Behaviour of Clays Close to Failure

Le Comportement des Argiles près de la Rupture

G.TER-STEPANIAN Lab of Geomechanics, Geological Institute, Armenian Academy of Sciences, Yerevan, U.S.S.R.

SYNOPSIS Creep of a clay during shear consists of mobilization and rupture phases characterizing by decrease and increase of the strain rate, correspondingly; these processes proceed proportionally with time. The increase of the strain rate in the rupture phase does not mean that the creep will end in failure. The failure takes place only at high stress level (in studied soils at $f = \tau/\sigma = 0.55$) and comparatively short stress age (100-200 days). Prolonged testing (over five years) shows that at slightly lower stress level ($f = 0.50$) the soil does not fail even more than 2000 days later and the creep proceeds with a decreasing rate ($\dot{\gamma} \simeq 10^{-11}$ sec^{-1}). This fact enables to increase considerably the design shear stresses of soils without fearing that the creep will end in failure after a very long time.

The practical necessity of estimation of the long term shear strength of soils at the stress level close to failure is doubtless. However the theoretical concepts and experimental data relating to rheological behavior of clays at a very long duration of stress action are still extremely insufficient.

The traditional understanding of the long-term shear strength of soils is based on the known interpretation of experimental creep curves (shear strain vrs. time). Failure is defined as that point on the creep curve at which the strain begins to increase i.e. where the strain acceleration becomes positive (e.g. Schmid, 1962).

Theoretical study and experiments have shown a picture which considerably differs from the above-mentioned (Ter-Stepanian,1975) The new physical theory describes the process of creep at shear as follows.

The soil structure is characterized by the type and arrangement of particles and therefore by a certain system of contact points and forces acting between them. Thus each soil structure has a limited possibility for deformation (endurance) and hence a limited strength. When the accumulated shear stress exceeds this value the soil structure becomes unable to sustain the stress and a jump-like reorganization of the soil structure takes place (Ter-Stepanian, 1936). Mechanical properties of the new-built structures are determined by accidental causes; therefore the jump-like reorganization of soil structures should be defined as a stochastic process.

Contrary to this the deformation of each soil structure proceeds smoothly obeying to the rheological equations deduced from the theory of rate processes (Eyring, 1936); therefore it is a deterministic process.

Based on the Eyring's theory, supplemented by Mitchell et al.(1968), the Author has deduced rheological equations taking into account several additional factors, such as shear stress level τ, soil structure deformability F/R, statical viscosity η, stress age t and average lifetime of bonds L. Depending on the direction of the soil structure change the lifetime of bonds increases or decreases; correspondingly the creep process consists of two phases: <u>mobilization</u> and <u>rupture</u>.

In the initial phase of mobilization the quantity of defects of soil fabric decreases, the soil structure becomes more regular and orientation of soil particles occures; therefore the lifetime of bonds increases. Assuming that the strain rate $\dot{\gamma}$ decreases directly with time we have

$$\dot{\gamma} = a(\tau - \tau_p)/\eta\, t \quad . \; . \; . \; . \; . \; . \; (1)$$

where a is the structural coefficient depending on the soil structure deformability F/R and on the average tangential force f_1, acting in each flow unit in points of contact and τ_p is the shear stress by which no creep occurs (creep limit).

In the final phase of rupture all these processes develop in the opposite direction, the lifetime of bonds decreases and the strain rate increases directly with time

$$\dot{\gamma} = a(\tau - \tau_p)t/\eta\, t_m^2 \quad . \; . \; . \; . \; . \; (2)$$

where t_m is the mobilization time. The outlined changes of structural defects during creep were shown by Vyalov et al. (1972).

Integrating Eqs.(1) and (2) we receive the magnitude of the creep strain for each structure.In mobilization phase it will be

$$\gamma = a\frac{\tau - \tau_p}{\eta} \ln \frac{t + \Delta t}{\Delta t} + C \quad . \; . \; . \; (3)$$

where Δt is a small time interval equal to one unit, e.g. one second. In the rupture phase it will be

$$\gamma = a(\tau - \tau_p)\; t^2/2\eta\, t_m^2 + C \quad . \; . \; (4)$$

The creep curves for successive soil structures consist of line segments of loga-

