SWEDISH GEOTECHNICAL INSTITUTE
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Justus Osterman

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Printed in Sweden by
Ivar Hæggströms Tryckeri AB
Stockholm 1962

PREFACE

When judging the shearing resistance of soils, semi-empirical rules have been used to a great extent in Sweden, for instance the Swedish slip circle calculation together with the fall-cone test results. In spite of the fact that low safety factors are for economical reasons frequently used, even for soft and quick clay soils, the methods have proved their value in Swedish practice.

Theoretical methods have been elaborated abroad, adapted to the types of soils and the loading conditions prevailing. Attempts to put these methods into practice in new regions have brought about discussions on the proper field use of the laboratory test results as interpreted in the light of the theories.

In the present report the author, utilizing the energy concepts of strength, has made an attempt at propounding a more general solution to the theory of the shearing resistance of soils with the intention of bridging the gap between the different approaches.

Stockholm, March, 1962

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CONTENTS

Abstract

The strength of soils is treated from different aspects. The imperfections of the theories and the limitations of experiences are discussed, mainly in connection with plastic clays. Attention is drawn to some fundamental properties which influence the resistance capacity to shearing.

Moreover, a failure theory is introduced in which energy considerations are utilized. The differences between the stability failure on one hand and the yield failure on the other are discussed. Finally, some applications on specific cases of stability are demonstrated.

1. Introduction

In the international literature on shearing resistance of soil the discussion usually is limited to various refinements of the laboratory testing procedures to elicit coefficients applicable to certain strength formulas, generally derived with the *Coulomb* equation as a basis.

The normal stress to be used in the equations should be corrected for the uplift of water, including the pore water overpressures, as pointed out by *Terzaghi.* The difference between the total normal stress σ and the pore water pressure u

$$\sigma' = \sigma - u \qquad (1)$$

is familiarly called the *effective stress* and is recognized as a fundamental quantity for calculating friction.

When a natural, undisturbed soil specimen is placed in a box and loaded, it is compressed to a small amount until the load has reached the so-called *preconsolidation load,* supposed to have lain upon the natural soil. Additional load is, in the case of cohesive materials, then firstly taken up by pore water pressures, as shown by *Terzaghi,* which decrease during the outflow of water. When the excess pore water pressures have vanished, the soil is, as is well known, called *normally consolidated.* The consolidation load is thus carried by the solid phase and can accordingly be defined by the *major principal stress,* which is recognized as a fundamental quantity, primarily for calculating cohesion.

SKEMPTON (1954) has made a refinement by writing:

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)] \qquad (2)$$

where A = coefficient
B = »
σ_1 = major principal stress
σ_3 = minor » »

In normally consolidated clay, A and B can often be plotted equal to one, but may in other cases differ very much from unity, especially in non-saturated soils.

The major interest concerns the modified *Mohr-Coulomb* formula of shear strength:

$$\tau_f = c' + (\sigma_n - u) \tan \varphi' \quad (3)$$

where τ_f = shear stress at failure
c' = apparent cohesion
φ' = angle of apparent friction
σ_n = total stress, normal to failure surface
u = pore water pressure in failure surface

In normally consolidated clays c' can usually be omitted. In its original form, this formula does not include the hysteresis effect but may be adjusted to the actual problem by a suitable choice of the c'-value and the φ'-value.

The relation between the change of void ratio and the major principal stress, in log-scale, is often almost linear for the consolidation test, and the actual cohesion appearing in the shear test is supposed to be determined by the *void ratio at failure*, which thus is recognized as a fundamental quantity for the cohesion.

Nowadays, the Hvorslev (1937) formula of shear strength is often used in theory. It is written:

$$\tau_f = c_e + (\sigma_n - u) \tan \varphi_e \quad (4)$$

where c_e = effective cohesion
φ_e = effective angle of internal friction

In this formula

$$c_e = \varkappa p_e \quad (4a)$$

in which $\varkappa$ = coefficient for effective cohesion
p_e = equivalent consolidation pressure (belonging to void ratio at failure)

The pressure p_e is referred to the vertical pressure p_1 on the virgin curve in the oedometer test in *Terzaghi's* simplified equation:

$$p_e = p_1 \cdot \exp\left[B' (e_1 - e) \right] \quad (4b)$$

where e = void ratio
e_1 = » » at pressure p_1
B' = coefficient
$\exp x = \mathrm{e}^x$

The basic parameters of the above formulas are shown in Fig. 1.

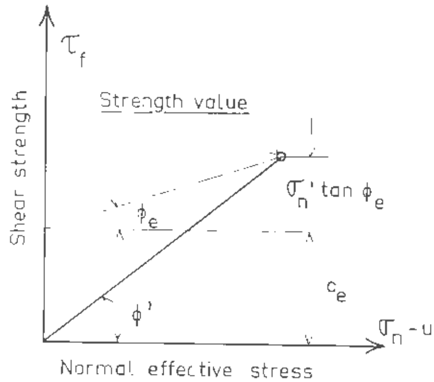


Fig. 1. Figure showing a certain strength value plotted against the effective stress in the modified Coulomb representation, and its separation, according to Hvorslev, in the effective cohesive and effective frictional components (normally consolidated clay).

Another formula in use for clays is the *Krey-Tiedemann* formula (see MUHS, 1957), which can be written:

$$\tau_f = c_n + (\sigma_n - u) \tan \varphi_r \qquad (5)$$

where φ_r = angle of working friction and

$$c_n = \sigma_{nc} \tan \varphi_c \qquad (5\text{ a})$$

in which σ_{nc} = max. working pressure (normal to the failure surface) = initial consolidation pressure

φ_c = angle of working cohesion

This formula is often regarded as being a not very satisfactory equation, as it does not follow the major principal stress and the void ratio requirements, mentioned above.

At consolidated undrained shear of a cohesive soil, a case showing up in the rapid shear-box test, the following equation, also regarded as not quite satisfactory, is used:

$$\tau_f = c_0 + (\sigma_n - u_0) \tan \varphi_{cu} \qquad (6)$$

where c_0 = cohesion intercept

u_0 = pore water pressure (before failure)

φ_{cu} = angle of apparent friction

For a normally consolidated clay, c_0 can usually be omitted.

At undrained shear it has been shown that clay behaves as a pure cohesive material (WESTERBERG, 1921) and the *Cohesion method* (in international usage called the $\varphi = 0$ method) could be used:

$$\tau_f = c = \tfrac{1}{2} (\sigma'_1 - \sigma'_3) \qquad (7)$$

where

c = shear strength, evaluated from test values

In Swedish tradition, for instance, the strength of a clay is often determined on samples tested according to the fall-cone method (J. OLSSON, 1919). This is calibrated to full-scale tests on piles and to landslides occurred. Later on *Skaven-Haug, T. Hultin* and *Caldenius* calibrated the method to the shear-box and the punch tests. The cone test, together with the Swedish slip circle calculation, thus constitutes a method based on experience which is too well established to be disregarded for actual clays.

