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Time Dependent Lateral Pressures, Poisson's Ratio Measurement

Pressions Latérales Dependent du Temps, Mésurement du Nombre de Poisson

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SYNOPSIS Lateral pressures have been measured as a function of the time in the Gauge oedometer under various stepwise loadings. This report presents a new method for the determination of the time dependent Poisson's ratio and its theoretical basis. Implications for practical applications are discussed.

INTRODUCTION

The stress distributions in soil masses are known from practice to change with the time. From the theory of elasticity we know that the stress distribution in multiply connected bodies depends on the Poisson's ratio ; its visco elastic analogue in soils is a parameter dependent on the time and the current effective stresses. As the time effects of shear creep, volume creep and consolidation are mutually interrelated, it is very difficult to measure the Poisson's ratio in soils. In the Design of earthpressure structures as strutted excavations, retaining walls and underground structures it is not only necessary to estimate the stress distribution in the beginning but also how it could change with the time. It is interesting to note that the ultimate state of stress may be hydrostatic as that around the subways tunnel in London (Glossop, 1950). This report presents an experimental and theoretical study of factors affecting the gradual changes in stress distribution.

DESCRIPTION OF THE GAUGE OEDOMETER

The Gauge Oedometer is a compressiometer, whereby the lateral pressure can be measured as function of the time (Tan, 1957-1961) . The oedometer consists of two symmetrical rigid half rings tightly connected by two thin steel strips (Fig. 1). A lateral pressure against the ring assemblage gives a tensile strain in these strips which can be measured with the help of two active strain gauges ; these gauges together with two dummy gauges are connected in a full wheatstone bridge. This bridge operates under a three volt D.C. and every strain variation can be recorded with the help of a sensitive Galvanometer G. The circuit is temperature compensated. The floating ring is suspended by springs such that only vertical displacements are allowed (not shown in sketch). The connection of the two half rings are provided by a watertight seal to prevent leakage of pore water. For the calibration an artificial sample i.e. hollow rubber cylinder with flat ends is made; it is completely filled with water and tested in the oedometer. As the state of stress in this artificial sample is hydrostatic the apparatus can directly be calibrated.

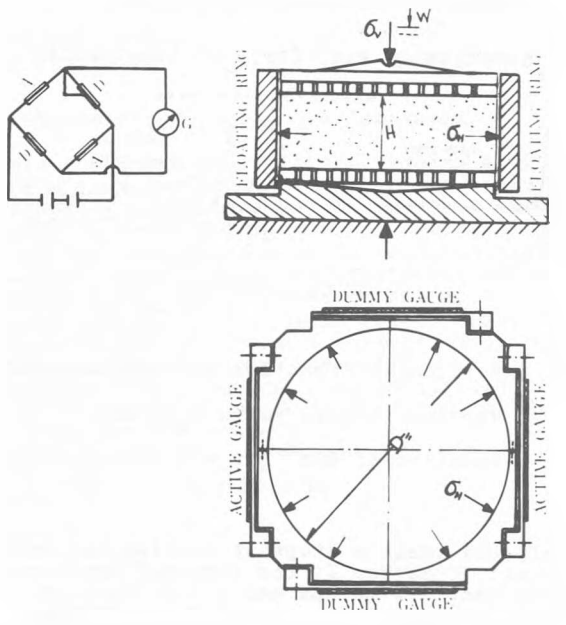

Fig. 1 Sketch Gauge Oedometer

DISCUSSION OF TEST RESULTS

The typical results of three series of tests will be presented and discussed:

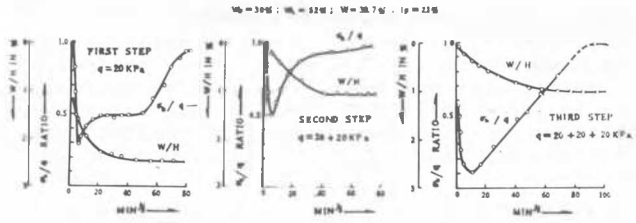

Fig. 2 Vertical Deformation w/H and lateral/ vertical stress ratio σ_h/q as function of $Min^{1/2}$ for a 3 step-loading

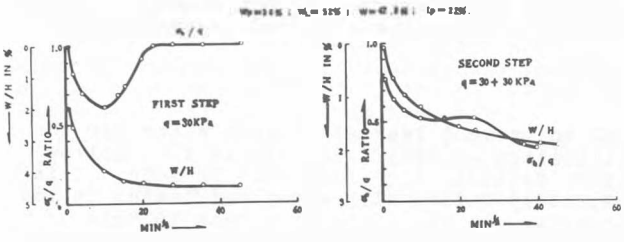

Fig. 3 Vertical Deformation w/H and lateral/ vertical stress ratio σ_h/q as function of $Min^{1/2}$ for a 2 step-loading

