STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 1

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

REPRINTS AND PRELIMINARY REPORTS

Supplement to the "Proceedings" and "Meddelanden" of the Institute

Views on the Stability of Clay Slopes

by Justus Osterman

Reprinted from Geologiska föreningens i Stockholm förhandlingar bd 82 1960 H 3

STOCKHOLM 1960

Preface

Preliminary or final research results have been presented by officers of the Institute in various lectures. The reports have sometimes been publishd in trade magazines. As far as resources have allowed, comprehensive and detailed reports have been published in the Proceedings series of the Institute.

In order to make a quick presentation of certain research reports of more general interest, although preliminary, the Institute will issue "Reprints and Preliminary Reports" as a supplementary series to its "Proceedings" and "Meddelanden". The actual series, which may also contain Swedish papers, will be distributed on an exchange basis.

The first issue of the series contains a preliminary description of research on shearing resistance, performed mainly in connection with the investigations of the Göta River Valley in southwestern Sweden.

Stockholm, December, 1960.

SWEDISH GEOTECHNICAL INSTITUTE

Views on the Stability of Clay Slopes

By

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Abstract. Some aspects are presented on the judgement of the stability of clay slopes in Sweden.

Among other things, the structural changes of soil elements, exposed to shear, and their influence on strength are discussed.

The relation between the soil plasticity and the ratio between strength and overburden pressure is touched upon, and the influence of water pressure and seepage on the latter.

Further, the difference between the condition of equilibrium and that of stability is discussed.

Introduction

Since the author was asked to give a lecture on the stability of clay slopes, he has published a paper (Osterman 1960) on the strength of soft, plastic clays. Only a few points will be recalled here, necessary to follow the reasoning, as well as some explanatory distinctions. The discussion will be confined to Swedish saturated clays in a state close to normal consolidation, and with normal sensitivity.

When a clay consolidates under an overburden, the skeleton is exposed to overexertion causing settlements and creep, delayed by the time-consuming pore water flow.

The rate of flow in a certain clay is dependent on the pore water pressures, u, which can be measured by means of an ordinary piezometer and are thus gravitation parts of the soil water stresses.

During this course of events the clay will decrease in volume, with a consequent increase in cohesion and possibly also in frictional resistance capacity. In natural terrain the horizontal dimensions of the clay elements are mainly unchanged, and a shearing deformation occurs. In principle, the problem refers to a reorientation of the grains.

It may be mentioned in this connection that the shearing resistance of the clay skeleton can be affected by the loading operation, giving extra pore water overpressures. These excess pressures seem to decrease rather rapidly as indicated by some oedometer tests performed by Hansbo (1960), who considers it to be a thixotropic effect. He also points out a possible extra delay in the pore water flow from that calculated on the basis of Darcy's law.

However, we may still discuss matters on the assumptions of a period of mainly primary settlements with considerable loss in pore water overpressures (and some thixotropic effects) and a period of mainly secondary creep with rather small or stationary pore pressures.

We can calculate an angle of apparent friction required from the formula

$$\tan \Phi = \frac{\sigma'_1 - \sigma'_3}{2\sqrt{\sigma'_1 \cdot \sigma'_3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

where σ'_1 = major principal effective stress $= \sigma_1 - u$
σ'_3 = minor » » » $= \sigma_3 - u$

In the case of consolidated equilibrium this angle is called Φ_{ce} by Tschebotarioff (1957) and others.

The strengthening effect of the secondary settlement was discussed in the paper of the present author. From that effect it follows that a small increment of load may be added before considerable primary settlements will begin. That may be the cause of the similar effects which have been found and discussed by Leonards and Ramiah (1960).

At failure the clay can resist greater stresses than at consolidated equilibrium, and the question of creep is therefore important, especially in the case of high plasticity clays.

Drained and Undrained Strength

When discussing the shearing resistance of soils, the expressions "drained" strength and "undrained" strength are often used. The meaning of the terms will be clear from a demonstration of the function of the simple shear model given in Fig. 1.

