Triaxial Tests on Soil at Elevated Cell Pressures

Essais triaxiaux sur les sols à pressions latérales élevées

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SUMMARY

The apparatus and techniques for the testing of soils at elevated cell pressures are briefly described. Examples are given of tests on a stiff-fissured clay and a sand. The cell pressures used in these tests lie in the range 0–1,000 lb/sq.in. The changes in the mode of failure and in the shape of the failure envelope are discussed. The stress paths for undrained tests on saturated samples of sand are considered in relation to particle crushing determined by sieving and by examination under the microscope. The coefficient of earth pressure at rest is measured for both soils over a wide range of stress. A pilot test for examining the validity of the usual expression for effective stress at elevated pore pressures is also described.

SOMMAIRE

Les appareils et techniques de l'essai triaxial à pressions latérales élevées sont brièvement décrits. Des exemples sont donnés d'essais sur une argile dure fissurée ainsi que sur un sable. Les pressions latérales utilisées dans ces essais se situent entre 0 et 1000 lb/po.ca. Les changements du mode de rupture et de l'enveloppe de rupture sont examinés. Les lignes de contrainte pour des essais à teneur en eau constante sur des sables saturés sont considérées en fonction de l'écrasement des particules mis en évidence par tamisage et observation au microscope. Le coefficient de pression des terres au repos est mesuré pour les deux sols pour un domaine étendu de contraintes. Un essai pilote est décrit destiné à examiner la validité à des pressions interstitielles élevées de l'expression habituelle des contraintes effectives.

DEVELOPMENT OF APPARATUS AND TECHNIQUES

TWO OF THE DEVELOPMENTS described in this section are concerned with the more accurate measurement of axial load and the third relates to the measurement of coefficient of earth pressure at rest.

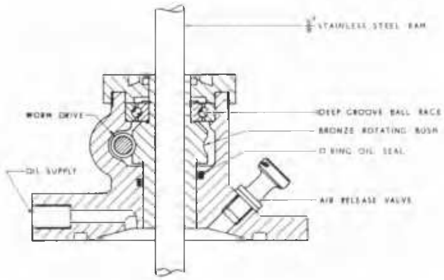

FIG. 1. Cell top with rotating bush for pressures up to 1,000 lb/sq.in.

The rotating bush (Fig. 1) used to minimize ram friction is a development of a type used at the Imperial College since 1956 and is designed to work at cell pressures up to 1,000 lb/sq.in., with high axial loads. A honed, stainless steel ram, ⅝ inch in diameter, passes through a honed bronze bush which is rotated by a worm drive at 2 rpm. No oil seal is used inside the bush, and with a radial clearance of about 2×10^{-4} in. and castor oil on top of the water in the cell the leakage under 1,000 lb/sq.in. is only a few cubic centimeters per day. The groove for the O ring seal on the outside of the bush is inclined to prevent rapid localized wear of the bronze bushing.

A calibration test showed that a horizontal force of 57 lb acting 1.1 in. from the bushing resulted in a vertical frictional force of 15.5 lb. This dropped to 0.23 lb when the bushing was rotated. The lateral force tends to be large at the post-failure strains at which residual stresses are measured. Under these conditions average differences between loads with stationary and rotating bushes for tests on stiff-fissured clay were 9.6 per cent with a tilting cap, and 15.6 per cent with a guided cap (for which 33 per cent was measured in an extreme case).

