A Down Hole Plate Load Test for Insitu Properties of Stiff Clays

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SUMMARY The screw plate load test is proposed as a method for field testing of stiff clays. The plate is advanced beyond the bottom of the borehole in a manner that minimizes soil disturbance. Values of undrained modulus, drained modulus, undrained shear strength and coefficient of consolidation are determined in a straightforward test procedure. The fundamental nature of the test as well as preliminary results suggest that extremely valuable data may be obtained at relatively low cost.

1 INTRODUCTION

Because of the difficulties associated with soil sampling and laboratory testing for efficient evaluation of soil properties, investigators have frequently sought suitable field testing procedures. Indeed, under ordinary circumstances, for determination of engineering properties of coarse-grained soils, field testing is the only viable procedure. The disturbance associated with a proper sampling process for soft insensitive clays is not particularly severe but frequent use of field testing procedures such as vane shear tests and Dutch cone penetrometer tests has been made. The methods have tended to become more popular in commercial testing of the latter soil types particularly because of economic advantages to be gained. On the other hand, in the case of stiff clays, there appears to have been a preference for laboratory testing and relatively little effort has been expended on the development of field testing procedures. This situation has prevailed in spite of the fact that it is well established (Hooper and Butler, 1966) that as far as borehole sampling is concerned even the most careful techniques cause considerable disturbance in these soils. Apart from possible direct physical disturbance of the soil structure the relief of high levels of insitu stresses may lead to development of fissures not present in the natural state. Although the original insitu stress conditions may be re-established in the laboratory in conjunction with proper stress path testing techniques the planes of weakness will remain. The test values obtained for strength and deformation parameters may be seriously affected. This has been demonstrated for Adelaide clays by Woodburn (1972). Furthermore, determination of the magnitude of horizontal stress may involve considerable uncertainty, and even when a reasonable prediction of horizontal stress can be made, few laboratories are equipped for testing in accordance with stress path procedures where the coefficient of earth pressure at rest, K_o, is greater than unity. In areas where high levels of solute suction prevail the influence of diffusion of incompatible test fluid may be significant. These arguments appear to indicate a need for a procedure that can economically measure the properties under field conditions before any significant change in the insitu stress condition has occurred. One recent development in this direction has been the self-boring pressuremeter but with this tool there is a question of economic viability for many projects. In this paper an adaption of the screw plate load test is proposed as an alternative.

2 PREVIOUS SIMILAR WORK

The screw plate load test has been used in sands and sandy soils by Janbu and Senneset (1973), Schmertmann (1970) and Dahlberg (1974) and in soft clay soils by Schwab and Broms (1977). In these tests a single pitch helical plate of diameter generally in the range 150 to 250 mm is screwed from the ground surface to the test depth. The load is applied through a hydraulic system to a piston near the plate. At regular intervals of load application the resulting deflection is measured and either or both of compressibility and strength parameters are obtained.

In the United Kingdom load tests using large diameter flat plates have recently been conducted on carefully prepared surfaces in shafts and large diameter boreholes in stiff clay. The Building Research Station (Marsland 1971, 1974) has used 865-mm diameter plates mainly as a control test for evaluation of other methods such as triaxial and cone penetration tests. A smaller version has been used as a commercial test procedure to some extent. Deflection control rather than load control has been the basis of the British tests and considerable attention has been given to test rate. For the 865-mm plate a penetration rate of 2.5 mm/min has generally been used. Tests conducted at rates several times higher and several times lower have shown little change in undrained property values.

3 TEST DESCRIPTION

The test developed at the University of Adelaide incorporates some aspects from both of these test types as well as some additional features. The plate, 88 mm in diameter, is of a low pitch helical form and, for tests in stiff clays, is advanced approximately 100 mm beyond the bottom of a 90-mm diameter pre-drilled hole. The rate of advance per revolution of the plate is controlled by a screw at the surface whose pitch is precisely the same as the underside of the test plate. In this way the axial distortion of the undisturbed soil is minimized during advancement of the plate. The leading edge of the helical test plate is chisel shaped so that the soil is cut and forced upwards into the open hole. Experience has shown that a solid plug of soil forms above the plate. This is significant in that it forms a seal above the plate and minimizes changes in water content or soil suction during the test period.

