

# INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



*This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:*

<https://www.issmge.org/publications/online-library>

*This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.*

# The Long-term Strength of Clays and Depth Creep of Slopes

## La Résistance à longue Échéance des Argiles et le Fluage en Profondeur des Pentés

by M. GOLDSTEIN, Professor, Chief of Soil Mechanics Laboratory of the Railway Engineering Institute, Dnepropetrovsk, U.S.S.R.

and

G. TER-STEPANIAN, Chief of the Laboratory of Soil Mechanics and Engineering Geology, Geological Institute, Armenian Academy of Sciences, Erevan, U.S.S.R.

### Summary

In the first part the relationship between the strength of clays and the duration of action of the load is discussed. The appropriate empirical formula is given. The influence of reloading samples and of rest periods on the long-term strength is described. Experiments have shown that the strain at the moment of failure of a sample is independent of the duration of action of the load. A method of determining the long-term strength using only one sample is proposed.

In the second part the so-called depth creep of slopes, a preparatory phase to the sliding, is discussed. The depth creep deformations are determined by values of the mobilized shear coefficient, which corresponds to the tangent of angles of maximum deviation. The nature of the difference between depth creep zones and sliding surfaces is explained and the expression for the velocity of the depth creep is given.

The first part is written by M. Goldstein, the second by G. Ter-Stepanian.

### Sommaire

Dans la première partie on traite de la relation entre la résistance de l'argile et le laps de temps pendant lequel l'argile est soumise à une charge. On donne les formules empiriques et on expose l'influence sur les échantillons de nouvelles mises en charge et de périodes de repos sur la résistance à long terme. Les essais ont démontré que la déformation de rupture d'un spécimen est indépendante de la durée de mise en charge. On propose une méthode pour déterminer la résistance à longue échéance à l'aide d'un seul spécimen.

Dans la seconde partie, on traite du 'fluage en profondeur des pentes', phase initiale du glissement de terrain. Les déformations de fluage en profondeur sont déterminées à l'aide du coefficient de cisaillement mobilisé, qui correspond à la tangente de l'angle de déviation maximum. On explique la différence entre les zones de fluage en profondeur et les surfaces de glissement et on donne l'expression de la vitesse de fluage en profondeur.

La première partie est écrite par M. Goldstein, la seconde par G. Ter-Stepanian.

### The Long-term Strength of Clays

When a load acts on a clay for a long time its strength is seen to decrease to a value depending on the duration of loading (Fig. 1).

The speed of loading in the ordinary strength-test of clays is usually fixed at some suitable value; therefore we shall call the

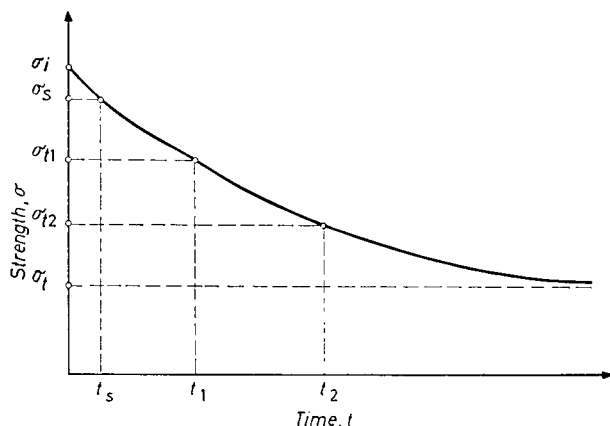


Fig 1

strength obtained in such tests the standard strength ( $\sigma_s$ ) of the soil.

The term 'instantaneous' strength ( $\sigma_i$ ) denotes the quantity obtained by extrapolation of test data back to  $t = 0$ .

The strength that corresponds to a given duration of load action is called the 'long-term' or 'current' strength ( $\sigma_t$ ).

The intensity of inter-particle bond depends both upon the distance between particles and their relative arrangement (LAMB, 1953). The particles move relative to each other

during deformation, and some bonds are destroyed while other new ones are formed.

The majority of the bonds require a long rest for their renewal; therefore, a continuous deformation must inevitably bring about a gradual weakening of the clay.

The inter-particle bonds may be divided into two groups: (1) brittle and (2) viscous bonds.

