Investigation of the Behaviour of Peat under Varying Conditions of Stress and Strain

Etude du comportement de la tourbe soumise à des conditions variables de contrainte et de déformation

E. T. HANRAHAN, M.E., PH.D., *Civil Engineering Department, University College, Dublin, Ireland*
J. A. WALSH, B.E., *Civil Engineering Department, University College, Dublin, Ireland*

SUMMARY

The paper deals with the behaviour of a soft remoulded peat under load. Observations were made of the settlement and pore water pressure during the so-called primary phase of consolidation. The load tests were carried out in two tanks each three feet high, the smaller being one foot by six inches in plan area and the larger three feet by one foot in plan area. The load was applied on a block of area one foot by six inches, the condition being therefore one-dimensional consolidation in the smaller tank and three-dimensional consolidation in the larger. A continuous record of strains was made by photographing, at three times full size, the ends of three pins arranged triangularly at a point 3 inches below the midpoint of the block. The strains were measured using a travelling microscope. The stress characteristics of the peat were measured using both the vane and the triaxial apparatus and the pore water pressure was recorded at side and base. The relations between stress and strain are examined using the measured values of elastic constants and the pore water pressure coefficients.

SOMMAIRE

L'étude porte sur le comportement de la tourbe remaniée soumise à un chargement. Le tassement observé et les pressions interstitielles mesurées durant la soi-disante phase primaire de consolidation y sont rapportés. Les essais de chargement ont été faits dans deux récipients à parois transparentes de trois pieds de hauteur, la section du premier étant d'un pied par six pouces, et celle du second d'un pied par trois pouces. La section de la charge étant d'un pied par six pouces, on a donc pu réaliser des conditions de consolidation à une dimension et à trois dimensions. On a pu mesurer d'une manière continue, au moyen d'un microscope mobile et de repères, les déformations en un point situé à trois pouces sous le centre de la charge. Un appareil photographique spécial a permis d'observer ces déformations continues. Les propriétés de la tourbe ont été mesurées au moyen de l'appareil moulinet et du triaxial, et la pression interstitielle fut enregistrée. Les relations entre les contraintes et les déformations sont examinées à l'aide de ces propriétés mesurées.

THE ONE-DIMENSIONAL CONSOLIDATION TEST has many defects when used for forecasting the deformation associated with a finite-area load. It does not account for the lateral spread, or time-dependent shear deformation, which accompany such loading. The magnitude of lateral pressure, shear stress, and initial pore pressures cannot be estimated in advance.

A method (Hanrahan, 1964) has been published for evaluating the effects of lateral spread and both the magnitude and rate of settlement due to shear deformation or creep. It was thought desirable, however, to have a direct comparison between the results of the two methods of loading. Accordingly two tanks with transparent walls were constructed each 3 ft deep. The horizontal dimensions were 3 ft by 1 ft (unconfined test) and 1 ft by 6 in. (confined test, Fig. 1), respectively. Load was applied on a block measuring 1 ft by 6 in. In loading the large tank the block was placed centrally on the surface with the long axis of the block coinciding with the transverse axis of the tank.

Observations were made at regular intervals of total settlement, pore pressure, and strain, the last two being measured with reference to a point 3 in. below the longitudinal centre line of the block.

TYPE OF PEAT

The material employed in the tests was a uniform macerated (remoulded) peat, wholly organic, free from coarse fibres and wood, (w_L = 850, G = 1.462), and humification No. 6 (0–10 scale). The peat was consolidated as closely as possible to an air-free condition at a water content of 800.

SHEAR TESTS

Vane Tests

Fig. 2 shows an average curve based on some 40 tests showing a relationship between shearing strength and shearing strain, the latter being taken as the tangent of the angle of rotation of the vane. The apparatus used was a laboratory type with ½ in. × ½ in. vane, and the normal rate of application of the strain was approximately 1° per second.

Effect of rate of strain on strength. Vane tests were also carried out at rates of rotational strain of approximately 0.07, 0.3, 1.6, 6.4, and 25.6 degrees per second. The resulting curves were all similar to Fig. 2 and it was concluded that both the vane strength and the shape of the stress-strain curve were independent of the rate of strain employed in the test.

Flow deformation. It was observed that the peat was capable of withstanding, without measurable relaxation, values of the vane torque almost equal to the full value at failure. The behaviour of peat in these respects would seem to be at variance with that of clay, as described by Geuze and Tan (1953).

