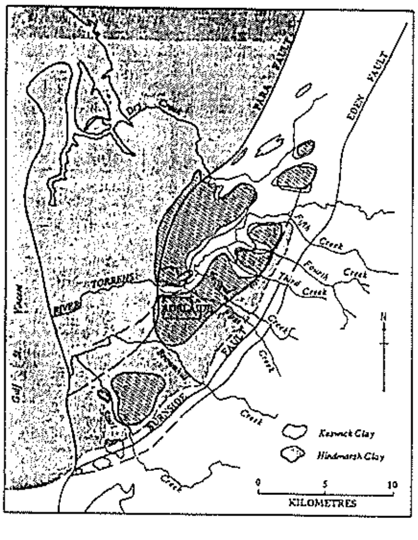

Fig. 1: Distribution of expansive clays around Adelaide

2. DESCRIPTION AND PROPERTIES OF EXPANSIVE CLAYS

A typical soil profile in the Adelaide metropolitan area consists of fill or recent deposits up to 2 metres, underlain by a layer of a highly expansive clay known as Keswick Clay, of variable thickness (typically 5 to 8 metres thick). There is a sand layer, typically 4 to 6 metres underneath, followed by another expansive clay layer, known as Hindmarsh Clay, which can be up to 5 metres in thickness. This clay layer is underlain by Hallet Cove Sandstone. The general water table is located within this sandstone layer. Occasional perched water tables are found in the Keswick Clay layer. The investigations at the South Australian Department of Mines and Energy by Sheard and Bowman (1987a,b) provide clear definitions of the two clay layers. The analyses reported in the present paper are limited to the upper Keswick Clay, with indicative data for the Hindmarsh Clay included wherever possible.

Generally, the particle size distribution curves show that average percentage of clay, silt, and sand are 65%, 20% and 15%, respectively. The clay soils have been described as highly plastic, reactive, and over-consolidated with a classification CH according to the Unified Soil Classification System. The predominant clay minerals are illite (50%) and kaolin (20%) and montmorillonite less than 20%. The specific gravity falls within the range $G_s = 2.68 - 2.77$. The in situ water content, dry density and degree of saturation are discussed in a companion paper by Jaksa and Kaggwa (1992) in the proceedings of this conference. Extensive data on the description and properties of the clay soils is available and the summary by Cox (1970) presents the basic information.

The variations of the dry density, moisture content and total soil suction with depth are shown in Fig.2. It can be seen that the dry density, moisture content and total soil suction all vary widely for a given depth below the ground surface. This is likely due to (a) the natural variability of geological materials in situ, (b) the highly fissured nature of the clay, resulting in moisture flow mainly along the fissures rather than through the entire soil mass, (c) the effects of sampling disturbance, and (d) the seasonal variations for samples taken closer to the ground surface. This large scatter in the soil