Similar results have been found by means of other apparatus, for instance the field vane-borer (Cadling & Odenstad, 1950). Through investigations at the Swedish Geotechnical Institute it has been shown that the field vane-borer and and the fall-cone give similar results, if properly evaluated. This question will be further discussed in a forthcoming paper from the Institute.

Skempton (1953) has given a diagram showing values of the quotient between c and the effective overburden pressure p as a function of the plasticity index I_p.

Supplementary values have been provided for instance by Bjerrum (1954) and Osterman (1960 a, b). The plasticity index I_p for normal Swedish clays or organic soils with clay content, $40 < w_L < 150$, may be expressed as

$$I_p \approx 0.82\,(w_L - 20) \qquad \text{(7 a)}$$

where $w_L =$ liquid limit

Hansbo (1957) has given the following relation between c/p and w_L in the case of vane tests:

$$c = 0.45\, w_L p \qquad \text{(7 b)}$$

which fits most Swedish clays rather well. The present author believes, however, that these c-values should be reduced for very soft and organic clays. Moreover, it is necessary to realize that p refers to the effective pressure before failure. Results from failure tests in the triaxial apparatus must therefore be corrected to fit in this figure.

From the above it can be seen that the apparent cohesive strength is put equal to the half of the difference between the major and minor principal stresses (total or effective) and that it depends primarily on the major principal effective stress. The expression "total stress analysis" (often met in the literature) on the cohesion method is thus not fully adequate.

2. Shortcomings in the Shear Strength Formulas

For a long time, some shortcomings have been acknowledged in the strength formulas, and corrections necessary for one or another influence have been pointed out.

One of the most important influences is that of volume changes (*dilatancy*) during shear. Casagrande (1936) and others have discussed the influence of dilatation in cohesionless soil and the so-called *critical density* in which state it can undergo deformation or actual flow without volume change. Additional terms necessary because of dilatation are discussed by BISHOP (1950), HVORSLEV (1953), OSTERMAN (1957, 1959), POOROOSHASB & ROSCOE (1961), and many others.

Moreover, the intermediate principal stress may have some influence, a question touched upon by several authors, and by the author (OSTERMAN, 1957) in connection with the question of volume change and boundary effects.

Another important influence is the loading rate and strain. This question has been discussed by CASAGRANDE & WILSON (1953), BJERRUM et al. (1958), CRAWFORD (1959, 1961), SCHMID et al. (1960), and others.

A serious question is to what extent one should take into account the excess pore water pressure arising at failure, mentioned by OSTERMAN (1960 a) and ZEEVAERT (1960).

The discussion has mainly dealt with the additional terms in the *Mohr-Coulomb* and the *Hvorslev* formulas, these being the two most generally used equations when interpreting results from the laboratory. In the case of the *Cohesion method,* corrections are recommended for clay fineness, strength reductions thus being made for the high plasticity clays and organic soils.

The author raised some objections to the *Mohr-Coulomb* equation (OSTERMAN, 1960 a and b) and expressed that results from short-term triaxial tests gave moderate scatter when the *Cohesion method* or the *Krey-Tidemann* equation was used.

It is interesting to note that still more severe objections have been raised and also demands for revision of the strength formulas. As an example, SCHMID et al. (1960) have tried to use formulas from the rheology.

As can be seen from the above, some curious contradictions between theory and practice exist. As the solution of the problem of the stability of natural slopes became urgent, in connection with an extensive exploration project, the present author had to examine the theories. It was obvious that the fundamental theories must reasonably contain shortcomings, which are dangerous mainly for regions with soft and sensitive clays.

The author also finds refinements necessary in the *effective-stress philosophy.* Recently, also MITCHELL (1961) has discussed a kind of separation of the effective stresses.

In the last-mentioned paper of the present author is was said that the conventional concept of strength is in reality rather primitive, and that an accurate estimation of the shearing resistance of a slope calls for an extensive study of the stability of the system from the *thermodynamical* point of view. Normally, however, one must and can confine oneself to obtaining knowledge of the main amount of energy mobilized at a displacement, arising during an estimated time, and to finding out if that event will occur during drainage.

In the following some problems taken from the new theoretical aspects will be discussed.

3. Separation of the Effective Stresses

In a saturated soil the load is carried by stresses in the soil particles and in the water. If the measured *pore water pressures* are subtracted from the *total stresses,* one arrives, as mentioned above, at the *effective stresses.* For non-saturated soil the air pressures must be taken into account (LAMBE, 1960).

From the consolidation theory, it is known that the effective stresses should control at least the primary consolidation process. However, the strength of a clay increases also during the secondary settlements, at which the effective stresses are almost constant.

During the latter settlements a creep is thus in process. Furthermore Fig. 2 shows that, theoretically, bearing in mind the secondary effects resulting from earlier loadings, the rate of settlement following the application of a small excess load should be of secondary order.

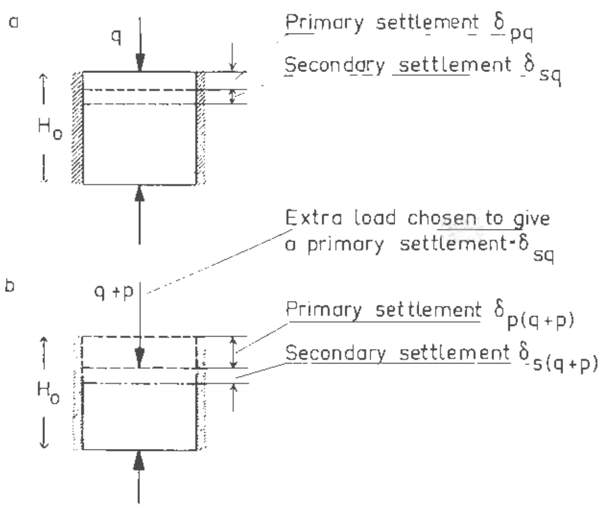


Fig. 2. Soil element, (a) subjected to primary and secondary consolidation, and (b) loaded by an additional load which, theoretically, should result in small changes of pore water pressures.

Obviously some repetitive loadings, vibration, *etc.*, influence the orientation of particles. Therefore, the extent to which the secondary effects refer to overconsolidation (by vibration effects, *etc.*) or to aging may be questioned. In some respect, the difference between static and dynamic friction shows a resemblance to the mentioned effects, and also to thermal effects.

This question is to some extent connected with the structural models of clay soil. The model suggested by EMERSON (1959) implies clay consists of a loose skeleton of coarse mineral grains (composed of foreign crystalline non-argilaceous material, such as quartz, feldspar, *etc.*), embedded in a colloidal clay matrix. (The characteristics of the particles as well as organic matter—usually in colloidal dispersion—electrolytes, *etc.* are important for the water sorption.)