The test is in a form that essentially permits the concurrent measurement of undrained modulus, drained modulus, coefficient of consolidation and undrained shear strength. Both a lever arm system and a geared drive are incorporated to enable either load or settlement control as desired. The test unit is shown in Figure 1. The procedure used in the test method is as follows:-

(a) The plate at test depth, 100 mm below the bottom of the drillhole, is loaded to the estimated value of the overburden stress. If movement occurs no further loading is applied until movement is less than 0.0005 mm/min.

(b) An estimate is made of the ultimate capacity of the plate based on the torque required to advance the plate to the test position. A correlation obtained from previous experience is used for this estimate.

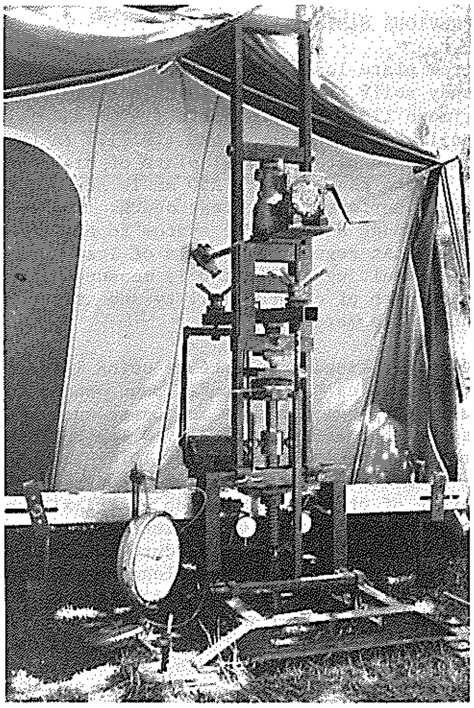

Figure 1 Test Apparatus at Constant Rate of Settlement Stage

(c) A load equal to one third of the estimated net ultimate capacity is applied to the plate through the lever arm system. Deflection readings are taken with time until the deflection is less than 0.0005 mm/min.

(d) The system is then operated in a settlement-controlled mode and the plate is advanced at a rate of 0.25 mm/min until a peak stress level or, alternatively, 2 percent of the plate diameter is reached. Records are taken of both load and settlement.

4 INTERPRETATION OF TEST DATA

4.1 General Load - Settlement Behaviour

The theory used for determination of an equivalent elastic modulus has been reviewed by Selvadurai and Nicholas (1979). They give the relationship for the settlement, $\Delta\rho$, of a loaded horizontal disc in an elastic medium as

$$\frac{\Delta\rho}{\Delta\sigma D/E} = \frac{(1+\upsilon)\ \sqrt{3-4\upsilon}}{4[1+\{\ln(3-4\upsilon)\}^2/\pi^2]} \qquad (1)$$

where $\Delta\sigma$ is the plate stress increment, D is the plate diameter, and E and υ are the elastic parameters of the medium. The disc is assumed to be rigid, to be bonded to the medium on its underside but to be unbonded on its upper surface.

Finite element studies made by the authors have shown that the effects of an unlined vertical shaft no nearer than one diameter above the disc are negligible. A possible additional source of error in applying the model to soil is the onset of local yield. D'Appolonia et al (1971) have investigated this matter and have shown that, whereas local yield may occur at lower stress levels in normally consolidated clays, such a condition is unlikely at less than 50 percent of the ultimate bearing stress in overconsolidated soils. In the proposed test modulus measurements are restricted to approximately the lower third of the stress range.

4.2 Undrained Modulus - E_u

The substitution, $\upsilon = 0.5$, in Equation 1 for the undrained case yields the equation for undrained elastic modulus, E_u.

$$E_u = 0.38\ \frac{\Delta\sigma D}{\Delta\rho} \qquad (2)$$

4.3 Drained Modulus - E_s

Little information is available for υ fcr the stiff Adelaide clays but the general similarity with London clay in many respects suggests that similar υ values would be reasonable. Wroth (1971) has indicated a value of 0.16 and on this basis the rounded value of 0.2 is chosen. Substitution in Equation 3 leads to:

$$E_s = 0.42\ \frac{\Delta\sigma D}{\Delta\rho} \qquad (3)$$

4.4 Coefficient of Consolidation - c_r

Janbu and Senneset (1973) have proposed a version of the square root of time approach for determination of a radial component of the coefficient of consolidation, c_r. According to these authors the time factor, T_{90}, is 0.335 so that

$$c_r = \frac{T_{90}\ R^2}{t_{90}} = 0.335\ \frac{R^2}{t_{90}} \qquad (4)$$

where R is the plate radius and t_{90} the time to 90 percent consolidation.