Those that permit elastic deformations and fail when the latter reach some defined magnitude are referred to as brittle. The viscous bonds give 'liquid-like' properties to bodies.

One of the possible simplified models of such an elasto-viscous body is shown in Fig. 2 (rheological designation M|H). In this scheme the soil is considered as consisting of two linked elements working together, one of them being an elastic framework and the other elasto-viscous filler.

The load on such a system first is taken up by elastic elements 1 and 2 (instantaneous deformation); then the viscous flow of element 3 begins with corresponding re-distribution of forces and growth of the part that is taken up by elastic springs 1. Finally the force will be transmitted entirely to these springs and the deformation will stop. In this case we have the phenomenon of creep. If in the process of deformation the forces in springs 1 exceed their strength, they fail and the total load will be taken up by viscous medium 3. It means that the flow of material has arrived at a point which must end in its inevitable rupture.

The scheme described permits us to determine the long-term strength limit as the maximum load not rupturing the elastic elements 1 (BIKOVSKY, 1954).

In our investigations samples of different undisturbed and remoulded clays were tested in the unconfined and triaxial compression apparatus. The samples were of stiff and hard consistency.

The experimental data lie in good agreement on straight lines

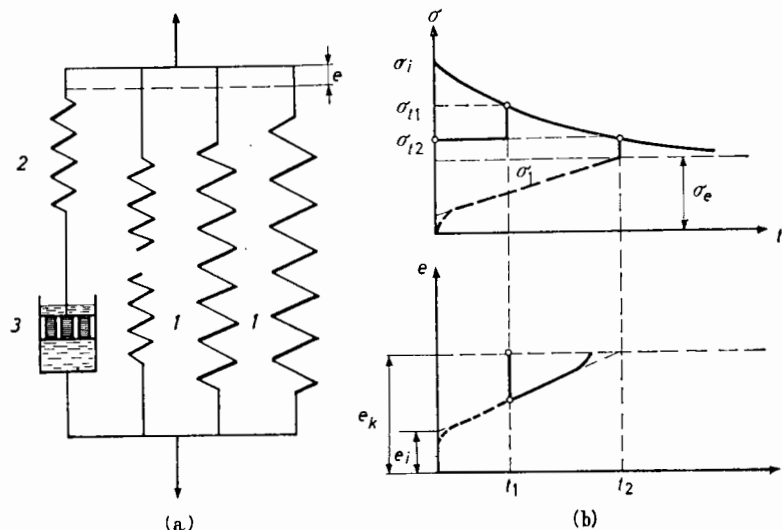


Fig. 2

on a semi-logarithmical scale and may be expressed by the formula:

$$\sigma_t = \sigma_o \log \frac{T}{t} \quad \dots (1)$$

or

$$t = T e^{-\sigma_t / \sigma_o} \quad \dots (2)$$

where  $T$  and  $\sigma_o$  are the characteristics of the long-term strength of a given clay.

The unloading and rest experienced by the sample influence the process only when they take place within the steady-flow part of the flow curve. In one of our tests, for example, the soil was a loam with  $LL = 31.9$ ,  $LP = 16.6$ ,  $W = 20.8$  per cent. The deformation assumed the character of an accelerating flow

per cent) remained practically the same and was equal to 0.03 at the moment of failure (Fig. 4).

If the hypothesis is confirmed by further experiment and the limits of its validity are known we will be able to establish the long-term strength curve of clays by testing only one sample.

Actually, the usual recommendation to test a new sample for every experimental point on the long-term strength curve is very troublesome and in some cases even unrealizable.

We suggest the following way of conducting such tests.

The load on the sample is increased by increments and the flow under each increment is observed until it attains a constant speed (stationary flow state).