The pore water pressures are, however, measured by means of piezometers, which do not record the specific pressures on the microscopic scale (OSTERMAN, 1960 b).

The author therefore suggests, in order to make it easier to discuss the behaviour of the cohesive materials, a separation of the effective stresses into two or three groups, *viz. intergranular* contact stresses (of the chrystalline solid phase) and the stress distribution associated with plastic deformation in the polyphase clay matrix. Since these latter stresses depend to an essential degree on the intermolecular forces in the liquid phase, they shall, for short, be designated as *molecular* stresses. Considerable changes in the molecular stress distribution may, of course, be expected, due to chemical effects or steric blocking of glide processes at excessive plastic deformation, giving rise, *e.g.*, to system labilities or strain hardening (stresses in *cementations*).

That means an expression

$$\sigma' = \sigma_i + \sigma_m + \sigma_c \quad \text{(8)}$$

where

σ_i = intergrannlar contact stresses
σ_m = molecular stresses
σ_c = stresses in cementations

Below, the question of the release of certain parts of the effective stresses during the shearing process will also be discussed.

4. Energy Consideration in Connection with Shearing Resistance

The strength of a material means a property enabling the material to withstand rupture or excessive deformation. At higher stress or increased strain, the material fails. The special question of stability means, usually, an investigation of the risk of excessive deformation at stresses lower than those of rupture.

As an example from a related science, a column is unstable when the work produced by the column load multiplied by the vertical lowering of the column top at a virtual deflection amounts to the strain energy of bending. If the buckling load is lower than the yield load, the column fails by buckling. This question is discnssed in most of the textbooks on strength of materials.

The same problem also seems to be frequently in question in soil mechanics, especially concerning the problem if slope failure will occur in undrained or in drained condition. This question will be dealt with in the following.

The discussion of the failure process must be presented with due regard to the first law of thermodynamics, *viz.* that the sum of all the energies in an isolated system is constant. No exceptions to this law have yet been found.

41. Granular Materials

In the case of typical granular materials, such as sand and gravel, the cohesive strength can usually be neglected. In such a soil, forces between grains exist as edge-to-surface contact pressures.

At the Swedish Geotechnical Institute, shear tests on edge-to-surface conditions show angles of friction about 8—10°, in a dry state. The apparent angles will increase somewhat in water and even more at intermediate humidities, depending on capillary phenomena, *etc.*

In natural soils, the travel of the contact points of the grains is curved in space because of interlocking of grains. A discussion of this effect has been made by Rowe (1954) and Osterman (1959). The effect is obvious even on rough surfaces.

During shear the frictional component of the contact forces will perform work at the travelling of the grains, and the shearing of the mass will be energy consuming. The resistance against shear must thus amount to the energy necessary for a virtual shearing, Fig. 3. Further, this means that the shearing forces necessary for a certain travel must be higher than in the case of edge-to-surface friction, *i.e.* a higher friction will show up.

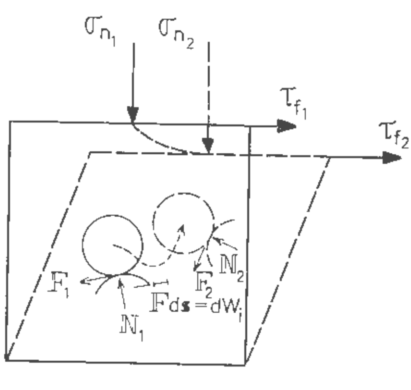


Fig. 3. Soil element showing positions of working stresses and reactions in different stages of shearing travel at failure.

By way of simplification, we regard the paths of a soil element as being linear and introduce an *apparent angle of friction* φ. Under the condition of constant volume of mass this angle is, for Swedish natural sands, about 32°, some variations being possible depending on shape and roundness of the grains, as well as on the pressures. At high pressures, the apparent angle of friction often decreases.

If we plot the shearing resistance τ_f as a function of the consolidation normal pressure σ_{nc}, we arrive at a slightly curved diagram, which usually may approximate a linear relationship:

$$\tau_f = \sigma_{nc} \tan \varphi \qquad (9)$$

A back pressure u may be present, the total pressure being $\sigma_n = \sigma_{nc} + u$ and

$$\tau_f = (\sigma_n - u) \tan \varphi \qquad (9\,a)$$

In the case of volume change, one can proceed as follows. If the forces are applied slowly, kinetic energy can be neglected, and the energy equation can be written

$$dW = dW_g + dW_f + dW_d + \dots \qquad (10)$$

where dW = external work
dW_g = work absorbed in grain deformation
dW_f = » » » friction between grains
dW_d = » » » dilatation

The work absorbed in edge rupture, for instance, is taken to be included in the three terms.

In the case of tightly packed sand, the dilatation will on the whole be positive. When the applied stresses become high enough, sufficient energy can be mobilized to overcome the resistance due to both friction and dilatation, as occurs at the *peak point* of the stress-deformation curve. The stress then falls off gradually to the *ultimate* value. In a loosely packed sand no peak point exists.

Let us consider an element as shown in Fig. 4. At the average angle of shear $d\gamma$ the unit dilatation will change $d\delta_n$ in a direction normal to the slip surface, $d\delta_t$ parallel to the surface in the plane of the section and $d\delta_3$ normal to the section. The volume change is here supposed to be positive when swelling occurs.

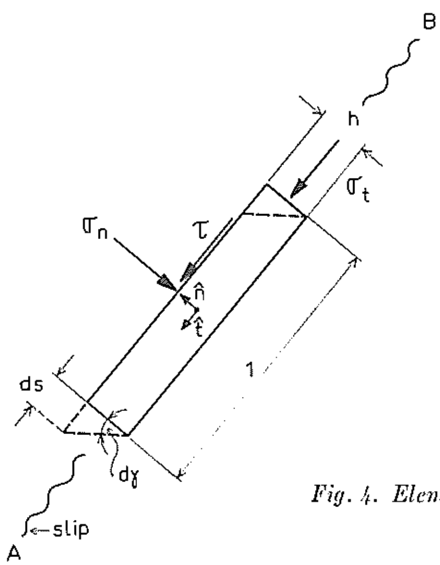


Fig. 4. Element of slip layer, AB, before and after shear.

At the peak point

$dW = dW_f + dW_d$

from which one can derive (OSTERMAN, 1957):

$$\tau_f = \sigma_n \tan \varphi + \sigma_n \cdot \frac{d\delta_n}{d\gamma} + \sigma_t \cdot \frac{d\delta_t}{d\gamma} + \sigma_3 \cdot \frac{d\delta_3}{d\gamma} \qquad (11)$$

The dilatation can be prevented from occurring in directions other than normal to the slip surface. In the case of a shear-box test, and putting $ds = h\ d\gamma$, we get (*cf.* BISHOP, 1950)

$$\tan \psi = \tan \varphi + h\ \frac{d\delta_n}{ds} \qquad (11\text{ a})$$

where ψ = apparent angle of *internal* friction.