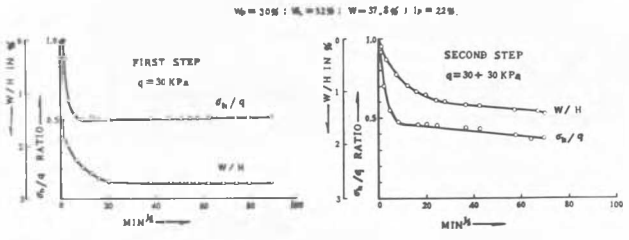

Fig. 4 Vertical Deformation w/H and lateral/ vertical stress ratio σ_h/q as function of $Min^{1/2}$ for a 2 step-loading

In all the tests a stepwise loading has been applied: in Series I, the vertical pressures q = 20 Kpa, q = 40 Kpa and q = 60 Kpa; in Series II and III two steps of q = 30 Kpa and q = 60 Kpa. The vertical deformation w/H and the ratio σ_h/q of the lateral to vertical pressure have been measured as a function of the square root of the time in minutes. Directly after application of the first step an instantaneous vertical deformation occured, which can be ascribed to the initially loose contact between porous plates and the sample: this phenomenon is not observed in further steps. In the first and second steps in series I (Fig. 2), W = 39.7%; the lateral pressure equals the vertical pressure instantaneously; this is an indication of full saturation. Due to the squeezing out of pore water the lateral pressure decreases, but simultaneously relaxation of deviatoric stresses within the sample occurs and stresses are transferred to the rigid ring resulting in the increase of the lateral stress. The result is a decrease in lateral pressure until a certain minimum, then gradually it rises up until a hydrostatic state of stress is attained.

In many cases the ratio can only reach a maximum value less than 1. It can be noticed that the relative compression decreases with the number of steps, i.e. that volume hardening due to the increase of the number of interparticle contacts has occured.

So in the third step (Fig.2c) the initial ratio is only 0.55, which then quickly decreases and gradually rises up until 1. This is an indication that under a vertical pressure of 60 Kpa yield values and frictional effects have been created and mobilised.

In Series II, W=42.8%; the first step (Fig.3a) shows the same tendency as Series I. In the second step (Fig. 3b) the hardening is obvious from the initial stress ratio of 0.8 and the delay in attaining a minimum.

In Series III, W=37.8%; the stress ratio decreases continuously. No rise has been observed within the period of one week. This is a clear indication that the viscosity and the time of relaxation has considerably increased. Due to the increased volume hardening, the number of interparticle contacts has become so large that the sliding and jumping of bonds between the particles becomes far more difficult. From the above results it can be concluded that in soils with higher watercontents the shearing stresses may quickly relax to zero and a state of hydrostatic equilibrum can be attained within a week. Stresses relaxing to zero have been meassured in a Relaxation Plastometer for Harbin Clay (Tan 1957).
For soils with lower porosities and water contents this process may be delayed. As we will discuss hereafter the state of stress is not hydrostatic if hardening effects may make the viscosity infinite (limited creep).

THEORETICAL ANALYSIS

In the lateral pressure three time factors are obviously interrelated, the waterpressure, the creep, relaxation of the soil skeleton and the hardening of this structure:
1. In fully saturated samples the waterpressure U_w instantaneously assumes the value $U_w = q$ after loading; then the porewater is squeezed out, the water pressure quickly decreases and gradually approaches zero in the secondary consolidation period (Tan 1959);
2. The horizontal pressure assume its initial value during the elastic deformation $\sigma_h = q\nu_e/(1-\nu_e)$; due to the relaxation the stresses are gradually transferred to the most rigid part of the assemblage i.e. the oedometer rings and consequently σ_h increases approaching $\sigma_h = q\,\nu_u/(1-\nu_u)$;

3. In the samples with higher porosity and watercontents the stress deviator $\sigma_v - \sigma_h$ relaxes to zero and an ultimate state of hydrostatic stress is attained after 5 to 10 days of loading. In samples with decreased watercontents due to 5% compression however, the state after a week is not hydrostatic as noticed from $\sigma_h / q < 1$. Further the stress ratio is less than one, directly after application of the loading; these are indications that frictional effects and stronger interparticle bonding forces have created, whereas the number of interparticle contacts must have been considerably increased due to volume hardening.