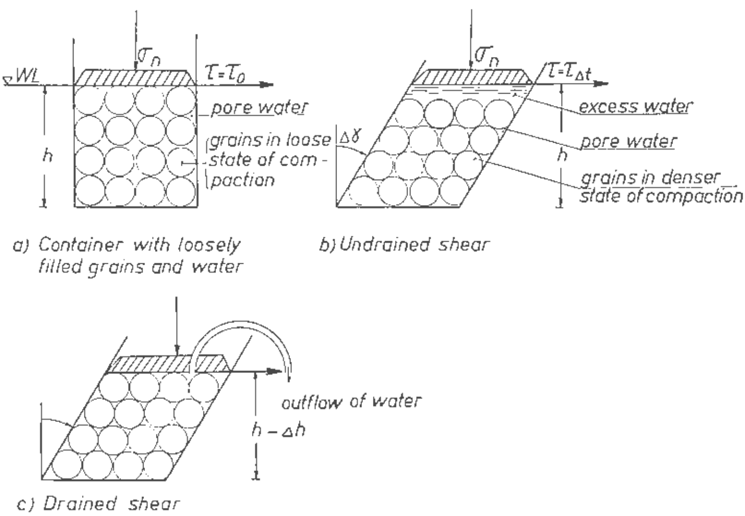

Fig. 1. Model showing the principles of undrained and drained shear in the case of a loose granular mass.

Fig. 1a shows a swayable box filled with saturated soil with a grain skeleton in a loose state of density. The soil is exposed to normal load, taken up by intergranular stresses, and thereafter the box is sheared.

In Fig. 1b is shown how the soil structure will then break down, and how the load after the shearing will be carried by pore water pressures in the case of no drainage. For the granular material the resistance to shear will of course vanish. One may perhaps talk about a kind of negative inner dilatancy.

Fig. 1c shows how the load will still be carried by intergranular stresses in the case of immediate drainage, and it is obvious that there will be a resistance to shear. The volume will decrease and thus there will also be negative outer dilatancy.

If, on the other hand, the grains were originally densely packed, and the soil structure swelled when sheared, a decrease in pore water pressure would, in the case of no drainage, influence the intergranular stresses.

Thus the undrained shear strength may be lower or higher than the drained strength. In the case of the grains in the so-called "critical" state of density, cf. Casagrande (1936), no volume change will occur, and the resistance to shear will be almost unaffected by dilatancy effects.

The above-mentioned model can be used for a simple description of the behaviour of frictional soil exposed to shear. Sand layers can often be found in the clay slopes. When the intergranular stresses are moderate, due to artesian pressures for instance, the frictional resistance to shear failure may be low. If the soil density is loose, a failure will occur quickly.

Recently, Bjerrum a. Simons (1960) have given values of the angle of apparent friction in fine sand and silt down to 10-11°.

Roscoe, Schofield and Wroth (1958) state that the yielding of a clay sample defines a loading path in the coordinate space, defined by the effective stress, σ'_n, normal to the failure surface, the void ratio, e, and the shear stress τ_f in the failure surface, and that the ends of the paths lie at a unique "critical void ratio line". Moreover, it is said that in the tests on overconsolidated samples the positive change (Δu) in pore water pressure is less than in tests on normally consolidated samples. For heavily overconsolidated samples, Δu may become negative. In the paper an overconsolidation ratio line at the critical state is discussed. All samples on the "wet" side will contract in drained tests, or show a positive pore pressure change in undrained tests; whereas all those on the "dry" side will dilate or develop a negative pore pressure change. In this connection the authors refer to earlier papers by Hvorslev and Haefeli.

Attention is drawn to the circumstances just mentioned, because of its importance when judging the behaviour of Swedish clays, most of which are on the wet side, but clays on the dry side might be found in riversides, for instance. That part of the shearing resistance which is due to friction will decrease when the value of u increases.