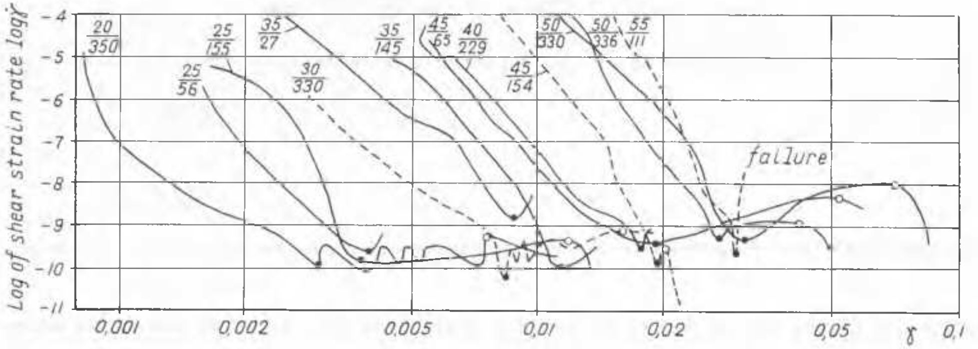

Fig. 1. Intrinsic curves for tests at normal stress σ = 100 kPa. Figures above denote shear stress in kPa and below, duration of test in days. Black circles denote the mobilization limit and white circles - the stabilization limit. Only one test at τ = 55 kPa (duration 111 days) was ended in failure; in remaining tests the process was gradually stabilized, if the test lasted more than about 200 days.

rithmic curves in the mobilization phase and of parabolic segments in the rupture phase; atjump-like reorganization of soil structures the process passes over from one possible curve to the next intersecting one.

These theoretical principles were confirmed by 20 creep tests carried out with two samples of undisturbed highly plastic sensitive overconsolidated diatomaceous clay from Sissian (Armenia) from the depth 40 and 76 metres. Some tests lasted over five years; a part of tests is still continuing. The stress levels are expressed as $f=\tau/\sigma$.Their discussion refers to the studied soils only.

The experiments enable to detect an important phenomenon directly concerned with the problem under consederation. The mobilization limit takes place at the stress age about 10-15 days, then the strain rate increases (Eq.2); this is true for all tests at shear stress exceeding the creep limit ($f\simeq 0.15$).As regards the traditional definition in all these cases the test should sooner or later end in failure since the strain rate accelerates. It follows from this that the long-term shear strength is very small (equal to the creep limit). This conclusion is inconsistent with the fact of existence of high natural slopes in clays where the shear stresses along the potential surface of sliding exceed considerably the creep limit.

Prolonged tests have shown that in most cases this increase of the strain rate continues only to a certain stabilization limit; afterwards the strain rate decreases uninterruptedly. A decrease of the strain rate after some increase was first observed by Bishop and Lovenburry (1969). Continuous increse of the strain rate leading to failure was detected only at high stress level (f = 0.55); the failure sets in after 100-200 days. At somewhat lesser stress level (e.g. at f = 0.50), the failure does not set in even after 2000 days (more than five and the creep proceeds with a decelerating rate ($\dot{\gamma} \simeq 10^{-11}$ sec^{-1}).

Fig. 1 shows intrinsic curves (strain rate vrs. strain in log scale). The turning point of intrinsic curves corresponds to the maximum strain rate $\dot{\gamma}_s$ in the rupture phase; it is denoted as stabilization limit S. Thence the strain rate decreases continuosly. This suggests that the increase of the strain rate after its decrease does not mean that the process should undoubtedly end in failure.

CONCLUSION. In a vaste domain between the creep limit ($f \simeq 0.15$) and failure (f=0.55) the creep process is stabilized, although a temporary increase of the strain rate took place. This enables to increase considerably the design stress for shear without fearing that the soil may fail after a very long time.

REFERENCES

Bishop A.W. and Lovenburry H.T.(1969).Creep characteristics of two undisturbed clays. Proc.7th Int.Conf.S.M.F.E., v.1, pp.29-37.

Eyring H.(1936). Viscosity, plasticity and diffusion as examples of absolute reaction rates, Journ.Chem.Phys., v.4, pp.283-291.

Mitchell J.K., Campanella R.G. and Singh A. (1968), Soil creep as a rate process. Journ. Soil Mech.Found.Div., Am.Soc.Civ.Eng. v.94, SM 1, pp. 231-253.

Schmid E. (1962).New concepts on shearing strength for saturated clay soils, pt.2, Sols-Soils, t.1, no. 2, pp. 19-28.

Ter-Stepanian G.(1936), On the influence of the scale-like shape of clay particles on the process of shear in soils, Proc. 1st Int.Conf.S.M.F.E., v.2, pp.112-116.

Ter-Stepanian G.(1975). Creep of a clay during shear and its rheological model, Géotechnique, v. 25, no. 2, pp. 299-320.

Vyalov S.S.,Zaretsky Yu.K.,Maximyak R.V. and Pekarskaya N.K.(1972).Problems of structural rheology of clays, Bull.Int.Ass.Eng.Geology, Paris, no.5, pp. 79-83.