When four or five different speeds of flow have been ob-

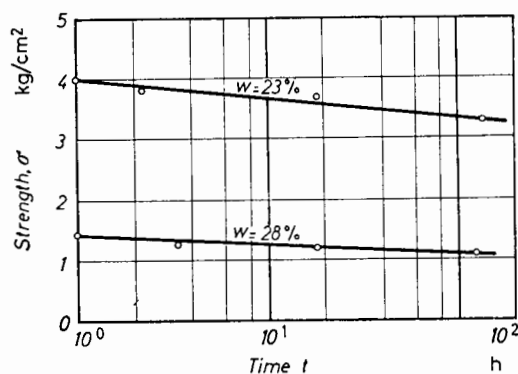


Fig. 3

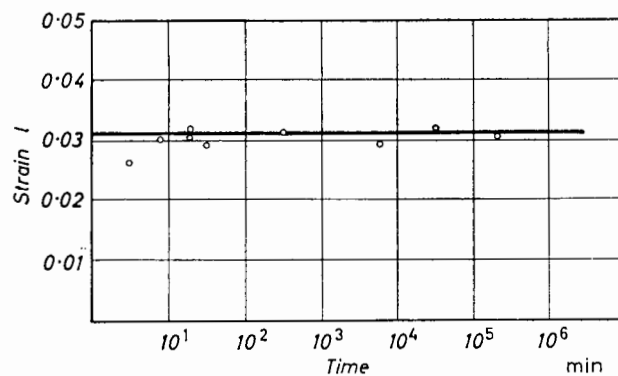


Fig. 4

after 90 days from the beginning of the test ( $\sigma_{III} = 3 \text{ kg/cm}^2$ ;  $\sigma_I - \sigma_{III} = 2.2 \text{ kg/cm}^2$ ).

At this moment the deformation of the sample was artificially stopped without unloading, and relaxation allowed for 17 days. Then the sample was allowed to deform freely under the same load, but the deformation now continued much less rapidly and the soil was again in a state of stationary flow.

It is a very important experimental fact that when all the conditions of the test (the size of sample, the consistency of the clay, the character of stress state, etc.) are the same the samples under different loads failed at a different time, but the deformation attained the same magnitude.

When, for example, the duration of the test changed from 3 minutes ( $\sigma_I - \sigma_{III} = 3.8 \text{ kg/cm}^2$ ) to 160 days ( $\sigma_I - \sigma_{III} = 2.1 \text{ kg/cm}^2$ ) the strain of all the samples of illite clay ( $W = 23.3$

tained, the load on the sample may be increased at the usual speed until the rupture takes place; in this way the total rupture strain may be determined.

Subtracting the instantaneous strain from this total strain, we obtain the steady-flow deformation. Dividing the latter by the average speed of flow under the given increment, we may find the time that would correspond to the moment of failure under this load (Fig. 5).

This should be done for every increment of load under which the flow-deformation takes place.

In our tests the samples of different clays, when tested as a closed system (undrained), decreased their strength during 1 to 3 months by 1.5 to 3 times; but when tested as an open system (allowing drainage) the samples did not fail under the same loads even when the tests lasted very much longer.

This method, apparently, is admissible when the standard quick shear test gives practically the same strength of clay if tested both before and after the long flow deformation as a closed system.

When the water-escape from samples during the test is possible (an open system), the creep or flow partly takes place simultaneously with the consolidation, but after the latter has ended they may go on further. In such cases, however, the samples fail under much greater loads than in the case of tests with a closed system.

### On the Depth Creep of Slopes

Slow gravitational movements of the material underlying slopes have been studied by many investigators; recently they have been considered by TERZAGHI (1950-53), who distinguishes 'continuous creep' or 'mass creep'.

One type of movement represents the phenomena which occur in the landslide-nidus, i.e. in that region of the slope body where the local concentration of tangential stress occurs. These movements occur preceding collapse of the slope.

Slow movements of this kind are called 'depth creep of slopes' (TER-STEPHANIAN). They are manifested more or less continually, although their speed varies with the changes of the intensity of landslide-producing agents. The complete cycle of sliding consists of the phase of depth creep, which may last

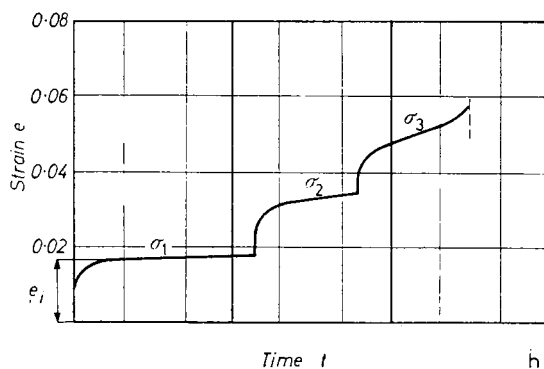


Fig. 5

for years, and the phase of shear, when a quick displacement occurs in a short time.