As appears, the low plasticity materials will give strength results which differ very much in the case of drained tests from that of undrained tests. Swedish clays, however, are rather plastic as can be seen from the plasticity chart,

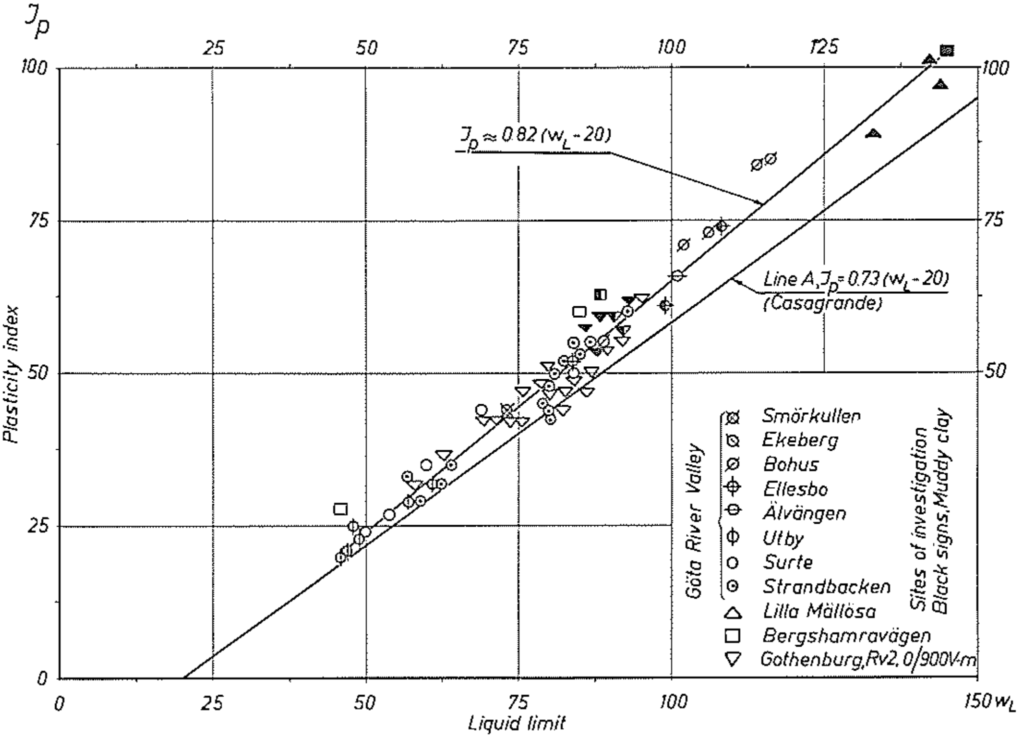

Fig. 2. Plasticity chart for some Swedish clays.

drawn according to Casagrande's principles, shown in Fig. 2, which demonstrates a relation between liquid limit w_L and plasticity index I_p for some clays, tested recently.

Results of Simple Strength Tests

During a drained standard test in a shear box, consolidation occurs and the apparent angle of friction made use of at failure (in the case of normal consolidation), will instead of from Eq. (1), be calculated from the formula

$$\tan \Phi_d = \frac{\tau_f}{\sigma'_n} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where τ_f = shearing resistance in failure surface
σ'_n = effective normal pressure on failure surface

as the principal stresses are not exactly known.

Fig. 3 shows the results of a number of drained shear box tests in relation to the plasticity index. Table I shows some additional data of the test specimens, volume weight γ tons/m^3, water content w in percentage of dry weight in natural state, and plastic limit w_p. Owing to the difficulties of making the