In tests to study the validity of the effective stress equation for saturated soils it is necessary to measure the effect on

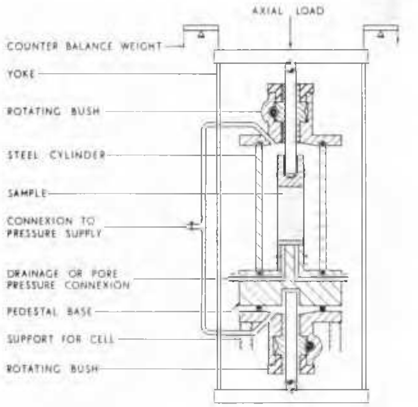

FIG. 2. Cell with rotating bushes and hydraulically balanced rams (diagrammatic).

strength of elevating both cell pressure σ_3 and back pressure u so as to maintain a constant value of $\sigma_3 - u$. The upthrust on the ram due to fluid pressure may in this case exceed the load due to the strength of the sample by a factor of 10 or more. In order to permit the accurate measurement of the strength of the sample, the upthrust on the ram is hydraulically balanced as shown diagrammatically in Fig. 2. This method also permits the cell pressure to be varied systematically during a test without any compression of the proving ring and consequent change in strain rate.

A calibration test with matched rams gave an axial load change of less than 0.05 lb over the cell pressure range 0–1,000 lb/sq.in. This apparatus is being used for development studies for equipment with a range 0–10,000 lb/sq.in. now being constructed.

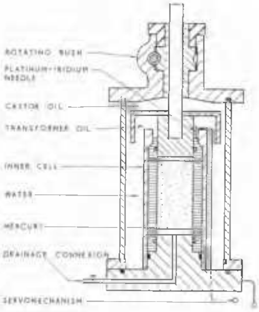

FIG. 3. Cell for consolidation with zero lateral yield (diagrammatic).

In the cell illustrated in Fig. 3 the condition of compression with zero lateral yield is achieved if the level of the mercury remains constant in the annular space between the "unstressed" inner cell* and the top cap, which is of the same cross-sectional area as the sample. Departure from this condition breaks the contact between a platinum-iridium needle and the mercury surface, which is covered by transformer oil, and operates a servomechanism controlling the axial load (following the procedure described by Bishop, 1958, for soils of low permeability).

With a 1½-inch diameter and 3-inch high sample, and with an accurately centred annular gap of 0.031-inch radial width, the change in mercury level corresponding to "make" and "break" in the circuit is equivalent to an average change in sample diameter of 2×10^{-5} in. The needle is placed in a local enlargement of the annulus.

RESULTS OF TESTS ON LONDON CLAY

The Mohr envelope representing failure conditions (peak stress) for drained tests is given in Fig. 4 for samples cut with their axis vertical from a block from Level E of the Ashford Common shaft. Level E is 114 ft below ground surface and the relevant index properties are: $w_L = 70$, $w_P = 27$, clay fraction = 57, activity = 0.75, initial water content = 24.2, and undrained strength $c_u = 60$ lb/sq.in.

Filter paper strips were used to accelerate drainage and the time to reach the peak stress was estimated from the

*The inner cell is unstressed in the sense that it is not subject to a significant pressure difference. However at high cell pressures the use of a material for the inner cell such as Perspex, having a high compressibility, can lead to an appreciable departure from the condition of zero lateral yield.