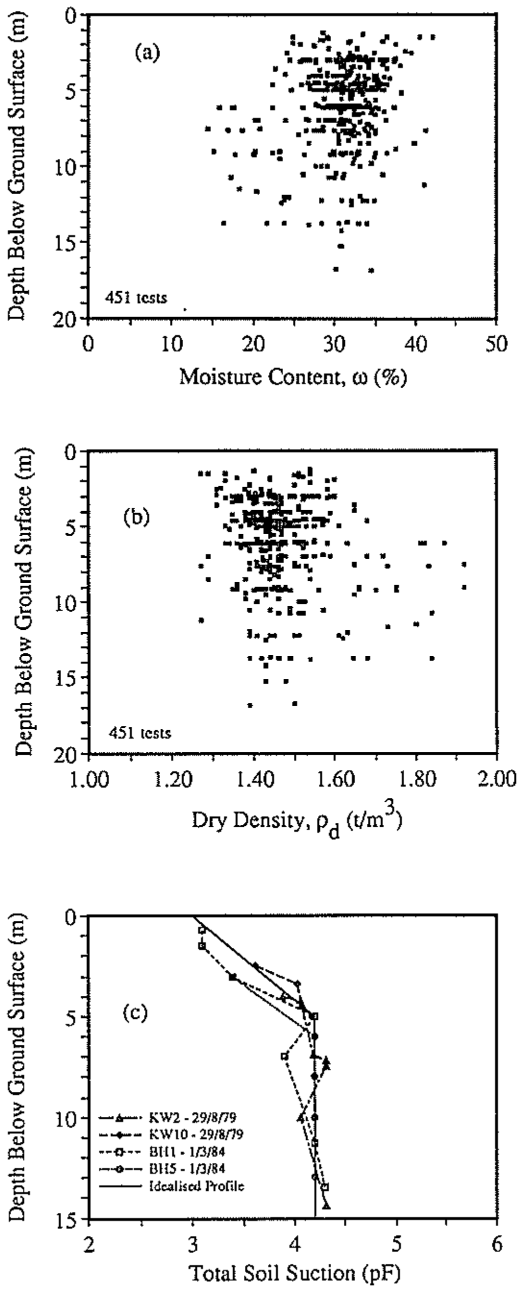

Fig. 2: Variation of moisture content, dry density and total soil suction with depth

state parameters has led many consulting geotechnical engineers to rely solely on field measurements relevant to the site under investigation.

Experience from investigations of the shear strength and compressibility of Adelaide clays has shown that for the case of residential buildings, shear strength and compressibility characteristics of the clays are not a controlling factor in the design of footings (see for example Pile and McInnes, 1984), but rather that the swelling and shrinkage of the soil due to moisture changes is the major factor. As a result, estimates of the shear strength and compressibility of the clays have been based on either past experience, results of pocket penetrometer or cone penetration tests, or unconsolidated undrained (UU) triaxial tests. Because of fissuring in the clays, laboratory tests have largely been used to confirm the results of field tests.

Detailed investigations have been carried out for large commercial building projects, with stratigraphy, field tests such as cone penetration, pressuremeter and screw plate load (SPLT) tests, and laboratory tests being performed. Isotropically consolidated triaxial tests using the in situ (total) stress, followed by shearing under undrained conditions (CIU), have been used to check the values estimated from field tests. In many UU tests, 3-stage shearing has been used, but there is uncertainty whether the excess pore pressures within the soil specimen dissipate before the commencement of the second and third shearing stages. The results of the in situ tests and laboratory tests show that at any given location, the undrained shear strength increases with depth for the upper Keswick Clay, with the ratio $(s_u/\sigma_{vo}) = 1.0$ to 1.5, where s_u is the undrained shear strength and σ_{vo} is the overburden pressure (total vertical stress). However, a slight decrease in undrained shear strength with depth has been obtained for the lower Hindmarsh Clay layer.

The compressibility of the clay samples has been determined from results of field tests especially plate loading, screw plate, and pressuremeter, tests. In the laboratory, the oedometer consolidation and shrink-swell tests have been widely used. It does not appear that the deviator stress versus axial strain response obtained from the triaxial test has been widely used to determine the value of Young's modulus under undrained or drained conditions. Thus, some of the data presented in this paper is based on analyses of the test results by the authors.

3. SUMMARY OF RESULTS FROM FIELD TESTS

In the field, the total horizontal stress is typically higher than the total vertical stress. The results of strength and compressibility measurements from field and laboratory tests are summarised in this section. It should be remembered that the computed in situ shear strength depends on the effective normal stresses within the soil mass and the drainage conditions during the test.

3.1 Undrained Shear Strength

Estimates of the undrained shear strength based on field tests including self-boring pressuremeter (SBPT) tests, cone penetration tests, and screw plate load (SPLT) tests, are shown in Fig. 3. The variation of undrained shear strength with depth below the ground surface for the three field test methods shows that the screw plate test results have the widest scatter, followed by the cone penetration test. The SBPT test results have the least scatter. The lowest undrained shear strength can be approximated by the ratio $(s_u/\gamma_w z) = 1.8$, where γ_w is the unit weight of water and z is the depth below ground surface. The highest values are given by the ratio $(s_u/\gamma_w z) = 5.0$, with a maximum undrained shear strength of 250 kPa for the Keswick Clay. The large variation in the undrained shear strength near the ground surface can be attributed to variations in moisture content (and total soil suction) caused by water infiltration during

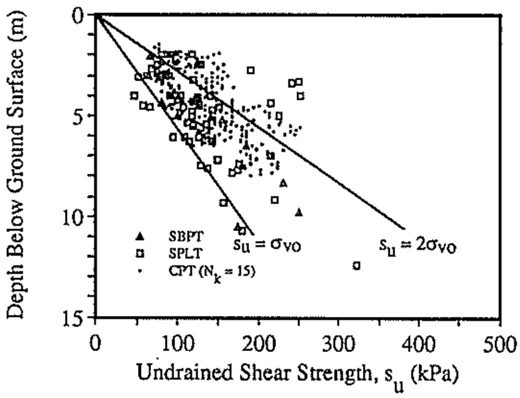

Fig. 3: Variation of field-determined shear strength with depth

winter, and evaporation during summer, with the active zone extending to a depth of about 5 metres.