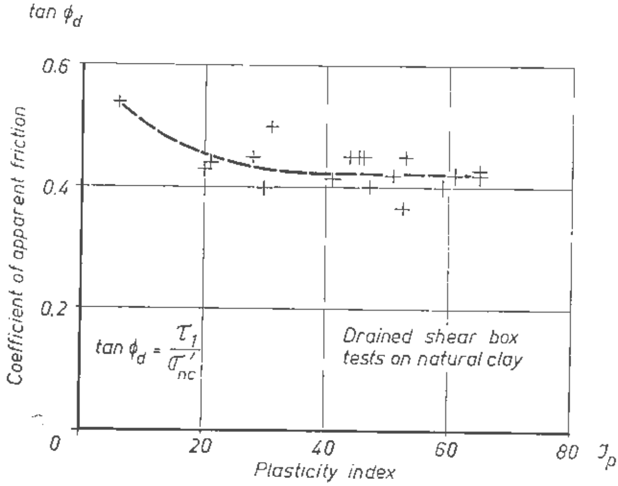

Fig. 3. Apparent angle of shearing resistance plotted against plasticity index, in normally consolidated drained shear-box tests.

tests perfectly drained and to wait for the final consolidation, the Φ_d-values may be a little too low.

During an undrained standard test in the shear box the consolidation is due to the normal pressure and no further loss of water will occur. The apparent angle at failure will be calculated from the formula

$$\tan \Phi_{cu} = \frac{\tau_f}{\sigma_{nc}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where σ_{nc} = total normal pressure on failure surface = initial consolidation pressure

It must be mentioned that this test applies to a special case of loading.

Table I. Data of slow shear-box tests, performed in the SGI-apparatus

Identification No.	Depth m	Volume weight t/m³	w	w_L	w_p	I_p	$\tan \Phi_d$	Soil classification
1403 K 6392		1.75	39	57	27	30	0.40	Black a. grey silty clay, seams of sandy silt, bands of iron sulphide
4745 K 6392		1.74	45	57	36	21	0.44	Dark-grey silty sulphide clay
4474 K 6392		1.74	42	55	35	20	0.43	Dark-grey sandy silty sulphide coarse clay
4037 K 6392		1.62	61	88	37	51	0.42	Dark-grey muddy silty clay
7652 K 6392		2.07	22	26	20	6	0.54	Grey varved silt, seams of fine sands
1873 K 5517, Bh 18	14					65	0.425	
5940 K 5517, Bh 18	28					41	0.415	
1235 K 5314 Ekeberg	20.5	1.65	62	90	31	59	0.40	Grey clay with shells
1147 K 7068 Älvängen	10	1.49	94	99	38	61	0.42	Blue-grey muddy clay, bands of iron sulphide