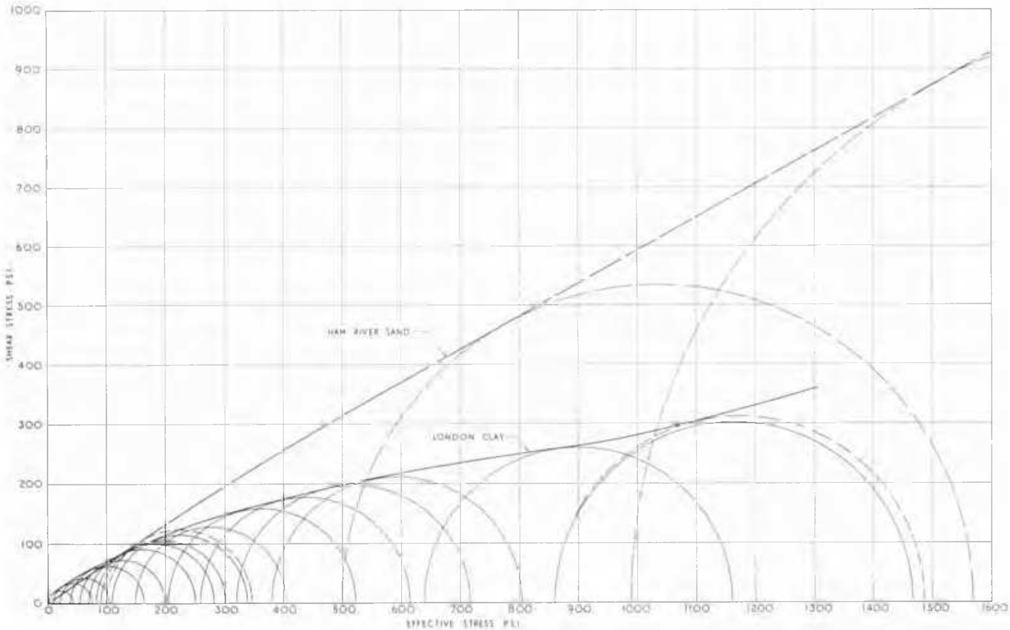

FIG. 4. Mohr envelopes for undisturbed London clay from 114 ft. below ground level and for washed Ham River sand.

rate of consolidation when the all-round pressure was applied. The time to the peak was typically 4½ to 7 days, reaching 22 days at very high pressures, where the permeability of the filter paper was greatly reduced. The methods of estimating the appropriate test duration are discussed by Bishop and Henkel (1962) and Bishop and Gibson (1963).

The slope of the failure envelope shows a very marked change in passing from the low stress range, where ϕ' exceeds 30°, to the high stress range, where ϕ' falls to about 10°. At low stresses the samples failed at 2 to 3 per cent axial strain, after which the axial load fell to as little as 15 per cent of its peak value. At very high stresses, on the other hand, the samples continued to shear at almost constant stress. At the highest cell pressure used in this series ($\sigma'_3 =$ 860 lb/sq.in.) the stress difference ($\sigma_1 - \sigma_3$) was still increasing and the volume decreasing when the limit of travel of the ram was reached. Extrapolation gave a peak value shown by a broken circle in Fig. 4. This suggests that the Mohr envelope may steepen at stresses well in excess of the preconsolidation load (estimated at about 600 lb/sq.in.), but tests at much higher pressures are required to confirm this.

The results of a limited number of tests which failed prematurely on an apparent fissure are not included in Fig. 4, and the Mohr envelope therefore approximates to that of intact clay. The lowest value obtained for a sample failing on a fissure corresponds to $c' = 0$ and $\phi' = 15°$. Fig. 4 includes some tests in the lower stress range in which a rotating bush was not used. The error in the peak stress in this pressure range appears to be of the same order as the natural scatter of the results.

The results of a test on a vertical sample to measure the coefficient of earth pressure at rest K_0 on reconsolidation are given in Fig. 5. The initial state of stress was an equal

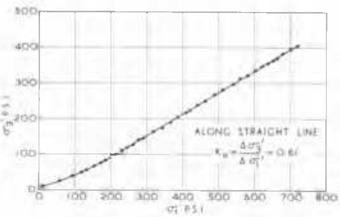

FIG. 5. Results of "K_0 test" on undisturbed London clay.

all-round effective stress of 10 lb/sq.in., to which the specimen was allowed to swell after sampling, when the observed swelling pressure was 100 lb/sq.in. Over the greater part of the stress range the ratio of $\Delta\sigma'_3$ to $\Delta\sigma'_1$ is constant and equal to 0.61. Previous tests on sand (Fraser, 1957; Bishop, 1958) have shown that the value of this ratio on reloading is less than the value of K_0 on first loading. From the approximate relationship $K_0 = 1 - \sin\phi'$ and the values of ϕ' for samples normally consolidated from a slurry it is estimated that on first loading K_0 would have been in the range 0.66 to 0.72, rising with increasing stress. As with sand these values exceed the reloading value.