4.5 Undrained Shear Strength - c_u

For conversion of the ultimate capacity of the plate to an undrained shear strength value Marsland (1974) has used a bearing capacity factor, N_c, of 9.6 for stiff London clays. He cites theoretical values of about 5 for E_u/c_u ratios near 20 and between 9 and 10 for E_u/c_u greater than 500 obtained from the expansion of a spherical cavity and approximate solutions by Meyerhof (1951, 1961). For Adelaide clays as for London clays the higher figure appears to be more appropriate (Cox, 1970) and the equation used by Marsland is proposed. That is:

$$c_u = q_u/9.6 \tag{5}$$

where q_u is the net ultimate capacity of the plate.

5 RESULTS OF PRELIMINARY FIELD TESTS

5.1 Tea Tree Plaza Test Site

The first series of tests were conducted in the undrained mode only. (See Figure 2). Both field tests and consolidated (isotropic) undrained triaxial tests on 35 mm diameter samples were conducted on soils from Tea Tree Plaza, an Adelaide shopping centre site. These soils consist of a stiff, highly plastic, pleistocence clay. The deflection rates used were relatively high and 20 percent of the plate diameter was used as the failure criterion. In these tests the bearing stress continued to rise at a relatively rapid rate after the initial yield period and it was only in later tests that the slower penetration rate compatible with that of Marsland (1974) was adopted. The undrained shear strength values obtained are likely to be high because of the high settlement rate and the 20 percent failure criterion. However, the relative results are of considerable interest. A rapid consistent linear increase in strength with depth is observed. There is a similar trend in the results of the triaxial tests but more scatter is apparent. The results for undrained modulus are less decisive but a constant ratio appears likely between modulus and strength. A factor of 4 to 5 appears to exist between laboratory and field results for undrained modulus.

5.2 Victoria Square Test Site

At this site the combined drained - undrained loading procedure was adopted. The soil is a typical Adelaide city soil profile - a stiff, highly plastic, grey-green Hindmarsh clay. The undrained shear strength based on the stress at a deflection of 2 percent of the diameter and a bearing capacity coefficient of 9.6 is indicated in Figure 3. Two of the ten tests, those at 7.5 and 9 metres depth were entirely deflection controlled. The values obtained for undrained shear strength for these cases appear to indicate that an intervening consolidation phase has little influence on the measured undrained shear strength value.

5.3 University of Adelaide Campus Site

For the tests in a brown silty clay at the University of Adelaide campus site, improvement in the apparatus enabled measurement of the additional drained soil parameters in what appears to be a reliable manner.

Figure 4 represents the settlement-time and stress-settlement graphs, respectively, for a typical test at 4.5 metres. The total time for the test including extension of the borehole was about 2 hours. In Figure 4(a) settlement was plotted in terms of square root of time with detailed construction as proposed by Janbu and Sennesett (1973). This enabled determination of undrained modulus, drained modulus and coefficient of consolidation. Drained