For the quantitative estimation of the rate of depth creep the following approximations to the deformative properties of soils at shear may be made:

(1) In the test we distinguish three successive phases according to the shape of the time-displacement curves for different increments of tangential stresses: stability, creep and shear. After the shear has been reached, the resistance decreases to its ultimate value  $\tau_c$ .

(2) A linear relationship in the phase of creep is observed between the value of tangential stress and velocity of deformation up to a definite time after load application. The slow plastic flow with constant rate starts when the bond resistance  $\tau_o$  (Hvorslev, 1937) has been reached.

(3) In a narrow range of normal stress, at the same value of tangential stress, there exists approximately an inverse ratio between the values of normal stress and the reduced shear deformations  $\gamma_o = \gamma/h_o$ , where  $\gamma$  = absolute shear deformation and  $h_o$  = reduced height of the soil sample. Hence, if the stress conditions of soil are expressed by the ratio of the tangential stress,  $\tau$ , to the normal stress,  $\sigma$ , or the shear coefficient  $f = \tau/\sigma$ , the reduced shear deformations can be considered as a function of this argument only, independent of the absolute values of the normal and tangential stresses.

In the stressed material the deformations occur along those

elementary surfaces where the ratio of tangential to normal stresses is the highest, i.e. along the surfaces of the angle  $\rho$  of maximum deviation of resultant from the normal. Hence, the tangents of the angles of maximum deviation are equal to the mobilized shear coefficients  $f$  of soil

$$\tan \rho = f \quad \dots (3)$$

Since shear deformations are defined by values of shear coefficients only, the slope can be divided into three zones according to the isogonic lines  $\rho$ , i.e. the lines of equal values of angles of maximum deviation: *stability zone*, outside the bounds of the isogonic line  $\rho_o = \arctan f_o$ , where  $f_o = \tau_o/\sigma$ ; *creep zone*, confined between isogonic lines  $\rho_o$  and  $\rho_c = \arctan f_c$ , where  $f_c = \tau_c/\sigma$ ; *shear zone* within the bounds of the isogonic line  $\rho_c$ .

The boundaries of these zones shift with changes of the stress conditions of slope. When the factor of safety of the slope

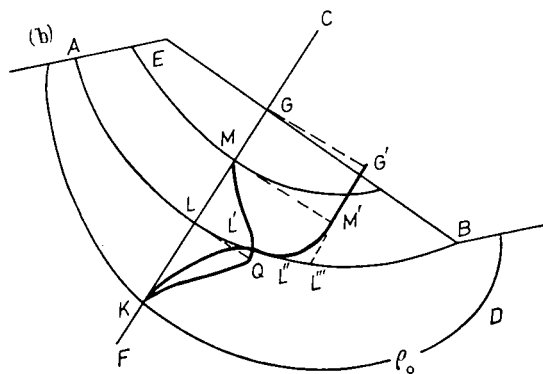
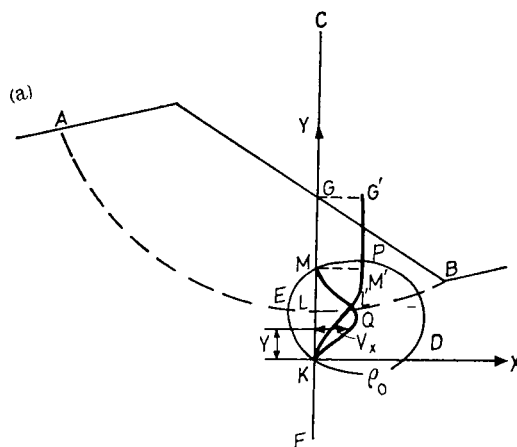


Fig. 6

decreases, island-like shear zones arise or develop along the potential surface of sliding, beginning from the landslide-nidus. When these zones occupy the whole surface, a catastrophic shear takes place.