In a *triaxial* test apparatus it is possible to compute a dilatation effect as follows.

We shall assume that the deformation in a unit element will be $d\varepsilon_1$ in the direction of the major principal stress σ_1, and $d\varepsilon_3$ in the directions of the both minor principal stresses σ_3. For the sake of simplicity, calculate with *real deformations*, thus $d\varepsilon_1$ being a compression and $d\varepsilon_3$ being an elongation. The total external work, including boundary energy correction, will be

$$dW = \sigma_1\ d\varepsilon_1 - 2\ \sigma_3\ d\varepsilon_3 \qquad (12)$$

Putting volume change $dv = 2d\varepsilon_3 - d\varepsilon_1$ one finds

$$\sigma_1 - \sigma_3 = \frac{dW}{d\varepsilon_1} + \sigma_3\ \frac{dv}{d\varepsilon_1} \qquad (13)$$

Both dW and $d\varepsilon_1$ contain friction and dilatation parts to an unknown degree, and the equation must be rewritten to show these contributions explicitly.

Put $d\varepsilon_1 = d\varepsilon_{1f} - d\varepsilon_{1d}$ and
$d\varepsilon_3 = d\varepsilon_{3f} + d\varepsilon_{3d}$

where f stands for friction and d for dilatatiou.

Now

$$dW = dW_f \left(1 - \frac{dW_d}{dW_f}\right) = (\sigma_1 - \sigma_3)\ d\varepsilon_{1f} - (\sigma_1 - \sigma_3)\ d\varepsilon_{1d} - \sigma_3 dv \qquad (13\text{ a})$$

and

$$\frac{dW_d}{dW_f} = \frac{(\sigma_1 - \sigma_3)\ d\varepsilon_{1d} + \sigma_3 dv}{(\sigma_1 - \sigma_3)\ d\varepsilon_{1f}} \qquad (14)$$

If pure swelling occurs, $d\varepsilon_{1d} = d\varepsilon_{2d} = d\varepsilon_{3d}$, Eq. (14) can be transformed into

$$\frac{dW_d}{dW_f} = \frac{\sigma_1 + 2\ \sigma_3}{\sigma_1 - \sigma_3} \cdot \frac{2\ d\varepsilon_3 - d\varepsilon_1}{2\ d\varepsilon_3 + 2\ d\varepsilon_1} \qquad (14\text{ a})$$

If the volume change is directed normally to the failure surface, the equation will be

$$\frac{dW_d}{dW_f} = \frac{\sigma_1 + \sigma_3 \cot^2\alpha}{\sigma_1 - \sigma_3} \cdot \frac{2\,d\varepsilon_3 - d\varepsilon_1}{2\,d\varepsilon_3 + d\varepsilon_1 \cot^2\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14\,\text{b})$$

where $\alpha = 45 - \psi/2$. As mentioned above, $d\varepsilon_3$ is presumed to act in two directions. If not, the change in the equation can easily be computed. Further reference is provided by OSTERMAN (1959). Friction at the ends of the samples should be avoided or corrected for.

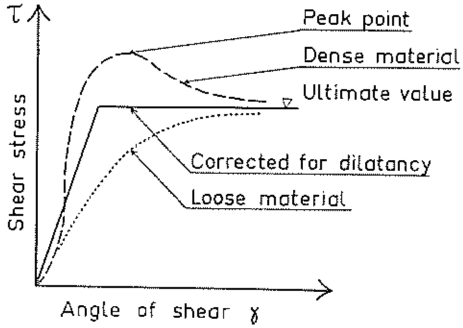


Fig. 5. Relation between stress and angle of shear obtained from shear test on friction soil.

Plotting the shear stress τ against the angle of shear γ, Fig. 5, the relation is usually the same as in the case of a plastic body with a maximum τ_f providing that the dilatancy correction is made. At the Swedish Geotechnical Institute several tests made on pebbles, macadam, *etc.* in a large shear box have demonstrated this circumstance.

In an *undrained* shear test, some important effects will appear. Fig. 6a shows a box filled with a saturated mass in a loose state of density. The load is taken up by intergranular stresses. When shearing the box, the structure breaks down, Fig. 6b, and the load is carried by pore water pressures. The mass structure is unstable at shearing.

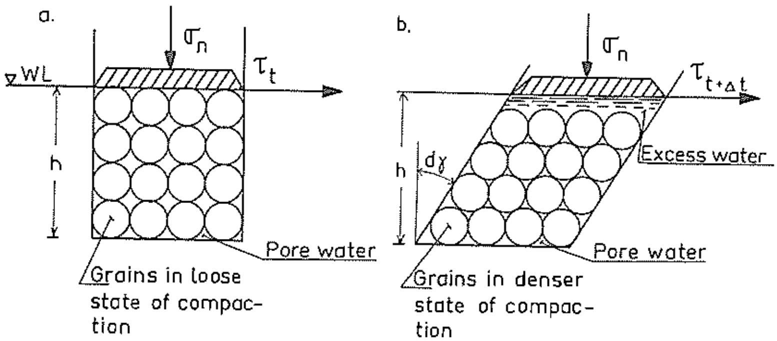


Fig. 6. Effect of undrained shear of swayable box. Container (a) with loosely filled grains and water before shear, (b) with grains in denser state after shear.

Such a break-down has, in a triaxial test on sand, been recorded by NASH & DIXON (1961) as a fast transfer from granular to pore water pressures.

If, on the other hand, the grains in the shear box were densely packed, the application of shear will tend to disturb the initial packing order, increasing the pore volume and giving rise to a reduction of the water pressures.

The undrained shearing resistance can thus be lower or higher than the drained one.

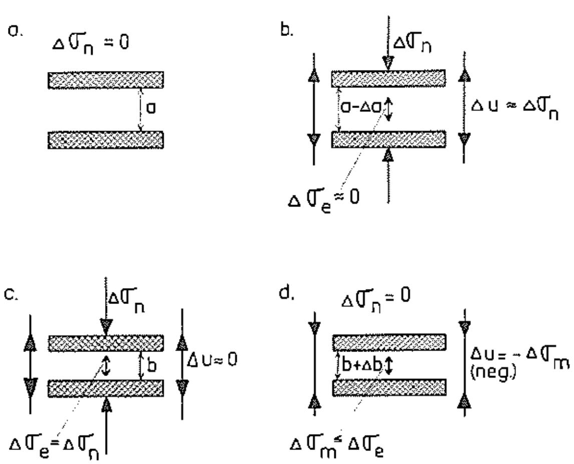
42. Cohesive Materials

420. General

The consolidation of a cohesive material under a given load involves normally an outflow of water due to pressure gradients. The reason for the water pressures is mainly an overexertion of the clay skeleton, similar to that occurring in a bath-sponge compressed by hand. But the reason for excess water pressures may also, in some cases, be negative dilatation of the inner skeleton, similar to that of the shearing of loose sand. This possibility of donble reasons allows for a misjudgement of events.