3.2 Compressibility

The undrained Young's modulus has been estimated from the pressure versus radial strain during the pressuremeter test and the immediate load-settlement curve of the screw plate load tests, using the small strain linear portion of the curves. The variation of the initial tangent modulus, E_{ui}, with depth is shown in Fig. 4. As in the case of the undrained shear strength, the screw-plate test results show the most scatter. In general, the SBPT test results suggest that E_{ui} (horizontal) increases with depth whereas no general trend can be inferred from the screw-plate test results. The lower value of E_{ui} (horizontal) can be expressed by the ratio $(E_{ui}/\gamma_w z)$ = 300 whereas the upper value, which would be applicable to intact clay and clay at low moisture content, can be expressed by

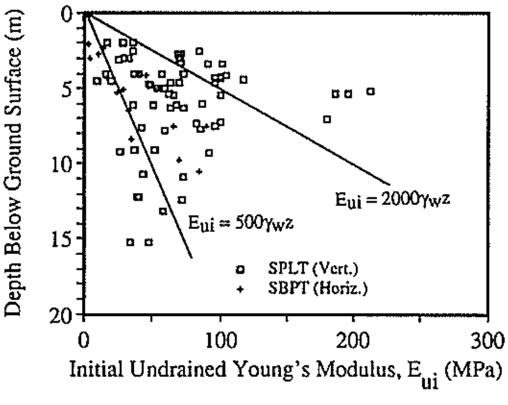

Fig. 4: Variation of field-determined undrained Young's modulus with depth

the ratio $(E_{ui}/\gamma_w z)$ = 2000. Thus, there is a variation of an order of magnitude of the undrained Young's modulus. It would also appear that values of E_{ui} greater than 100 MPa should not be used.

It is not advisable to use the initial tangent modulus when designing foundations except where the imposed loads are low. For typical loading conditions where the factor of safety against bearing capacity failure is between 2 and 3, the above moduli would have to be reduced by factors of 3 to 5, in order to estimate the expected foundation settlements.

4. SUMMARY OF TRIAXIAL TEST RESULTS

In the case of laboratory triaxial tests, the measured shear strength is affected by factors such as soil disturbance, in particular stress relief during soil preparation prior to re-consolidation, and the use of distilled water as the pore fluid, which causes swelling of the expansive clay.

4.1 Shear Strength of "Undisturbed" Samples

The laboratory shear strength tests that are most widely used are the unconsolidated undrained (UU) triaxial tests in which a confining pressure equal to the estimated overburden pressure is applied to the specimen. In a few cases, "consolidated quick" triaxial tests have been performed. This test involves saturation of the clay (by applying a back pressure at low effective confining pressure), isotropic consolidation with the effective all-round pressure equal to the overburden pressure (total stress), followed by shearing under undrained conditions. Fig. 5 shows the variation of undrained shear strength with the initial moisture content of the soil specimen. It can be seen in Fig. 5 that there is a wide scatter between the lower and upper values of undrained shear strength (the ratio $(s_u)_{min}/(s_u)_{max}$ varies from 2 at high moisture contents, to 4 at low moisture contents, where the subscripts min and max denote minimum and maximum values, respectively. Thus, the moisture content of the soil specimen should not be used alone as the basis for estimating the undrained shear strength.

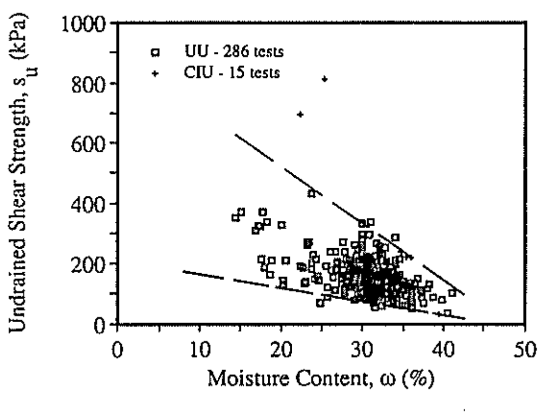

Fig. 5: Variation of laboratory-determined shear strength with moisture content

Fig. 6 shows the variation of undrained shear strength s_u, with confining pressure on undrained shear strength. In the UU tests, there appears to be a maximum value of undrained shear strength approximately equal to 300 kPa. The low values of s_u (less than 60 kPa) have been attributed to failures along fissures. The results from CIU tests indicate an increase in s_u with consolidation pressure, with some scatter though much less than that for UU tests.