Three types of behaviour can be distinguished:

1. σ_h/q decreases → minimum → increases → maximum → $\sigma_h/q \to 1$
2. σ_h/q Decreases → minimum → increases → maximum → $\sigma_h/q < 1$
3. σ_h/q decreases gradually → limit; no increase observed in period of observation.

The second case has been observed for a dense, fat clay with a water content of W = 21.5% and $c_v = 18 \times 10^{-8} m^2/sec.$, $\nu_e = 0.41$, $\nu_u = 0.44$ under q = 21 Kpa (Tan 1961).

The same features as in Type 2 can be noticed from the test results by Vidmar (Suklje), who studied stress relaxation phenomena in earth pressure laboratory models. The stress ratio first decreased to a minimum and then gradually increased to a 0.8 within a period of 1.2×10^6 sec. As the rigid wall is allowed to move laterally and to rotate, the measured lateral pressure must be lower than in the case of a completely fixed rigid wall as in the gauge oedometer.
The above findings can be expressed mathematically as follows. Assuming homogeneity and isotropy the stress- strain relation for stresses below the upper yield value can be written:

$$\sigma(t) = I\int_{-\infty}^{t} \left\{K(t-t^*) - 2G(t-t^*)/3\right\} \mathrm{tr}\dot{e}(t^*)\, dt^* + 2\int_{-\infty}^{t} G(t-t^*)\, \dot{e}(t^*)\, dt^* + U_w I; \qquad (1)$$

Whereby $K(t^*)$ and $G(t^*)$ are functions of the effective stress-invariants; after Laplace transformation, thereby neglecting the history up to t = 0, and for simplicity not considering the time effects in the stress invariants:

$$\bar{\sigma} = I\,(\bar{K} - 2\bar{G}/3)\ \mathrm{tr}\bar{e} + 2\bar{G}\bar{e} + \bar{U}_w I; \qquad (2)$$

$$\bar{\sigma}^* = \frac{2\bar{G}(1+\bar{\nu})}{3(1-2\bar{\nu})}\,\bar{e}^* + \bar{U}_w \qquad (2a)$$

Whereby:

I = unit matrix, σ = stress matrix;

e = strain matrix; $\mathrm{tr}e = e^* = e_1+e_2+e_3$ = volume strain; $\sigma^* = (\sigma_1+\sigma_2+\sigma_3)/3$ = hydrost. pressure;

K_e = elastic bulkmodulus : G_e = elastic shear modulus; U_w = waterpressure;

$\bar{G} = G_e/(1 + s\bar{G}_v)$; $\bar{K} = K_e/(1 + s\bar{K}_v)$;

K_v = transform of volume creep function, which is a monotonously increasing function, approaching a finite limit;

G_v = shear creep operator whereby four types can be considered:

creep with log t: $\bar{G}_v = G_e/(1-\phi_o \log Cs)$;

creep with t: $\bar{G}_v = G_e\eta s/(\eta s + G_e)$;

creep with t^r: $\bar{G}_v = G_e s^r/(\phi_o + s^r)$; for $0<r\leq 1$

creep with definite limit: $G_v = (A_n s^n + .. A_o)/(B_n s^n + .. B_o)$;

η = viscosity; $\phi_o, A_n \ldots B_n$ = Constants.

In all the above and further formulas a bar above a quantity denotes its Laplace transform. In practice creep limited to a certain amount is only observed for very small stresses and in the case of poor silty clays, whereas the first type is most general. As we have analysed above the increase of contacts between the particles due to volume compression makes the sliding and disrupting of inter particle bonds more and more difficult and the viscosity increases and even can become infinite (see formula (8)). From (2) the following formulas can be derived;

$$\bar{\sigma}_h/q = 1 - 2\bar{G}\,\bar{w}/Hq \qquad (3)$$

$$\bar{\nu} = \frac{1}{2}\left(1 - \frac{\bar{G}}{\bar{K} - \bar{G}}\right) \qquad (4)$$

The second term in the right part of (3) expresses the effects of the relaxation of shearing stresses and the waterpressure which is contained in w. By measuring the stress ratio and the vertical deformation as function of the time it is possible to evaluate the shear operator $\bar{G}_v$. So the Gauge oedometer is essentially a relaxation plastometer for confined conditons.