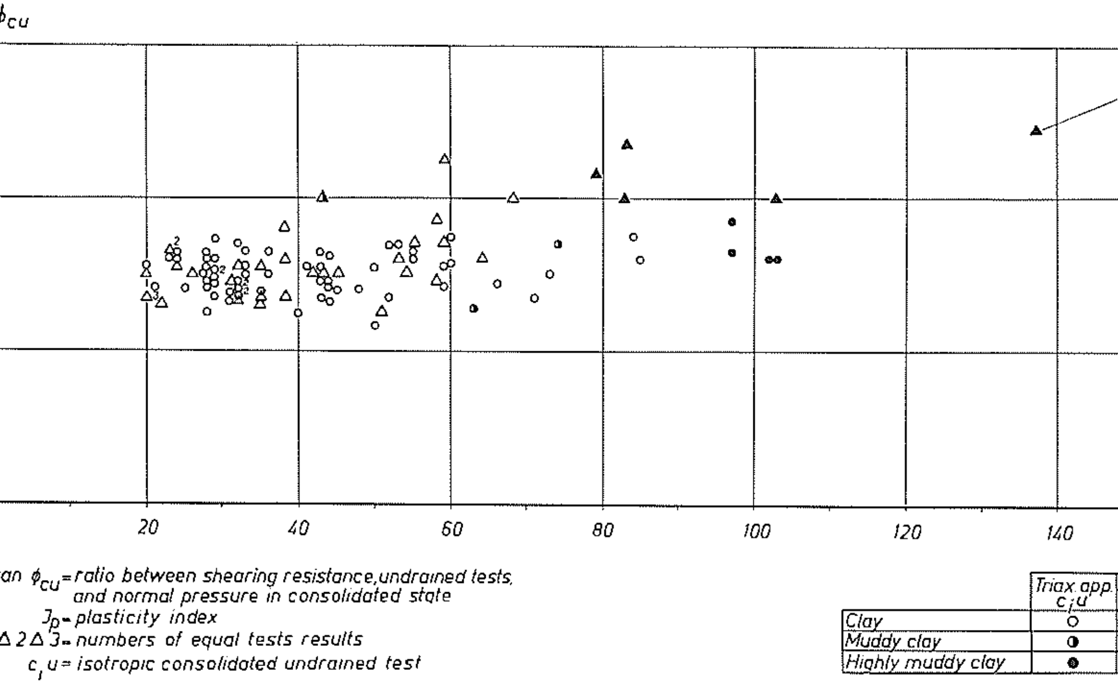

Fig. 4. Apparent angle of shearing resistance plotted against plasticity index, in normally consolidated undrained

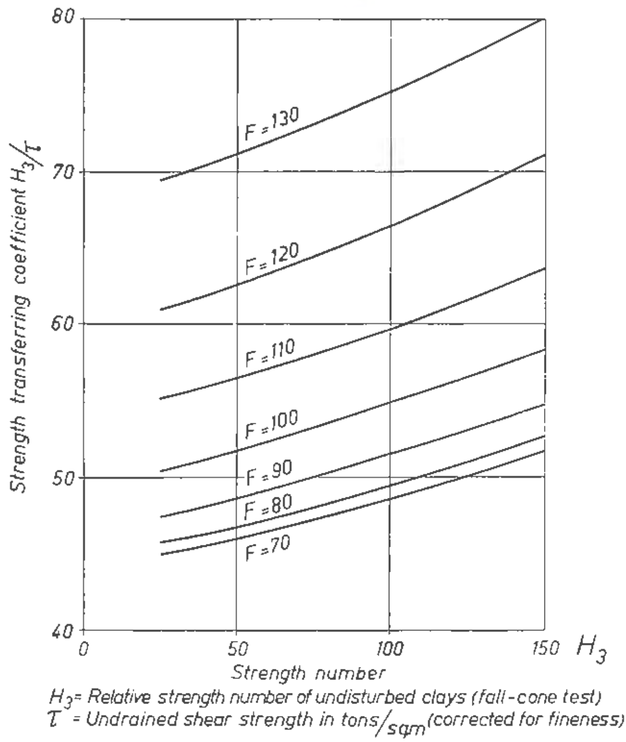

Fig. 5. Diagram showing the value H_3/τ_f as a function of strength number H_3 (see also the text) and fineness number F in the case of the fall-cone test according to J. Olsson.

Fig. 4 shows the results of a number of undrained shear box tests in relation to the plasticity index. In the figure a number of triaxial results are also plotted. For reasons discussed below these results must be regarded with some care.

Owing to the difficulties of making the tests perfectly undrained and quickly, the Φ_{cu}-values may often be too high at small values of I_p.

In the laboratory the undrained strength can also be measured by use of the fall-cone test, a very careful study of which was made by the inventors, the Geotechnical Commission of the Swedish State Railways, 1914—1922 (see Final Report 1922). Terzaghi (1927) introduced a theoretical attempt for the interpretation of the fall-cone test.

On the suggestion of the present author ultra rapid pictures were taken of the motion of the cones and the results were worked up. A comparison was made between the fall-cone test and the vane test results by Hansbo (1957), who used a more precise attempt. The investigation showed that both tests were of similar character, and that an interpretation from the one test to the other did not necessitate to account for the organic content.