Fig. 7 shows a plot of peak stress difference $q = (\sigma_1 - \sigma_3)$ versus mean normal effective stress $p' = (\sigma_1' + 2\sigma_3')/3$, from the CIU tests. The effective shear strength of Keswick Clay can be expressed by the Mohr-Coulomb strength parameters $c' = 0$ and a friction angle $\phi' = 18°$ to $21°$. The angle of internal friction, ϕ', falls within the expected range of values reported in previous investigations (for example Richards and Kurzeme, 1973).

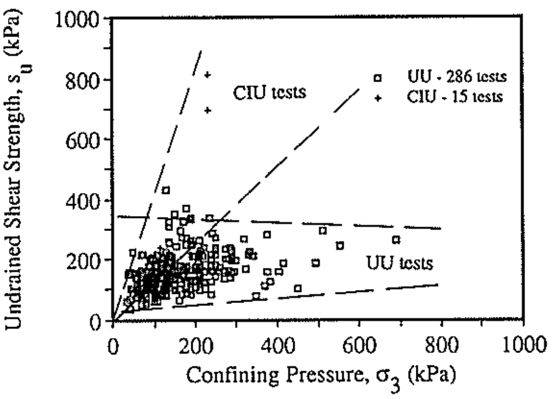

Fig. 6: Variation of laboratory shear strength with consolidation pressure

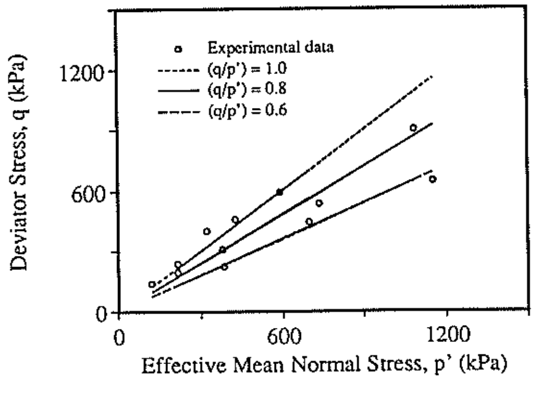

Fig. 7: Peak and Residual strength envelopes in p' - q stress space

4.2 Compressibility

The value of undrained Young's modulus at a fraction of the failure deviator stress has been computed from the deviator stress versus axial strain response. Fig. 8 shows the variation of the undrained Young's modulus corresponding to 50% of failure deviator stress, E_{u50}, versus the isotropic consolidation pressure, p_o'. There is a wide scatter probably due to soil disturbance and the natural soil variability as the boreholes were taken along a line of 900 metres. The ratio E_{u50}/p_o varies between 75 and 200, with an average of 120. Compared to the results of in-situ tests, the initial tangent modulus E_{ui}, is between 4 to 10 times higher than E_{u50}.

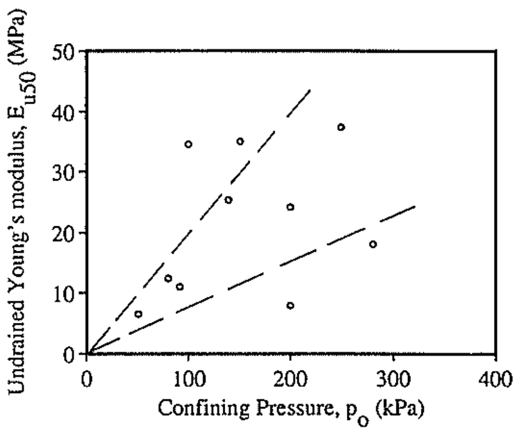

Fig. 8: Variation of Young's modulus with confining pressure

5. INTERPRETATION OF TEST RESULTS

5.1 In-situ Earth Pressure Coefficient K_o

The earth pressure coefficient K_o, $(\sigma_{ho}'/\sigma_{vo}')$, has been estimated using the total horizontal stress estimated from pressuremeter tests, the total vertical stress based on overburden pressure, and total soil suction estimated from the psychrometer test.