The Poissons ratio ν_e, which is the initial value of the operator of lateral expansion can be computed from the following formulas:

$$\frac{w}{H} = 2q\, m_{vc}\left\{\frac{C_v\, t}{\pi H^2}\right\}^{1/2} \quad ; \quad T_v = \frac{C_v t}{H^2} \quad ; \qquad (5)$$

$$\frac{\sigma_h}{q} = 1 - \frac{2(1-2\nu_e)}{1-\nu_e}\left\{\frac{C_v\, t}{\pi H^2}\right\}^{1/2} \qquad (6)$$

The formulas (5) and (6) have been established for the initial process, where all deformations are mainly elastic and where the effects of the creep and relaxation are negligible.
The ultimate value of the operator for lateral expansion can be obtained from

$$\frac{\sigma_h}{\sigma_v} = \frac{\nu_u}{1-\nu_u} \qquad (7)$$

By plotting the stress ratio σ_h/q and the vertical deformation w/H as a function of the square root of the time and making use of (5),(6),(7) and Taylor's fitting method the following table has been obtained.

	C_v in $10^{-8}m^2$/sec.	ν_e	ν_u
Series I			
Step I	16	0.16	0.5
Step II	18	0.23	0.5
Step III	10	0.0	0.5
Series II			
Step I	20	0.27	0.5
Step II	12	0.25	0.25
Series III			
Step I	30	0.27	0.33
Step II	11	0.09	0.23

In an earlier theory on the consolidation and flow of clay layers the volume deformations are considered to be elastic, whereas a Maxwell creep linear with t is assumed for the shear behaviour of the soil skeleton (Tan 1953); These assumptions imply that the long term state of stress will be hydrostatic. Experimental evidence for such behaviour are shown in the typical examples of the Figs 2 and 3A. Furthermore the state of stress around the subway tunnel in London is hydrostatic many years after the end of the construction, (Glossop 1950). The formulas (2),(3) and (4) show us that if the creep will not stop, but proceeds continuously - independently of the manner it increases with the time, as log t, t^r, t etc - then $G_v \to 0$ and $\nu_u \to 1/2$ for $t \to \infty$ and $s \to 0$. As the relaxation times of soils have been determined in the order of days,weeks months, years and even longer it can be expected that a possible state of hydrostatic stress may be attained in practice after months, years and even much longer. For soils with logarithmic creep at very slowly decelerating rate,this process is much slower.On the other hand if the creep may come to a standstill and thus the shearing deformations will be limited, then the ultimate state of stress will not be hydrostatic. The Figs 3B and 4 show us that there is a critical compression,whereby the stress ratio does not rise up i.e. the viscosity becomes infinite, when the compression attains a critical value e_{cr}.
A tentative formula is given as follows:

$$\eta = \eta_0 / \left(1 - \frac{e}{e_{cr}}\right)^n \quad ; \qquad (8)$$

which describes the hyperbolical increase of the viscosity with the compression.

CONCLUSIONS

For engineering designs it will be desired to have an idea of the initial and ultimate stress distributions in soil masses;for these purposes Gauge oedometer tests must be combined with creep and or relaxation tests. In this way we can get the data and functions needed for computation analysis: shear creep function, relaxation function, the elastic parameters G_e and ν_e, the coefficients of compressibility m_{vc} and consolidation C_v, the ultimate Poissons ratio ν_u and the bulkmodulus, the hardening.
In constructions it is important to design an effective drainage system in order to reduce the waterpressure and to achieve an efficient volume hardening of the soil skeleton to arrest the creep. In this way a reduction of the lateral pressure in earthpressure structures can be achieved.
A finite element analysis is very helpful in getting an idea of the stress and deformation fields to be expected and the efficiency of our measures.

REFERENCES

Glossop, R. (1950) Geotechnique. I.

Tan Tjong-Kie (1953). Proc. III ICSMFE,III,196; (1957). Three dimensional theory on the consolidation and flow of clay layers, Scientia Sinica 6, (1), 203; Structure Mechanics of Clays, Scientia Sinica 8, (I), 83;(1961) Proc. IV ICSMFE. III 40 (disc.);(1961) Consolidation and secondary time effects of homogeneous, anisotropic, saturated clay strata, Proc. V ICSMFE, I (63),367;(1964) Determination of the rheological parameters and the hardening coeficients of Clays:Rheology and Soil Mechanics,IUTAM Symp,Grenoble.

Vidmar, S and Suklje, S. (1969) Relaxation effects on the earth pressure, in Rheological aspects of Soil Mechanics, 571 pp, Wiley- Interscience.