RESULTS OF TESTS ON SAND

The stress-strain and volume change curves for three drained tests on loose saturated sand are given in Fig. 6, and the stress-strain and pore-pressure change curves for

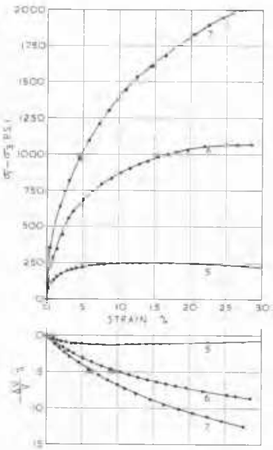

FIG. 6. Results of drained tests on saturated Ham River sand.

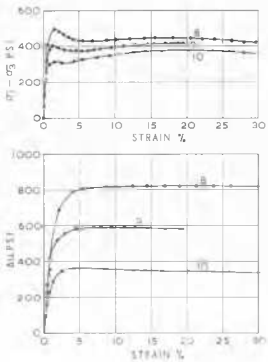

FIG. 7. Results of consolidated undrained tests on saturated Ham River sand.

three consolidated-undrained tests in Fig. 7. The stress paths, together with those of two special tests, are given in Fig. 8.

The results illustrate the difficulties encountered in determining shear parameters in materials in which structural breakdown, in this case involving crushing of the particles, occurs during shear. Of the drained tests in Fig. 6, test 5 ($\sigma'_3 = 100$ lb/sq.in.) shows the usual characteristics of an uncompacted sand, but test 6 ($\sigma'_3 = 500$ lb/sq.in.) begins to shear at constant stress but with decreasing volume, while in test 7 ($\sigma'_3 = 990$ lb/sq.in.) the value of ($\sigma_1 - \sigma_3$) is still increasing at the limit of the test. The maximum values of ϕ' in the three tests are respectively 33.5°, 31.1°, and 30.2°, which give only a relatively small curvature to the Mohr envelope (Fig. 4). A drained test following a stress path designed to lead to failure in the upper stress range under a decreasing average stress (test 12, Fig. 8) gave a value of 33.0°, the rate of change of volume per unit volume

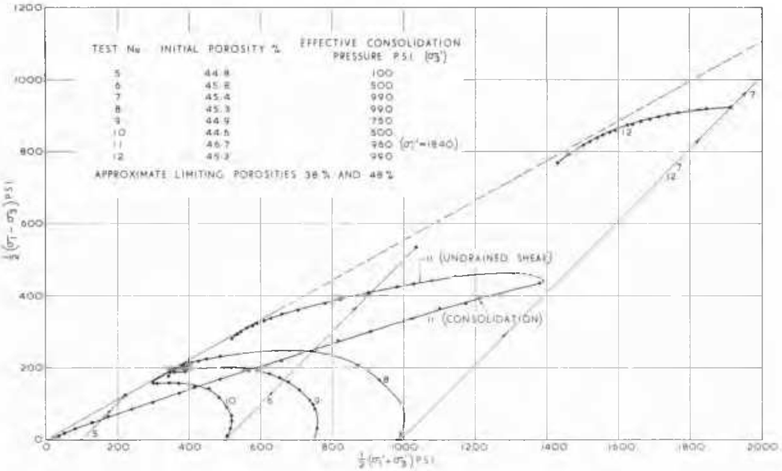

FIG. 8. Stress paths for tests on Ham River sand.

at failure still being slightly negative, i.e. $(\Delta V/V)/\Delta\epsilon_1 = -0.057$. These results may be compared with data presented by Vesić and Barksdale (1963), and Hirschfeld and Poulos (1963).