The results of pressuremeter tests consistently indicate that the total horizontal stresses in the Adelaide clays are greater than the estimates of total vertical stresses based upon the overburden pressures. It is assumed that the values of total soil suction, measured using the psychrometer, are representative of the negative pore pressures acting within the clay mass.

The earth pressure coefficient K_o can be determined from the relation:

$$K_o = \frac{\sigma'_{ho}}{\sigma'_{vo}} = \frac{\sigma_{ho} - u_o}{\sigma_{vo} - u_o} = \frac{K - \alpha}{1 - \alpha} \qquad (1)$$

where

σ_{ho}, σ_{vo} = total horizontal and vertical in-situ stresses, respectively

$\sigma'_{ho}, \sigma'_{vo}$ = effective horizontal and vertical in-situ stresses, respectively, and
u_o = in-situ pore pressure.
$K = \sigma_{ho} / \sigma_{vo}$ the ratio of total stresses
$\alpha = u_o / \sigma_{vo}$ the pore pressure ratio

Based on the results of consolidated undrained triaxial tests where the excess pore pressures are negligible, even at failure, (Skempton and Bishop's parameter $\bar{A} = \Delta u/\Delta(\sigma_1-\sigma_3) \leq \pm 2\%$), it is assumed that during the pressuremeter test, the negative pore pressures remain unchanged.

In using Eqn.1, u_o can be estimated from the equivalent total soil suction p" (the sum of matrix suction p''_m, and solute suction p''_s), from the values of pF determined using the psychrometer tests. Aitchison (1965) presented an equation for estimating the equivalent negative pore pressure u_o, which can be written as follows:

$$u_o = \chi_m p''_m + \chi_s p''_s \quad (2)$$

where

$$p''_m = -0.0981(10^{pF_m}) \text{ in kPa} \quad (3)$$

and

$$p''_s = -0.0981(10^{pF_s}) \text{ in kPa} \quad (4)$$

p''_m is the difference between air pressure and water pressure, p''_s is the solute or osmotic suction, χ_m and χ_s are factors which lie within the range 0 to 1. Because the clay is very close to full saturation in situ, termed quasi-saturated (see Jaksa and Kaggwa, 1992 and Kaggwa, 1992), and the common practice of using distilled water as the pore fluid in laboratory tests, it is assumed as a first approximation that $\chi_m = \chi_s = 1$. This approximation is being investigated by the authors, using a modified triaxial base pedestal, ensuring that the pore water within the specimen is isolated from the distilled water used in the back pressure system.

The results of pressuremeter tests and psychrometer tests (from the Adelaide Subway Project, 1979, and the Commonwealth Centre Project, 1984) have been used to determine the variation of K_o with depth, which is shown in Fig. 9. It can be seen that in general, the earth pressure coefficient at rest decreases from about $K_o = 1.45$ at a depth of 2 m below the ground surface, to $K_o = 1.05$ at a depth of about 4 m below the ground surface. For depths between 5 m and 10 m, the average value of $K_o = 1.20$. It should be noted that the effects of seasonal variations in surface temperature and moisture are less pronounced as the depth below the ground surface increases, and are insignificant at depths greater than 5 m.

The above values of the coefficient of earth pressure, K_o are to be expected in an over-consolidated clay. The over-consolidation ratios in laboratory specimens would therefore be expected to be much higher for the typical consolidation pressures based on overburden pressure.

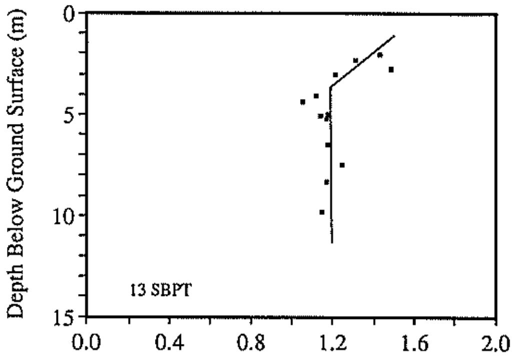

Fig. 9: Variation of predicted in-situ K_o with depth

5.2 Normalised Undrained Shear Strength

Due to the semi-arid climate in Adelaide, the clays have been subjected to cycles of wetting and drying. As the clay dried, large negative pore pressures were generated, leading to high normal effective stresses, and during wetting the negative pore pressures dissipated, leading to lower normal effective stresses. This is the main cause of overconsolidation, and the in-situ normal effective stresses are less than the maximum stresses to which the clay has been subjected during the stress history. It is uncertain whether the overconsolidation ratio is related directly to the highest normal effective stresses or to some intermediate value. The reason for this uncertainty can be appreciated from a consideration of the changes in void ratio which accompany the drying and wetting processes.