Triaxial Compression Tests

A typical triaxial test result is shown in Fig. 3. A specimen 1½ in. in diam. and 3 in. high was tested under a cell pressure of 10 lb/sq.in. The pore pressures were obtained from

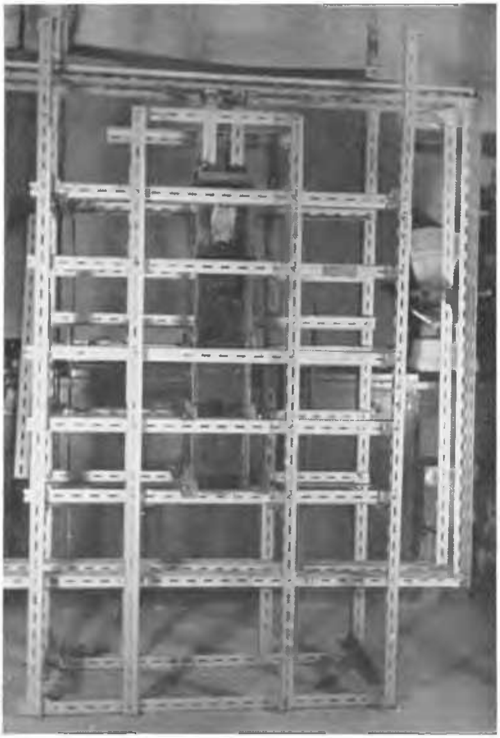

FIG. 1. 1 ft × 6 in. × 3 ft tank used in confined load test.

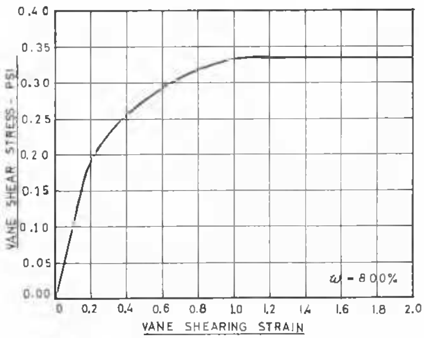

FIG. 2. Observed shear stress *vs.* shear strain relationship from vane tests.

direct observations carried out simultaneously, using both the probe and the base methods of measurement. Both sets of readings were identical. The main feature of the test is that even after a deformation of more than 30 per cent of its original length, the stress-strain increments were linearly related. The measured change in water content which took place during the test was virtually nil. Tests on a similar peat in connection with another project confirmed that, as in the case of the vane tests, the stress-strain behaviour of peat in the triaxial tests is similarly independent of the rate of strain application, at least for normal rates.

Pore water coefficients. The Skempton *A* and *B* pore water pressure coefficients were found to be 0.125 and 1.0 respectively (Fig. 4). The low positive value of the *A* coefficient indicates a material which undergoes negligible volume change on shear. In the early stages of shear the *A* coefficient is seen to be zero (Fig. 3).

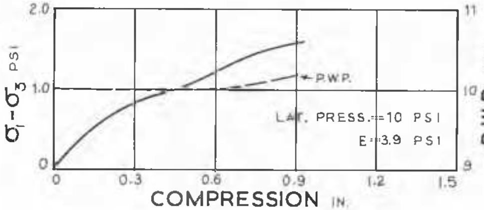

FIG. 3. Quick shear triaxial test with pore water pressure measurements.

Lateral strains in triaxial test. Fig. 4 shows values of lateral strains observed in accordance with the optical method of Escario and Uriel (1961). The method was found to be convenient, and appeared to yield reproducible results in a number of triaxial tests. The values of volumetric strain, derived from these observations in accordance with the recommended method, were found to be of the order of 20 per cent when the triaxial tests were discontinued. This value is not consistent with the fact that the specimens were completely air-free, and it was decided to discount the results. An investigation of this puzzling inconsistency suggested that under large axial deformations, the decrease in surface area of the membrane caused puckering or corrugation of the surface of the specimen. Tangential readings of the surface tended to be taken in the troughs of these corrugations, thus yielding a falsely low value of the volume.

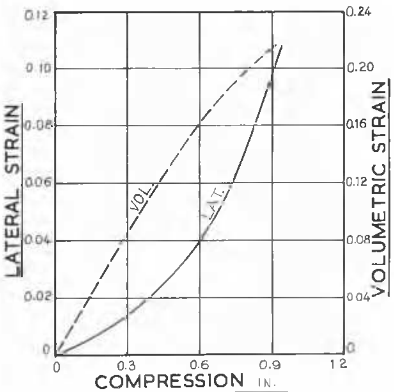

FIG. 4. Observed strains in triaxial test using optical method.