In the case of the fall-cone test, J. Olsson has calculated a strength correction for the clay fineness necessary. Fig. 5 shows a diagram of the reduction

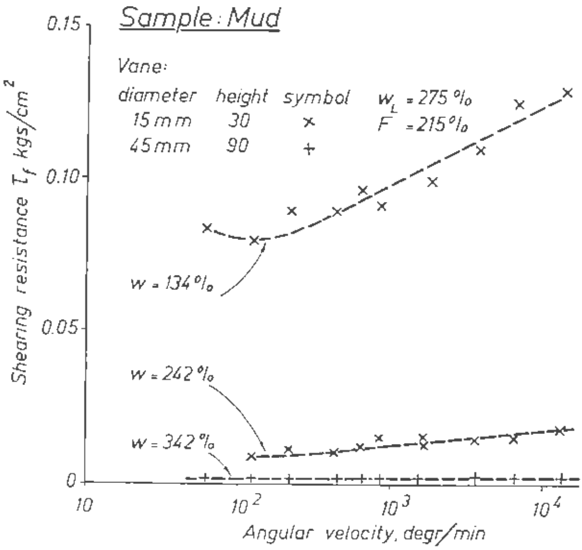

Fig. 6. Diagram showing the shearing resistance of mud as a function of the angular velocity of the laboratory vane.

mentioned as a function of the fineness number which he has sketched. In this connection the fineness number F can be identified with the Atterberg percussion liquid limit. (The H-number of any clay is the weight in gm. of the 60° cone, which is given a penetration of 10 mm, when dropped from the surface of the sample, divided by 6 gm. thus giving, in the case of 60 gm.-60° cone, an H-number equal to 10. Index 3 refers to undisturbed samples.)

The shearing resistance of a remoulded clay can also be determined by the fall-cone test or by the laboratory vane test. In the latter case it is possible to observe the influence of the shearing velocity.

Fig. 6 shows the shearing resistance of a remoulded mud at varying angular velocity (in log-scale) of the laboratory vane. The disturbing effects in disperged systems, when the viscosity is decreasing with increasing velocity, are normally due to coupling on the molecular scale between the dispersed phase and the solvent in addition to orientation[1] effects of anisotropic particles and straightening of coiled threads, etc. As can be seen from the figure the apparent viscosity decreases when the shearing velocity increases, and some characteristics, similar to those just mentioned, seem to influence the behaviour of the mud.

Owing to similar phenomena it may be necessary to reduce strength results obtained by means of quick testing, especially at high percentages of organic matter or high periphery velocity, also in the case of vane tests. Thus, the size of the vane can have an influence.

Fig. 7 shows the ratio between uncorrected vane strength c and overburden

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[1] A diagrammatic representation of the build up of coarse grains in a surrounding colloidal matrix at failure is sketched by Trollope and Chan (1960).