In nature, both primary and secondary settlements, as well as chemical aging, occur simultaneously in the various layers, giving rise to irregularities in the stress-deformation curve. Irregularities also occur in, for instance, the case of quick clays, and thus confusing the interpretation of results.

When unloading of frictional material, the expansion is very small but sufficient to ensure the almost perfect vanishing of the internal stresses. In the case of clay, however, the expansion is greater, although in Swedish clays it is usually not followed by a release of the molecular stresses, which would necessitate a sorption of pore water or gases. As a consequence and a reaction, negative pore water pressures arise to provide eqnilibrium of molecular stresses, Fig. 7. The residual bonds are included in the concept of cohesion.



$\Delta\sigma$ = load increment
$\Delta\sigma_e$ = increase of effective stresses
$\Delta\sigma_m$ = » » molecular »
Δu = » » pore water pressures
Δa and Δb = small quantities

Fig. 7. Model showing, schematically, a hypothetical origin of residual bond in clay.

In this connection it can be mentioned that BISHOP (1961) reports a test in which a saturated sample from 45 ft below the water table, having a calcnlated *in situ* vertical effective stress of 5.3 lb/sq.in., was found to show a negative pore water pressure of 2.5 lb/sq.in., as measured by a certain apparatus. The size of the pressure was explained as a consequence of the release of shear stress.

The present author supposes that the suction and water sorption also depends on clay type and presence of excess water.

421. Properties of Shearing Resistance

Also in a clay, the resistance against shear, even in the case of stability, must consist of energy mobilized at a virtual shearing.

The coarse grains, being solid, increase the rigidity of the clay matrix. Some frictional and, in the clay matrix, mainly cohesive energy, and other effects of thermodynamical natnre, occur at the shearing. Some long-term reorientation of soil particles and organic matter also takes place in highly stressed zones.

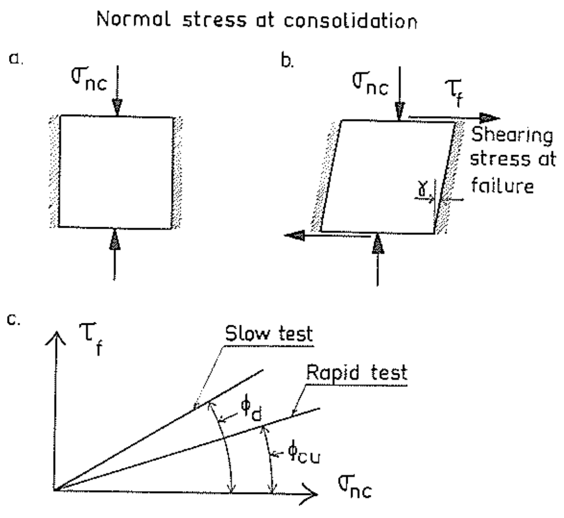


Fig. 8. Element of shear box specimen, (a) before and (b) after shearing; (c) relation between shearing resistance and normal stress at consolidation during slow and rapid tests.

As mentioned above in reference to undrained shear, one usually finds a linear relationship between shear strength τ_f and pressure at consolidation stress σ_{nc} in a rapid shear-box test, Fig. 8, written.

$$\tau_f = \sigma_{nc} \tan \varphi_{cu} \quad \text{(6 a)}$$

If this rapid test is to be interpreted using effective stresses and parameters, we must introduce an extra pore water pressure, Δu_f, arising at failure. By introducing this in Eq. (3) we obtain

$$\tau_f = (\sigma_{nc} - \Delta u_f) \tan \varphi' \quad \text{(3 a)}$$

or

$$\tau_f = (\sigma_n - u_0 - \Delta u_f) \tan \varphi' \quad \text{(3 b)}$$

where u_0 is the measured value in the field.

However, this is a question of unloading, where Eq. (3), which in its original form refers to normally consolidated clays, does not account for hysteresis effects. If Δu_f is introduced in Eq. (4), we deduce:

$$\tau_f = c_e + (\sigma_n - u_0 - \Delta u_f) \tan \varphi_e \quad \text{(4 c)}$$

The value of Δu_f is difficult to measure in the laboratory and impossible to measure in the natural soil. Only a rough value of its magnitude can be estimated for the time being, but it should be possible to compute it by using the principal stress ellipsoid, and the actual strains. Thus the *effective-stress philosophy* cannot, without correcting its routine use as proposed in literature, yet be employed in practice for soft clays.

The reason for the above effect is an inner, negative dilatation of the clay skeleton. If, on the other hand, the dilatation is positive, the routine interpretation gives usually a value which differs from reality in another direction.

In the laboratory, the shear-box test specimen is often sheared slowly under drainage. Also here a linear relationship between shearing resistance τ_f and effective, normal stress σ_n' appears:

$$\tau_f = \sigma_n' \tan \varphi_d \quad \text{(15)}$$

where φ_d = angle of apparent friction.

It is discussed if this angle and the abovementioned angle φ' should be equal. The author believes that this is not the case.

During the shear, consolidation is proceeding in this case, and the effective principal stresses are in inclination to the direction of the shearing force.

Because of the loss of water, some energy is dissipated in the consolidation process. Therefore, the extent to which the change of volume should be considered a pure dilatation effect in the boundary energy correction (Hvorslev, 1953) may be questioned. However, the problem can apparently be circumvented in most cases.

a. Tests in the Triaxial Principal Stress Apparatus

The loading conditions might, at least theoretically, be made more pure in the triaxial principal stress apparatus, in which an isotropic cell pressure and a vertical extra load are applied to the test specimen.

To fulfill these requirements, the specimen in the undrained test should first be consolidated anisotropically, and then sheared rapidly or slowly. At shear, influence of the horizontal friction at the upper and lower ends of the test specimen should be avoided, if possible. An extrapolation to the proper situation of failure, calling for a series of tests at different stress ratios, seems to be necessary. In the drained test of soft soils, the deformations are, however, so great that the properties become changed, for example: the quick clay loses its high sensitivity. Such results must be regarded as being not representative for the original clay.

In the isotropically consolidated specimen, the effective normal stress in the surface of failure decreases when a deviator load is applied, the reason being the high pore water pressures arising as a reaction to the deviator load. An application of the *effective-stress philosophy* in its common usage must here be erroneous, since the cohesion and friction members of the strength equations belong to different consolidation pressures.