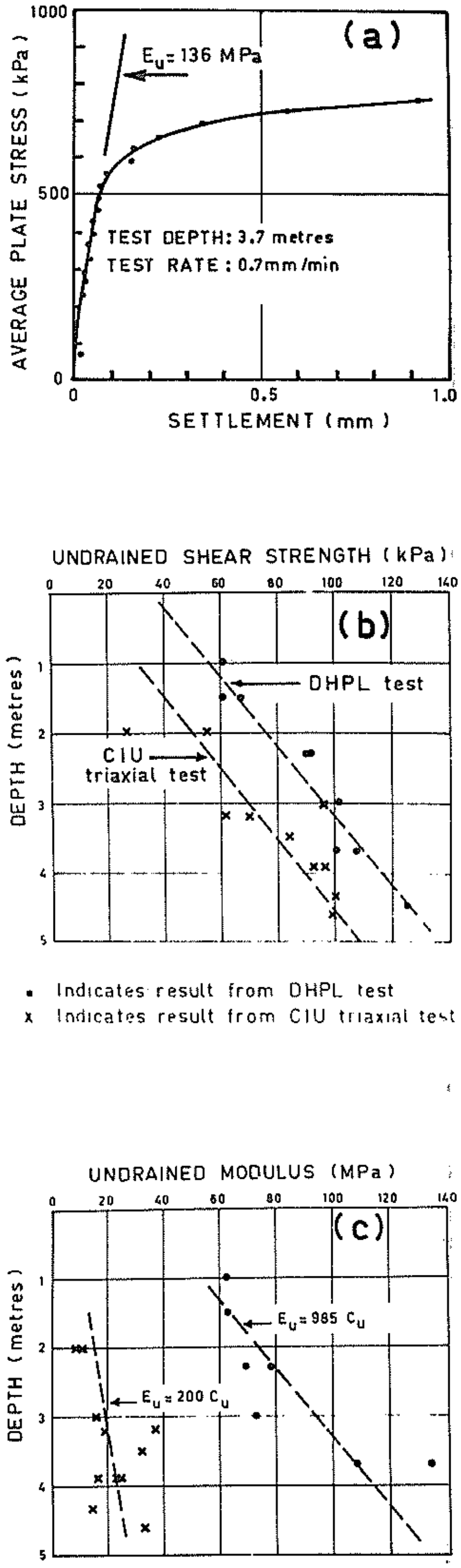

Figure 2 Tea Tree Plaza Controlled Settlement Results

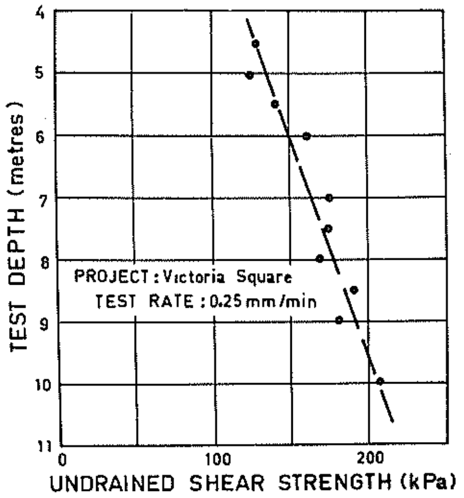

Figure 3 Undrained Shear Strength - Depth Profile for Victoria Square Site

and undrained moduli were found to be 28.3 and 105.7 MPa, respectively. The stress-settlement plot in Figure 4(b) produced the values E_u = 48.9 MPa and C_u = 35.9 kPa. The discrepancy between the two E_u values highlights the arbitrary nature of the usual definition of E_u. It is to be expected that the value for E_u interpreted in an instantaneous sense from the $\sqrt{t}$ plot will be higher than that from a strain controlled test.

6 ADVANTAGES OF THE PROPOSED TEST

6.1 K_o Conditions

Simons and Som (1973) have discussed the influence of lateral stresses on deformation characteristics of London Clay. In particular they indicate that the axial compressibility in a triaxial test is greatly influenced by the ratio of lateral to vertical effective stress. The difficulties associated, firstly, with determining the insitu K_o conditions and, secondly, with reproducing these conditions in the laboratory for stiff clays would tend to indicate that this is likely to be a large source of error in compressibility estimation. The nature of this field test ensures that the true K_o conditions exist at the test start.

6.2 Approximation of True Stress History

The usual aim of the first part of reproduction of the insitu stress history in a laboratory programme is automatically achieved as discussed above. The next step in the laboratory test is to reproduce the stress conditions at representative point at working load (Davis and Poulos, 1968). To apply stresses that correspond to about one third of those associated with the failure stress level is frequently considered appropriate. This is the procedure used in this test. However, there is no control over the horizontal stress in-