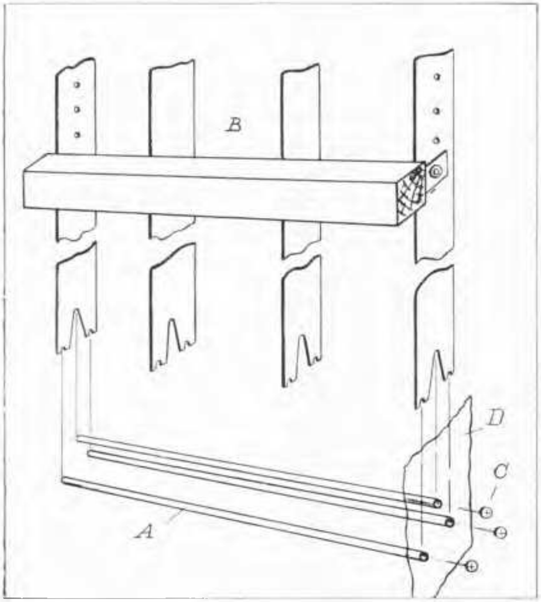

FIG. 5. Frame for inserting pins as strain gauge.

DIRECT OBSERVATION OF STRAINS

Method

A novel method was adopted comprising the photographic observation of three stainless steel tubes, A (Fig. 5) 1½ mm external diameter, 1 mm bore, arranged independently as an equi-angular rosette of ⅝ in. side. A special frame, B, to which the pins were attached by thin rubber bands, subsequently released, was devised for inserting the groups into the peat. To render the ends of the tubes visible, stainless steel pins, C, attached to soft iron discs of ⅛ in. diameter were inserted, with a sliding fit, into the tubes. The discs marked with a fine cross were photographed at three times full size. The strained lengths of the rosette were subsequently measured with a travelling microscope. To maintain the face of the discs in the same vertical plane, i.e. the inner face of the tanks, a magnet was used to pull the discs forward; this action also displaced any peaty water between the discs and the tank walls. This problem was troublesome, and it was finally necessary to fit a small vertical apron of thin flexible rubber membrane at the ends of the tubes, D. This expedient was successful in maintaining visibility of the discs for approximately 10,000 min.

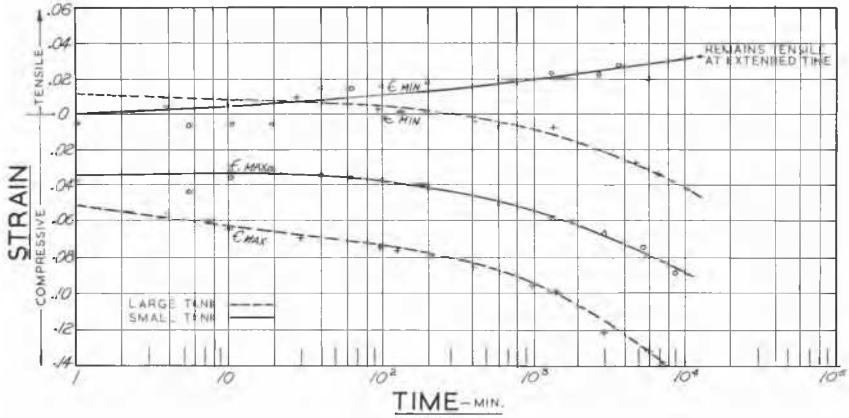

FIG. 6. Observed variation of principal strains with time.