To avoid these excess water pressures, one can also decrease both minor principal stresses to such an extent as to keep the overpressures at zero. Also, in this case, the normal stress decreases, giving rise to discussion concerning which interpreting method should be used. Thus, the reason for the constant pore water pressures is most probably that overpressures, appearing from shear in the failure zone, are compensated by suction, arising as a reaction to the decrease in total normal stresses. This may mean that, at least with regard to the cohesion, the failure occurs in a state of overconsolidation, which demands corrections of results.

As mentioned above on c/p-values, one should use that value of p which corresponds to the effective overburden pressure before rapid failure and not at failure.

Crawford (1961) made an interesting study of some similar complications of the triaxial testing, and stated that "non-uniform consolidation occurs due to application of all-around pressure and that water moves away from the plane of shear", and that "under constant axial stress pore pressure increases in direct proportion to strain". It may be such effects which influenced the results of the triaxial tests on soft clays, discussed by Osterman (1960 b).

In view of the present state of the theoretical development in this field, it is a major inconvenience of the triaxial test that it requires a theory for its interpretation.

Obviously, the time has not yet arrived for utilizing triaxial test results for soft saturated clays. The apparatus may, however, be used for unsaturated materials, if the demands on the accuracy are moderate, and for coarse mixtures, if the cell dimensions are appropriate. If horizontal friction at the ends of the test specimens occurs, it should be corrected for.

b. Special Influences on Resistance

As in the case of sand described above, there may be a kind of *critical* state of density also in the case of clay. Roscoe et al. (1958) state the existence of a unique *critical void ratio line* in the coordinate space, defined by σ_n', void ratio e and shear strength τ_f. For heavily overconsolidated clay, Δu_f may become negative. (A clay was considered as heavily overconsolidated when it had been loaded eight times as much as the existing overburden.)

Samples on the *wet* side, contract in drained tests or show a positive pore pressure change in undrained tests; whereas those on the *dry* side dilate or develop a negative pore pressure change. In reality the terminology *wet* and *dry* implies an oversimplification, since the very complex phenomena in the deformation of the clay matrix play an important rôle in the case of clay soils.

In this connection a rather intricate question is to what extent the shearing resistance, due to overconsolidation, will be reduced by swelling from water flow and gases.

In the Swedish clays swelling occurs if water flows through the overconsolidated clay due to a pressure gradient caused by an artesian water pressure, an excess pore water pressure from a load giving a flow through a dry crust, *etc.* In a certain river, for instance, softening has been observed 2 à 3 m below the bed surface, which is quite reasonable. Fissures are observed in Swedish clays, especially in the upper layers of the clay.

The softening of zones in stiff fissured clay has been discussed several times in a series of articles on London clay.

When evaluating the possibilities from the specifics of the natural case, it is therefore necessary to know which test method is utilized and if the clay in question has been influenced by excess loadings or artesian pressures.

c. *Sensitivity*

Sensitivity usually means the quotient between the strength of natural undisturbed clay and that of remoulded clay, in the case of undrained tests. As the remoulded strengths are often low, the results are highly dependent on the test methods used. The author prefers to employ a test with the fall-cone apparatus.

When discussing the strength of disturbed clays, it is necessary to be judicious in the use of the above formulas, as is made clear from the following.

Fig. 9 shows the relation between consolidation pressnre and compression for undisturbed and remoulded samples, respectively. At the same compression, the quotient between the higher and the lower stress can be regarded as a certain factor, correlated to the sensitivity, but giving another sense than that mentioned above. The vertical difference indicates a structural influence.

There are two major reasons for a high sensitivity, namely, a reversible effect, *thixotropy*, and an unstableness of the soil skeleton, *quickness,* as described below.

c 1. *Thixotropy*

When a clay is remoulded, the inner structure and the equilibrium of the molecular stresses are disturbed. This means that the bonds between particles are weakened and the resistance to shear is decreased. Apparently some particles having edge-to-surface orientation have turned to positions in parallel. Present knowledge has been discussed by Lambe (1958) and by Seed et. al. (1960), and others.

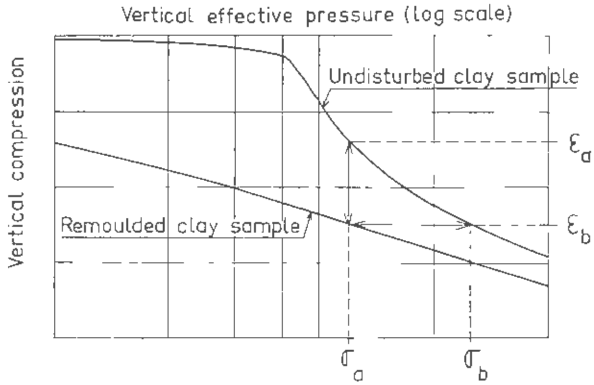


Fig. 9. Relation between results of undisturbed and remoulded clay consolidated in oedometer. The difference is shown in resistance to external load at one water content, and in internal soil structure at the same vertical effective pressure.

The energy put in during remoulding brings about an unstable structure, in which the particles and the water will move to more stable positions.

The build-up of the new structure thus necessitates a rearrangement of particles and water, which takes time. During the rearrangements, the strengthened bonds must be brought into equilibrium and this demands pore water suction. Thus, pore water suction is built up during the hardening.

The increase with time of the tension of the pore water has been observed by Bishop (1961).

It follows from the above that the restoration of strength, thixotropy, must reasonably be higher the higher the cohesion, or rather the fineness, and, to a certain extent, the water content. The thixotropy is also dependent on the particle structure.

c 2. Quickness

A *quick clay* may be defined as a clay which, when undisturbed, possesses a certain strength and can be considered a solid, but which when disturbed can apparently be considered an imperfect liquid. If the sensitivity exceeds a certain number, say 30—50, depending on the method of determination, a clay is in practice termed *quick*.

The transition point of the clay from the plastic to the liquid state can, as a rule, be identified with the *Atterberg percussion liquid limit* w_L or with the *fineness number* w_F (Swedish State Railways, 1922, and R. Karlsson, 1961). Both methods give nearly the same values, apparently a little on the *dry* side, especially for some organic clays.

The liquid limit (or, as sometimes advocated, the plasticity index) can thus be roughly correlated to the sorption capacity or apparent *fineness* (the total area) of the particles.

The reasons for the possibility of the excess in the water content may be a skeleton of coarser materials and perhaps also the hardening of the clay during the reorientation of the grains at the secular deformations. These extra bonds should, in such cases, correspond to cementations between the particles. At shearing the intergranular and cementation stresses may be released.

The sorption capacity of the particles of a clay can be changed by adding dispersing agents, which seem to be the most important influence (Söderblom, 1960), or by removing or adding other chemicals, or also by both processes. There are several substances possessing dispersing effects on clay, natural humates and carbonates, and also other chemicals.

If these changes are not followed by corresponding changes in the water content, the water can exceed that of the liquid limit. Usually, the quick clays possess some 20 % excess water.