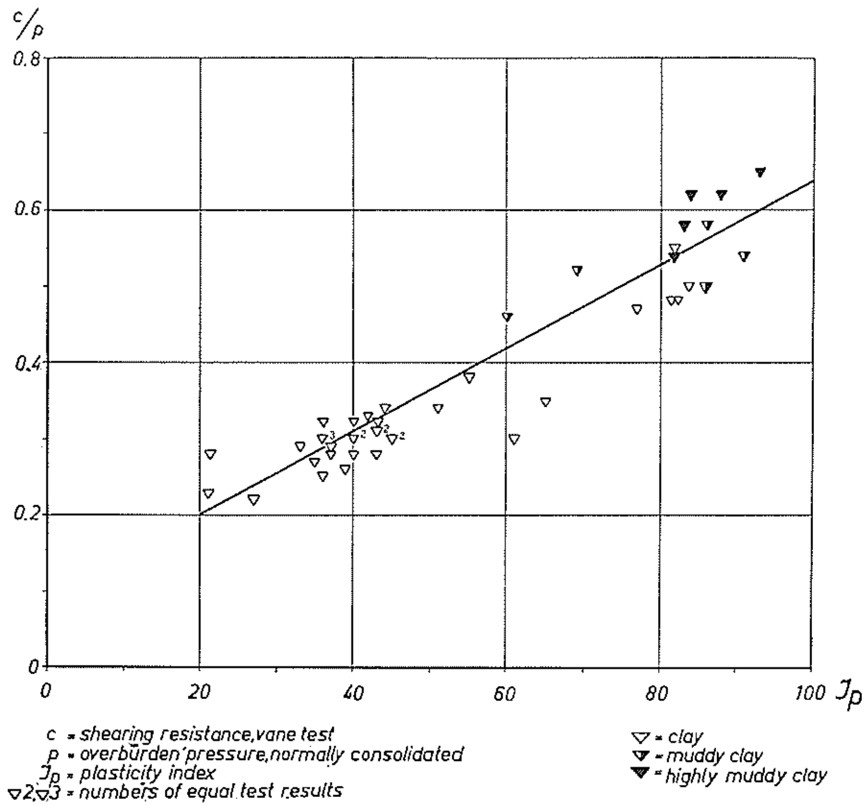


Fig. 7. Ratio between shearing resistance c and overburden consolidation) pressure p as a function of the plasticity index.

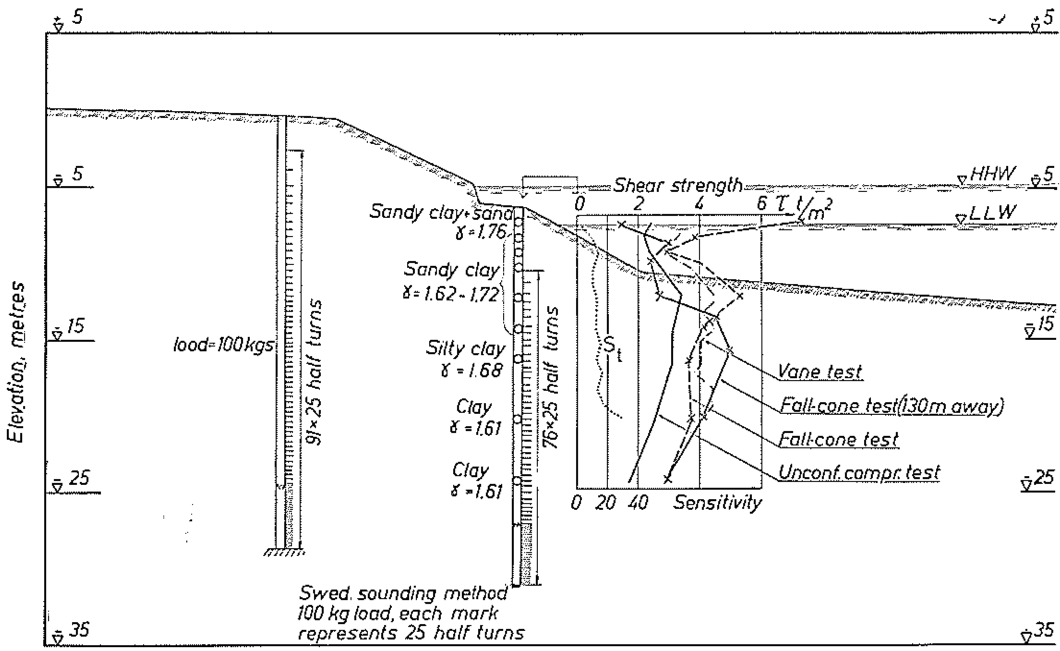


Fig. 8. Riverside slope with decreasing shear strength at greater depths in a tested profile.

pressure p as a function of the plasticity index I_P. The reduction just named should increase with increasing index.

The organic matter plays an important rôle for the strength. Figs. 4 and 7 show some earlier results, here revised with regard to organic content. The clays have been called muddy, when the loss of ignition has been round about 2.5 as percent of dry weight, or more. Also the type of mineral is important, see for instance Lambe et al. (1960).

Another problem of importance is the question of the swelling of the clay, due to suction, in Sweden mainly related to decrease in effective stresses. This suction effect may break the part of the cohesion which depends on over-consolidation.

In Swedish clays of low permeability this process seems to take a very long time. Old parts of a dry crust have been found which have lain under water for thousands of years and which are still very hard (compare the Final Report of the Geotechnical Commission of the Swedish State Railways).