The consolidated-undrained tests (Fig. 7) show very high pore pressures and in the case of test 8 (effective consolidation pressure = 990 lb/sq.in.) a very small strain at failure. In test 8 the pore-pressure parameter A at peak $\sigma_1 - \sigma_3$ is 1.13 and its value approaches 1.8 at larger strains. At peak $(\sigma_1 - \sigma_3)$ (Fig. 8) the value of ϕ' is 21.3° and only in the ultimate state does ϕ' rise to 34.2°. This is a further example of the way in which soils undergoing structural breakdown during shear depart from the behaviour of idealized materials envisaged by Roscoe, Schofield, and Wroth (1958), who state that the critical state is reached in undrained tests when $(\sigma_1 - \sigma_3)$ reaches its maximum value, after which no decrease in stress should occur. Similar results have been obtained in the lower stress range on very loose sand (Waterways Experimental Station, 1950; Bjerrum, 1961) and on a sensitive clay (Taylor and Clough, 1951).

Test 11 (Fig. 8), where the sample was consolidated under the condition of zero lateral yield, similarly gave a value of $\phi' = 20.8°$ at peak $(\sigma_1 - \sigma_3)$, a maximum value of $\phi' = 33.9°$ being reached when both $(\sigma_1 - \sigma_3)$ and the mean effective principal stress had greatly decreased. In this test, in which the volume correlation method of control was used (Bishop, 1958), the values of K_0 on first loading were 0.48, 0.50, and 0.52 for values of the vertical effective stress of 500, 1,000, and 1,800 lb/sq.in., respectively. For $\phi' = 33°$ the relation $K_0 = 1 - \sin\phi'$ would give 0.46.

The amount of crushing of the grains appears to depend both on the consolidation pressure and on the shear strain to which the sample is subjected before sieving. For test 8, with a consolidation pressure of 990 lb/sq.in. and an axial strain of 31 per cent, the percentages passing the various B.S. sieves before and after testing were respectively: 52, 95.5, and 96.0; 72, 47.4, and 53.9; 100, 11.4, and 19.6; 150, 0.3, and 5.9. Under the microscope the fines produced were seen to consist of fragments and splinters of a widely varying size, largely of quartz. Even in the undrained test at 500 lb/sq.in. consolidation pressure the percentage passing the 150 sieve increased from 0.4 to 3.4. Structural breakdown may therefore be expected in the range of stresses encountered both in high dams and in piled foundations. Vesić and Barksdale (1963) provide additional data on this point over a wider range of stress, and the results of undrained tests by Lowe (1964) indicate a similar effect in coarse granular fill.

TESTS TO EXAMINE THE VALIDITY OF THE EFFECTIVE STRESS EQUATION

The equation for the effective stress w.r.t. shear strength in a saturated soil may be written:

$$\sigma' = (\sigma - u) + a(\tan\psi/\tan\phi')u.$$

where σ denotes total stress, a the unit contact area, ψ the angle of intrinsic friction, and ϕ' the angle of internal friction of the soil particles (Skempton, 1960). To determine the magnitude of the term $a(\tan\psi/\tan\phi')$, which is small at low stresses, it is necessary to use very large values of Δu compared with the value of $(\sigma - u)$ to produce a measurable percentage difference in the value of σ'.

In the multi-stage compression test in Fig. 9 using the apparatus shown in Fig. 2, a ratio of $\Delta u/(\sigma - u)$ of 174

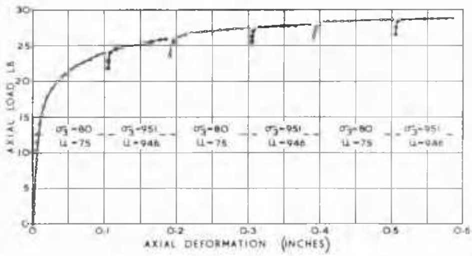

FIG. 9. Results of multi-stage compression test with $(\sigma_3 - u)$ constant and u varied systematically.

does not produce a significant change in strength and hence, by inference, in σ'. Tests at much higher values both of u and $(\sigma - u)$ are to be carried out in the apparatus under construction.

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