The sodium-chloride and similar electrolytes generally hinder the formation of quick clays (Rosenqvist, 1958). Thus, the quick clays are found in deposits sedimented in fresh water or in salt water and leached out.

As can be seen from the above, the restoration of strength cannot be made reversible by reforming the clay. However, it is possible to change a consolidated clay by osmotic or other processes.

Natural quick clay deposits often occur in Sweden, especially in the southwestern region. They are mostly confined to small areas and have a thickness of some metres. From a study of diagrams of the sensitivity in relation to the soil types, the present author arrived at the conclusions that quick clays are often situated close to more permeable layers and that water flow might often be present.

In nature one can find quick clays having sensitivity values up to 650 or even more. In the laboratory it is possible to break down the bonds as to form sensitivity values up to about 1,500. Since the strength results are dependent on the velocity of testing and methods used, these results are rather arbitrary.

In some respect a sand, built up of a loose skeleton with local contact bonds, a "true" quick sand, may behave in a similar way as certain quick clays.

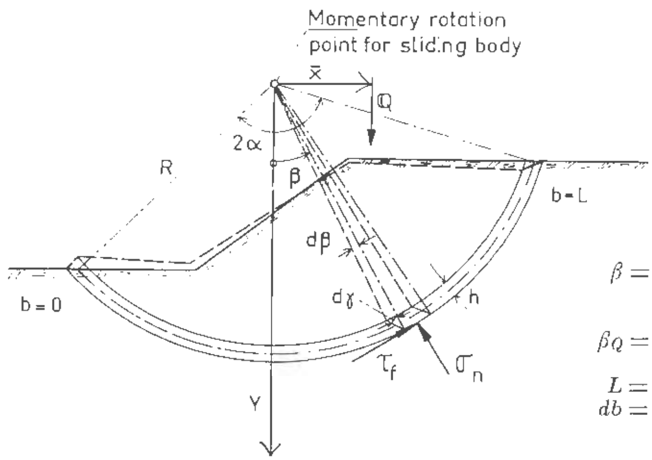
422. Stability and Yield Conditions

a. Quick Failures

In a clay slope or other earth mass, the margin of safety may be jeopardized by some rapid loading. When the critical situation has thus been built up, the state of consolidation corresponds to a pre-existent stress distribution, here usually with lower intensity.

The calculation should be made under the assumption of quick, undrained shear.

Energy is stored in the system by the position of the soil mass with its load, which at failure are assumed to slide on a slip surface or, rather, on a soil layer set in motion. At failure, this mass Q moves a certain distance during a time-



β = angle between radius vector (to a certain point) and the vertical axis
β_Q = value of β to the point of gravity of sliding body
L = length of slip circle
db = length of element of slip circle

Fig. 10. Supposed failure layer in a clay slope. Notations used.

interval, dt, the vertical component of the distance multiplied by the weight forming the external work.

If the slip surface is assumed to be circular, as in the case of Fig. 10, and the distance between the momentary rotation point and the centre of gravity of the sliding body is r, and the angle to the vertical is β_Q, the work done at an angular change $d\beta$ is calculated at:

$$dW_e = Qr \sin \beta_Q \, d\beta \qquad (16)$$

In the slip layer, with thickness h, an element with the unit width and the length db is influenced by the normal stress σ_n and the shear stress τ_f. The inner work will correspond to the expression:

$$dW_i = \int_0^L \tau_f \, h \, d\gamma \, db \qquad (17)$$

By assuming $hd\gamma = Rd\beta$, we derive

$$dW_i = R \int_0^L [\tau_f \, d\beta] \, db \quad {}^{1}$$

and, if τ_f is constant during the shear, also

$$dW_i = Rd\beta \int_0^L \tau_f \, db$$

Therefore:

$$Qr \sin \beta_q = R \int_0^L \tau_f \, db \qquad (18)$$

[1] This presumes an integration with respect to $d\beta$ from 0 to $d\beta_{\text{critical}}$.

which corresponds to the common calculation.
By plotting $r \sin \beta_q = \bar{x}$ and $M_q = Q\bar{x}$ and also $L = 2aR$, and assuming $\tau_f = c =$ constant, we find

$$M_q = 2aR^2c \qquad (18\,a)$$

as usual. In certain cases this may thus involve unpermissible simplifications of Eq. (17).

It can be seen from the above that calculations in this specific case can be transferred into the simple assumptions of equilibrium, provided that τ_f is constant during the shear and that no volume changes occur. A release of certain parts of the effective stresses is, however, possible[1].

If the shearing resistance τ_f is dependent on the shearing deformation γ, as for instance in the case of pore water changes, Fig. 11, the variation of the resistance must be taken into account when solving the integrals.

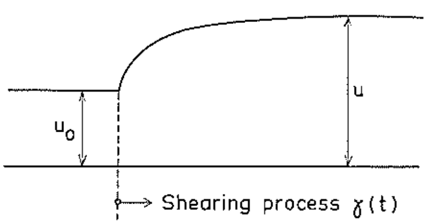


Fig. 11. Variation of pore water pressure during the shearing process, in principle.

In practice, a value is employed which has been derived from a test process in which similar pore pressure effects become apparent, such as the fall-cone or quick shear-box tests or the vane test. When utilizing these methods, especially the shear-box test, it is necessary to take into account the state of consolidation owing to the fact that the pore pressure set up at shearing is dependent hereon. Of course, in the samples used in the cone test, there may exist extra disturbances from the sampling operation or swelling, *etc.*

Moreover, if the shear strength is dependent on loading rate, for instance increases with velocity because of viscous effects, theoretically one should replace $d\gamma$ by ωdt, which means:

$$\tau_f = F\ (\omega,\ t,\ \sigma,\ u) \qquad (19)$$

The question of which failure velocity will arise is also connected with the rate of excess of load and kinetic effects. The influence of dynamic effects, vibration *etc.*, is especially important for the occurrence of secondary slides.

Generally one has to reckon with a possible minimum of shearing resistance at least for the initial slide. At investigations after a slide, care should be exer-

[1] The possibility may be explained from Fig. 6. If the voids, instead of pore water, contained normally consolidated clay, we realize that the intergranular pressures will be interchanged for the pore water pressures in the clay during a rapid shearing.

cised in the analyses of the soil and samples should also be taken adjacent to the slip layer in which the soil might have been subjected to thermal effects.

Finally, upon sliding the clay will lose a part of its resistance by sensitivity effects.

In the laboratory there is a risk that stability break-down can escape observation because of a time lag in the measuring system (or because of hydraulic loading or controlled strain systems).

In the opinion of the author only the quick failure may be regarded as a stability case.

b. Slow Failures

Slow failures in clay slopes are rather infrequent in Sweden and would imply that the value of the resistance against shear is relatively constant or, also, that the resistance, and the exertion, undergo some change.