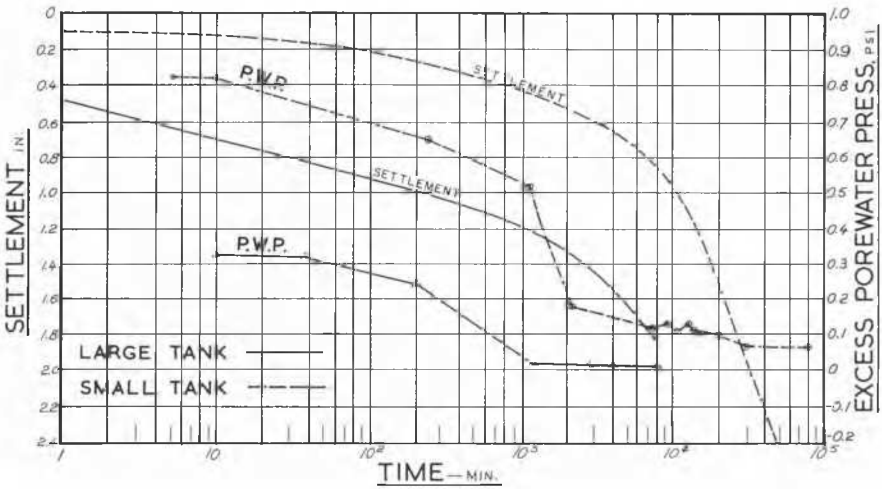

FIG. 7. Observed settlement and reduction of pore water pressure with time.

Beyond this interval, however, further trouble was experienced from folding of the rubber membrane.

Reduction of Readings

Fig. 6 shows a representative set of results for both tanks, presented as curves indicating the variation of principal strains over the period of observations. For convenience, the reduction was based on the assumption of infinitesimal strain, instead of using the theory of finite homogeneous strain as might be more appropriate. Meyer (1963), however, suggests that the former assumption may be more satisfactory in the present instance.

SETTLEMENT-TIME OBSERVATIONS

Fig. 7 shows the observed settlement-time relationships for a unit load of 0.82 lb/sq.in. on a rigid foundation block, area 1 ft by 6 in., for both small and large tanks. Fig. 7 also shows the measured excess pore pressures for points in both tanks 3 in. below the surface, and vertically under the centre of the block. Attention is directed to the shape of the pore pressure curves in the later stages. In numerous large consolidation curves previously carried out, measuring the pore pressure at the midpoint of the specimen, it has invariably been observed that the "tail" of the curve flattens out at some small positive residual value. This feature is repeated in the present instance at a point which is close to a "free" boundary. In the large tank the excess pressure seemed to be reduced virtually to zero, but even in this instance the decreasing slope of the "tail" of the curve indicates at least the possibility of a positive excess, although minimal. It may suggest the possibility of the rate of settlement being controlled by increasing tension in the pore water.

ELASTIC MODULI

The initial strains, stresses, and pore pressures are generally evaluated by the methods of elastic theory. This requires a knowledge of the so-called elastic constants, especially Poisson's ratio, which are difficult to determine. It was thought preferable to compute μ from the fundamental relationship.

$$G = E/2(1 + \mu) \tag{1}$$

in which the shear modulus is equated with the slope of the stress-strain curves from the vane tests. E is considered to be a constant. Table I shows the results of such a computation.

TABLE I. COMPUTED VALUES OF POISSON'S RATIO BASED ON EQUATION (1)

E (lb/sq.in.)	G (lb/sq.in.)	μ	Range (per cent of ultimate vane strength)
3.9	1.18	0.65	0–25
3.9	1.10	0.73	0–50
3.9	0.69	1.83	0–75
3.9	0.28	6.10	0–100

The slope of the vane test curve (Fig. 3), is extremely variable and four different values of the secant modulus were determined, over the ranges 0–25 per cent, 0–50 per cent, 0–75 per cent, and 0–100 per cent of the ultimate vane strength.

DISCUSSION OF TEST RESULTS

Unconfined Loading Test in Large Tank

If the three-dimensional effects which necessarily accompany large plastic strains are neglected, the loading system adopted in the large tank may be reasonably assumed to produce a condition of plane strain. Solving the basic equations of elasticity

$$e_1 = (1/E)\,[\sigma_1 - \mu(\sigma_2 + \sigma_3)] \tag{2a}$$

and

$$e_3 = (1/E)[\sigma_3 - \mu(\sigma_1 + \sigma_2)] \tag{2b}$$

for the condition $e_2 = 0$, yields the relationship

$$\frac{e_{\max}}{e_{\min}} = \frac{\sigma_1(1 - \mu^2) - \mu(1 + \mu)\sigma_3}{\sigma_3(1 - \mu^2) - \mu(1 + \mu)\sigma_1}. \tag{3}$$

The appropriate values of principal stresses may be derived from the Boussinesq equations for a strip footing $\sigma_1 = (p/\pi)(\psi - \sin\psi) = 0.671$ lb/sq.in. for $p = 0.82$ lb/sq.in. and $\psi = ½\pi$ radians, and $\sigma_3 = (p/\pi)(\psi - \sin\psi) = 0.149$ lb/sq.in. for $p = 0.82$ lb/sq.in. and $\psi = ½\pi$ radians. These values of principal stresses are independent of μ.