Fig. 10 shows a plot of the undrained shear strength ratio (s_u/σ'_{vo}) (where σ'_{vo} is the effective in-situ vertical stress) versus depth below the ground surface based on SBPM tests. This ratio properly takes into account the effects of soil suction. The range of values of the ratio (s_u/σ'_{vo}) is between 0.1 and 0.2.

The results presented by Wroth and Houlsby (1985) show that the undrained shear stress ratio increases as the overconsolidation ratio increases. Values of (s_u/p'_o) for isotropically consolidated triaxial specimens of clay range from 0.25 and 0.3 for normally consolidated specimens, to 0.5 and 0.6 for isotropic overconsolidation ratio (p'_{max}/p'_o) = 3, where p' is the mean normal effective isotropic consolidation pressure and the subscript max denotes the maximum value to which the specimen is subjected prior to triaxial testing using a consolidation pressure p'_o.

5.3 Normalised Young's Modulus

The values of undrained Young's modulus obtained from the interpretation of pressuremeter and screw plate load tests have been normalised with respect to the effective vertical stress σ'_{vo} and plotted against depth below the surface of the Keswick Clay layer in Fig. 11. The ratio (E_{ui}/σ'_{vo}) falls within the range 10 to 50.

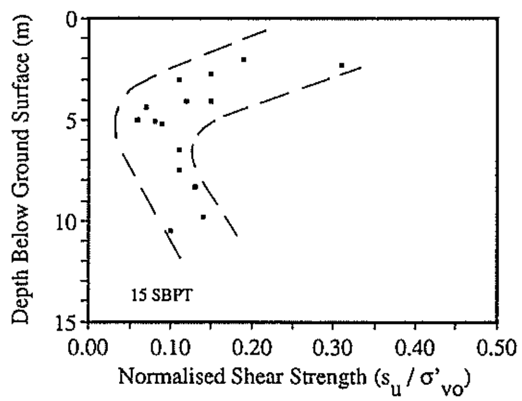

Fig. 10: Normalised shear strength ratio (s_u/σ'_{vo}) versus depth

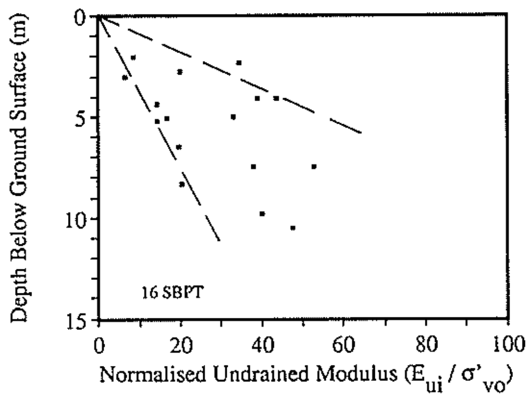

Fig. 11: Normalised undrained Young's modulus (E_{ui}/σ'_{vo}) versus depth

On the basis of Fig. 10 and Fig. 11, the ratio of E_{ui}/s_u lies within the range 100 to 250. As pointed out earlier, the undrained Young's modulus for use in design calculations E_u, would be typically 3 to 5 times less than the initial Young's modulus. Thus the ratio (E_u/s_u) would be within the range of 20 to 80, which is comparable to values obtained for most overconsolidated clays.

6. CONCLUSIONS

Although Keswick Clay is subjected to negative pore pressures in-situ, the normalised undrained shear strength and Young's modulus, with respect to the in-situ vertical effective stress, fall within the expected range of typical over-consolidated clays subjected to positive pore pressures. Provided the in-situ effective stress state within the clay mass can be properly estimated, and in particular the negative pore pressures, then experience from other clays can be used to estimate the undrained shear strength and Young's modulus for use in the preliminary design of foundations.

7. ACKNOWLEDGEMENTS

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