At low failure velocities one may, in fissured and perhaps also in very coarse clay, assume a pore water flow and dilatancy, which must be taken into consideration.

Where there are dilatation effects in the slip zone, the failure can be discussed on the assumption that the strain-work in the sliding mass can be disregarded, and that the energy equations can be studied by taking the total external work, now from action stresses τ_f and also σ_n. If compared with a similar occurrence in a laboratory test, the problem of the work done by water flow may be circumvented.

At variance with the quick failure, the course of slow failure can more easily be studied in the laboratory.

In the opinion of the author the slow failure may be regarded as a yield failure.

423. Natural Clay Slope Applications

a. Initial Slides

In a natnral slope the margin of safety against instability or yield failures can gradually vanish, see Fig. 12, for instance by successively increased loading due to land heave or changed water levels or to fissures. Decreasing resistance capacity due to softening from water sorption, chemical aging, *etc.* may also occur.

A change of resistance because of orientation of particles and organic substance can show up during *creep, i.e.* a type of *long-term* stability question.

When the critical state has thus slowly been built up, the clay is consolidated aud in some high-stressed zones deformed excessively by creep. In the layer of the potential failure, the clay has been adjusted to some principal stresses σ_{3c} and σ_{1c} (the quotient of which corresponds relatively well to the coefficient of an active earth pressure) and to the intermediate principal stress σ_{2c}. Fig. 13 shows

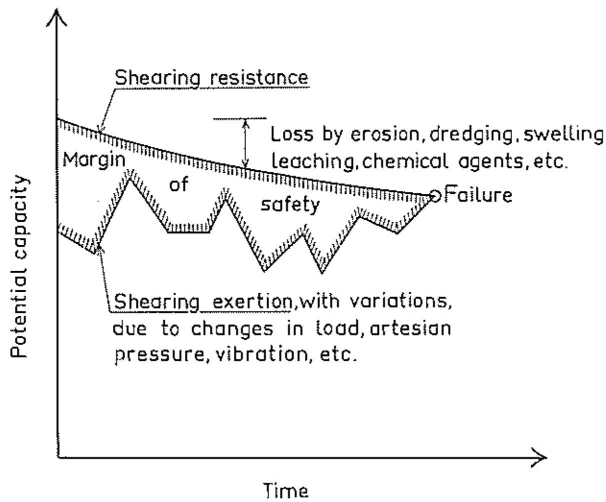


Fig. 12. Variation of shearing resistance and exertion as a function of time, in principle.

a sketch of the failure situation. In the slope shown, it is assumed that the relation between τ_f and $d\gamma$ may vary along the potential slip surface, *e.g.*, by a snpposed removal of mass below the original ground surface. For calculating such a case, it can be necessary to perforu special tests on clay from various points to find out the behaviour of the layer in the actual failure conditions.

The shape of the slide bottom can often be recorded very accurately by means of electrical resistivity measurements. The method implies the obtaining of a jump in the resistivity log, caused by the changed position of the sliding

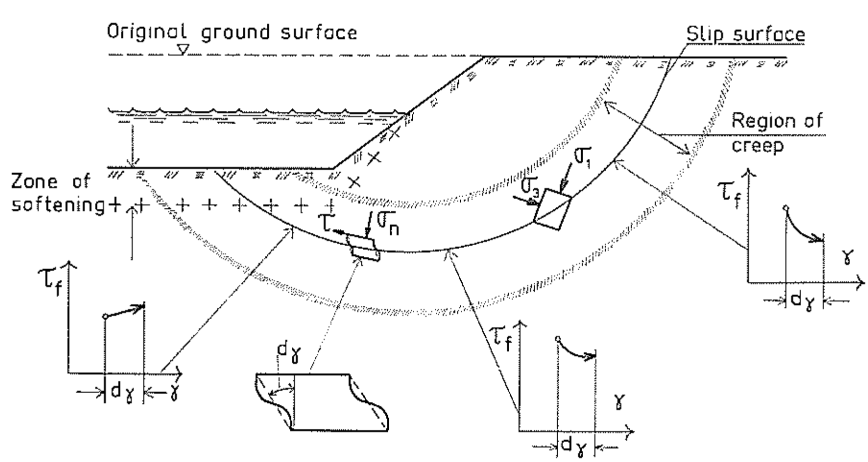


Fig. 13. Possible differences in the relationship between resistance τ_f and average shearing $d\gamma$ during the critical time.

mass. This method was first used at the investigation of the landslide at Göta, near Gothenburg, 1957.

It is often argued that, in a natural slope, one has to consider a *drained* failure. The author feels justifed in asserting the following opinion.

Obviously, drainage may occur from consolidation, swelling or osmotic influences *etc.* during the long-term existence of a natural slope before failure. However, it is the condition of the slope and not its age which is the pertinent factor.

Thus, when dealing with normally consolidated or slightly overconsolidated clays, and with due regard to the negative dilatation of the inner structure, the failure occurs when the stability of the slope is hazarded at a virtual shearing. In the case of heavily overconsolidated clays, and with due regard to the positive dilatation of the inner structure, the failure occurs when the shearing resistance of the slope is exceeded.

The causative factor in the sliding of a mass is thus the lowest active exertion, load or vibration at which the reactive forces are unable to re-establish a stable position. If the loading or reactive ability can be altered by factors, snch as the swelling of the clay, the calculation of stability may require an examination of a couple of situations or procedures.

A natural slope can thus fail under quick shear, and that case would be the most frequent in Sweden, where the clays mostly are normally consolidated or slightly overconsolidated, thus being on the "wet" side of the critical state of density or critical void ratio.

b. Secondary Slides

When an initial slide can occur for some reason, it is of the utmost interest to estimate the risk of secondary slides. The occurrence of secondary slides is rather frequent in Sweden, especially in regions of quick clays.

At the vanishing of the earth pressures between the initial slide and the secondary one, a shearing strain in the slip layer of the secondary slide is released by the draw-down of the mass involved in the initial slide.

A certain inner deformation of the mass of the potential secoudary slide may cause uneven stress distribution. A deviation from the circular form of the initial slide often seems to develop a tension at the rear of the sliding mass, which is to be added to the mentioned strain of the masses behind.

That means not only that the slip layer (in this case often composed of both linear and circular parts) is rapidly loaded but also that vibration effects can appear due to the rebound from the rapid change of loading. Such a vibration is most serious in quick clays or other metastable structures.

The author has made calculations of a couple of secondary slides, and there the excess influences seemingly amounted to the size of the earth pressures released at the previous sliding, or also an uneven stress distribution of the same influence is built up.

Acknowledgements

The author wishes to extend his thanks to Mr. *B. Fellenius,* Dr. *E. Forslind,* Mr. *G. Lindskog* and other colleagues, also from abroad, who have participated in discussions concerning various aspects of these studies.

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