The formation of broad deformation zones in the phase of depth creep and surfaces of sliding in that of shear is illustrated by Fig. 6. Fig. 6a shows a slope in the phase of depth creep. Here  $AB$  = potential surface of sliding, simplified by a circular arc;  $KEMD$  = isogonic line  $\rho_o$ ;  $FKLMG$  and  $FKL'M'G'$  = a segment of a line before and after deformation, correspondingly. Within the zone of depth creep  $KLM$  the flow of material proceeds with the maximum gradient of velocity at  $L$  (curve  $KQM$ ). In the area  $GMP$  no deformation occurs, but a total displace-

ment takes place due to the flow of the underlying zone of creep.

In the phase of shear (Fig. 6b) the zone of depth creep expands. To the creep  $LL'$ , in the surface of sliding  $AB$ , is added a considerably greater shear deformation  $L'K''$  (for the sake of convenience in drawing, the distance  $L'L''$  is abbreviated. Since the depth creep deformation  $LL' + L'L''$  is negligibly small compared with the shear deformation  $L'L''$ , practically the total observed displacement  $GG'$  is due to the latter. Only in this sense can one speak of the transition from creep zones to shear surfaces.

The rate of soil deformation differs within the zone of depth creep. On the boundaries of this zone (points  $K$  and  $M$ , Fig. 6a), the relative velocity equals zero while at the potential surface of sliding (point  $L$ ) it is a maximum. The functional relationship between the velocity of depth creep and the distance  $y$  from the origin of coordinates  $K$  is determined by the law of distribution of values of angle of maximum deviation.

For the approximate determination of the velocity of slope deformation in the phase of depth creep the Bingham rheological equation can be used

$$\tau - \tau_0 = \eta \frac{dv}{dy} \quad \dots (4)$$

where  $\tau$  = tangential stresses, acting on the elementary surface at the angle of maximum deviation,  $v$  = relative velocity of creep along this surface, and  $\eta$  = the coefficient of viscosity.

Dividing both parts of equation 4 by the normal stress,  $\sigma$ , acting on the same surface, we obtain

$$dv = \lambda(f - f_0)dy$$

where  $\lambda = \sigma/\eta$  is the coefficient of flow. The relative velocity of a point along the axis  $x$  is

$$dv_x = \lambda(f - f_0) \sin \alpha dy$$

where  $\alpha$  is the angle between the normal to the elementary surface and the axis  $x$ . Integrating between limits  $O$  and  $y$ , and taking equation 3 into account, we obtain the absolute velocity of the depth creep of slope

$$v_x = \frac{\lambda}{2} \int_0^y \left[ \left\{ \frac{(\sigma_y - \sigma_x)^2 + 4\tau_{xy}^2}{\sigma_y \sigma_x + \tau_{xy}^2} \right\}^{1/2} - f_0 \right] \sin \alpha dy$$

where  $\sigma_x$ ,  $\sigma_y$ , and  $\tau_{xy}$  are the components of stress at the point considered. At  $y = 0$ ,  $v_x = 0$ . Maximum absolute velocity of depth creep is at  $y = KM$ .

The development of deformations in the zone of the depth creep changes the stress conditions in the earth mass. This leads to changes in location of isogonic lines, and consequently to changes in the magnitude and distribution of creep velocity. Externally this is revealed in the phenomenon of the increased deformation of the landslide-nidus.

## References

- BIKOVSKY, V. N. (1954). Determination of long-term strength of wood. *J. Techn. Physics, U.S.S.R.*, 24, No. 9
- CASAGRANDE, A. and WILSON, S. (1951). Effect of rate of loading on the strength of clays and shales at constant water content. *Géotechnique*, 2, No. 3
- HVORSLEV, M. (1937). *Ueber die Festigkeitseigenschaften gestörter bindiger Boden*. København
- LAMB, T. (1953). The structure of inorganic soils. *Proc. A.S.C.E.*, Sep. 315
- POOZIREVSKY, N. (1934). *Foundations*. U.S.S.R.
- TER-STEPANIAN, G. I. (1955). *The Depth Creep of Slopes and the methods of its investigation*. Dr. Thes. Erevan (in Russian)
- TERZAGHI, K. (1950). Mechanism of Landslides. Berkley vol., *Geol. Soc. Am.*, p. 83
- (1953). Discussion on Earth Pressure, etc. *Proc. 3rd International Conference on Soil Mechanics and Foundation Engineering*, vol. 3, p. 205