TABLE II. VALUES OF MAJOR PRINCIPAL STRAIN AND PRINCIPAL STRAIN RATIO FOR VARIOUS VALUES OF POISSON'S RATIO

(1) μ	(2) $\frac{e_{max}}{e_{min}}$ Equation (3)	(3) e_{max} Equation (6)
0.3		0.18
0.5	−1.0	0.14
0.65	−0.36	0.11
0.73	−0.16	
0.90		0.034
1.83	+0.61	
6.10	+0.90	

Table II shows the various values of the ratio of principal strains obtained by solving Eq (3) for the four computed values of μ in Table I, and also for the condition of volume incompressibility ($\mu = 0.5$). At the instant immediately following loading, the appropriate value of μ would appear to be within the range 0.5 to 0.65 and the principal strain ratio would therefore be within the range −1.0 to −0.36. This compares badly with the measured ratio of −4 at 1 min. If Eq (3) is solved for μ using the measured ratio, the computed value of μ becomes 0.3.

Estimation of Initial Pore Pressure

Scott (1963) has modified the well-known Skempton equation for the estimation of the initial pore water pressure (u) in a saturated soil under a three-dimensional stress system

$$u = \sigma_{oct} + (D/3C_s)\tau_{oct} \tag{4}$$

where

$$\sigma_{oct} = \tfrac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3),$$

$$\tau_{oct} = 1/3\sqrt{[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2]},$$

$$D/(3C\ ) = (3/2)(A - \tfrac{1}{3}).$$

For the condition of plane strain

$$\sigma_2 = \mu(\sigma_1 + \sigma_3).$$

Using the value of μ (0.65) indicated by the results of the vane tests the value of the pore water pressure (u) calculated from Eq (4) was found to be 0.35 lb/sq.in. On the other hand using the computed value of μ (0.3) obtained by inserting the measured value of the principal strain ratio into the left hand side of Eq (3) the value of u from Eq (4) equals 0.22 lb/sq.in.

The observed value of u (0.33 lb/sq.in.) is closer to the former value. Considerable scatter has frequently been experienced in carrying out repeat tests involving pore pressure measurements, and the fair conclusion to be drawn is that the measured value is of the right order of magnitude.

Confined Loading in Small Tank

In solving Eq (2a) and (2b) for one-dimensional loading in the small tank, it may reasonably be assumed that $\sigma_2 = \sigma_3$. The further assumption which is normally made and which would seem to be confirmed by the readings is $e_2 = e_3 = 0$ which yields the relationship

$$\sigma_3 = \sigma_1\ [\mu/(1 + \mu)]. \tag{5}$$

Inserting Eq (5) into basic Eq (2a) and (2b) yields

$$e_{max} = 1/E[\sigma_1 - 2\mu\{\mu/(1 + \mu)\}\sigma_1]. \tag{6}$$

Table II also shows the result of solving Eq (6) for various values of μ. Inserting the value of μ (0.65) indicated by Eq (1) the computed value of e_{max} was 0.11 compared to the measured value of 0.034 approximately. Using the previously determined value of μ (0.3) from the measurements made in the large tank the corresponding value of e_{max} was 0.18. Using the assumption of an initially incompressible material ($\mu = 0.5$), e_{max} was 0.14. Also shown on Table II is the value of μ computed from Eq (6) using the observed value of $e_{max} = 0.9$.

The accuracy of the minor principal strain measurements shown to be tensile at 10,000 min is confirmed by previous experience with peat. On completion of extended consolidation tests, it has frequently been observed that on removal of the vertical load the specimen may fall freely out of the container. On occasions tensile strains of the order of 0.04 have been observed.

CONCLUSIONS

The novel method of observing strains was found to be convenient and the measured values would appear to be reliable. The readings confirmed the accuracy of available methods of estimating initial values of pore pressure. The observed principal strain ratio for the unconfined condition differed considerably from the value computed from elastic theory, using either the assumption of incompressible material ($\mu = 0.5$) or a value of μ based on a comparison of vane and triaxial tests. These two tests yielded results not readily reconcilable. The shear characteristics of peat were found to be independent of the rate of strain adopted in the test, and flow deformation was negligible. There is a need for further research on the properties of peat in shear.

ACKNOWLEDGMENTS

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