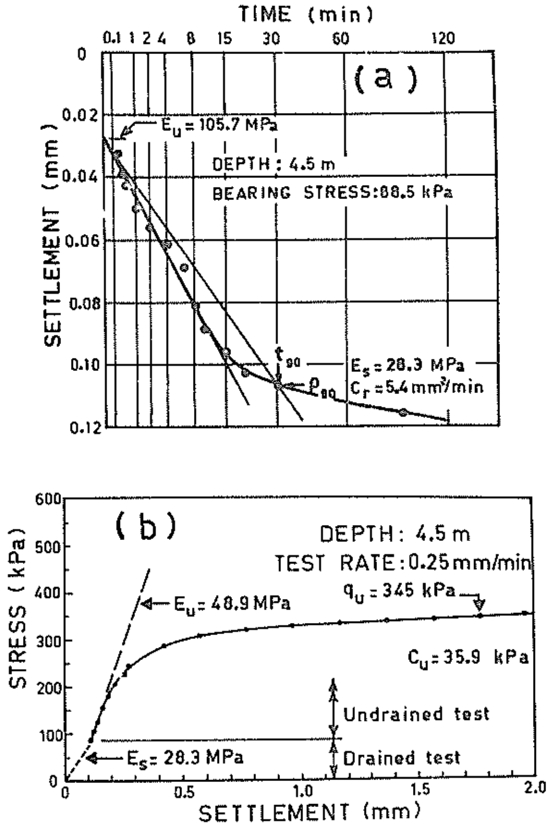

Figure 4 Settlement-Time and Stress-Settlement Results for University of Adelaide Site

crease as in the triaxial test and in this regard the stress path is not entirely reproduced as desired. Nevertheless, the approximation to the required stress path is far superior to that in many test types and certainly to all currently used field tests.

6.3 Test Orientation

The proposed test essentially models a large scale structure and in doing so automatically accounts for some of the complexities. Many other tests fail to do this, and, in these cases, possible effects of anisotropy of soil properties need careful consideration. For most structures a vertical orientation of applied load is appropriate and this is the normal situation for this test. By applying theory appropriate to isotropic conditions, as is usual, it is likely that an equivalent rather than a true property value is obtained. Because of the appropriate orientation of the down hole plate load test it is likely that the equivalent value as obtained in this test is quite appropriate for design.

6.4 Physical Disturbance

The nature of the penetration of the plate ensures little disturbance of the insitu soil.

6.5 Natural Porewater and Temperature Environment

Particularly in the Adelaidearea where soils of high expansion potential are associated with high solute suctions there is considerable doubt associated with the validity of conventional oedometer and triaxial test procedures. The use of back-pressure testing in conjunction with tap water or distilled water is not possible under these conditions. A field test procedure is a logical alternative where natural porewater conditions appear to be necessary. Proper temperature conditions are sometimes of considerable importance and these too are automatically provided by the insitu test.

6.6 Range of Soil Properties

The single test arrangement can be used to provide a valuable set of engineering properties all of which are potentially more realistic than values obtained by present methods. These include the undrained shear strength, the undrained elastic modulus, the drained elastic modulus and the coefficient of consolidation.

6.7 Economics

Experience to date indicates that a two man crew supported by equipment at a capital cost of about $4000 could complete 3 tests per day including the measurement of drained properties. This would seem to be a viable commercial operation when it is considered that the associated laboratory time would be almost eliminated.

The plate size in use at present is smaller than that used in European practice and much smaller than the British plates. This aspect must be further studied in relation to Adelaide clays as it is likely that lower strengths will be obtained for larger plates as has been the case in Britain due to natural fissures. However, in terms of volume of soil tested, the present size of plate is superior to most laboratory tests and many field tests. Equipment for drilling 90-mm diameter holes exhibits much greater convenience and lower cost. In addition, the reaction requirements are very reasonable even for very stiff soils. This convenient size means that a larger number of tests may be run and better representation of the overall conditions may be obtained. The limitations on test depth are essentially the same as those associated with conventional drilling equipment.

7 SUMMARY AND CONCLUSIONS

Preliminary results obtained from the down hole plate load test apparatus specially developed at the University of Adelaide for testing of stiff clays appear to indicate that it has some potential as both a research and commercial testing tool in these types of soils. In particular, the capability for measurement of drained modulus and coefficient of consolidation in a reasonable test time period is a valuable one. Much work remains to be done to further improve interpretation procedures and to consider the influence of variables such as the test plate diameter. However, there is good reason to believe that the parameters as presently obtained exhibit a higher level of reliability than those associated with many currently used test methods.

8 ACKNOWLEDGEMENT

Financial assistance for the work presented herein was obtained by the first author from the Australian Research Grants Committee.

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