On the other hand, at a test area at Väsby near Stockholm, where, as a consequence of the loading, water percolates the crust, the strength of it seems to decrease. In the valley of the Göta River in south-western Sweden

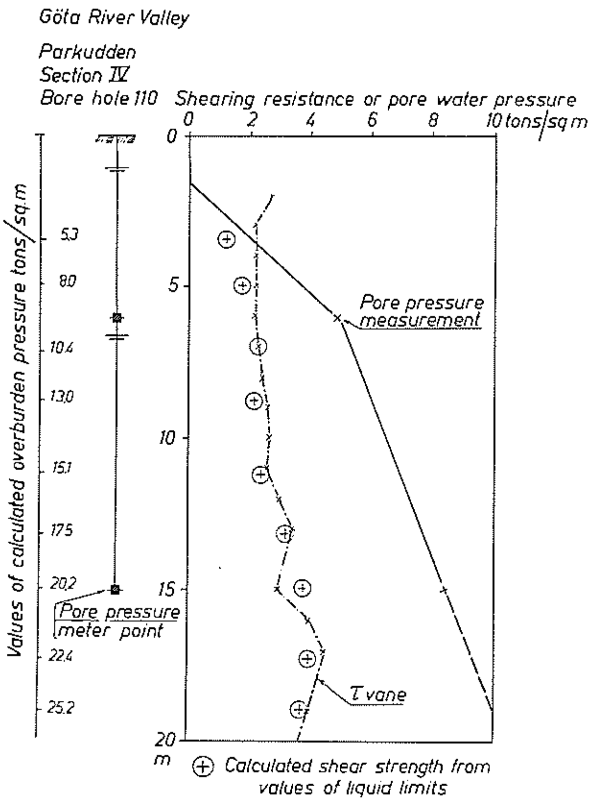


Fig. 9. Profile where the pore water pressures are increasing at a lower rate than the change in depth.

one often finds water under artesian pressure, especially in and around sand layers. This water flows to the river through the clay and sometimes in an upward direction. Obviously, this may explain some of the low strength results which are measured close to such permeable layers.

Fig. 8 may indicate such a phenomenon, where the undrained strength results obtained with various methods are lower at greater depth.

Fig. 9 on the other hand shows a case, where the pore water pressures are rather low in the lower part of the profile investigated, seemingly indicating a pore water flow downwards. The strength distribution is measured and calculated, the results of both methods being similar, indicating rather stationary flow conditions.

Bases of Strength Assumptions

In other countries, the modified Coulomb formula is often used for clay. It is written

$$\tau_f = c' + (\sigma_n - u) \tan \Phi' \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

where c' = apparent cohesion

Φ' = angle of apparent friction

σ_n = total pressure normal to the failure surface

At normally consolidated clays the origin values of c' can usually be omitted, and it should be possible to write

$$\tan \Phi' = \frac{\tau_f}{\sigma'_n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

In the drained case this formula is equal to Eq. (2). The Coulomb formula can thus be used at the first loading.

In the interpretation of the undrained triaxial test, however, some complications will arise. Suppose for instance that the test sample is consolidated for an all-around pressure σ_c. If the failure is caused by increasing the axial pressure by an amount $\Delta\sigma_1$ and, which may often be assumed, if the pore water pressure increases by the same amount, the normal pressure σ'_n will decrease, giving rise to difficulties of interpretation. If the state of consolidation should be dominated solely by σ'_1 this interpretation should give the same results as the drained shear-box test, in which σ'_1 is also higher than σ'_n.

The same problem will arise if the failure is performed in decreasing σ_3, and with zero pore water overpressures, $\Delta u = 0$.

Fig. 10 shows the results of a number of undrained triaxial tests in relation to the plasticity index (examined strictly after the modified Coulomb formula, corrected for measured pore water pressures, see Fig. 11a), which differ, however, very much from the Φ_d-values of the shear box. Also the relationship is rather strange and the scattering of results is high.

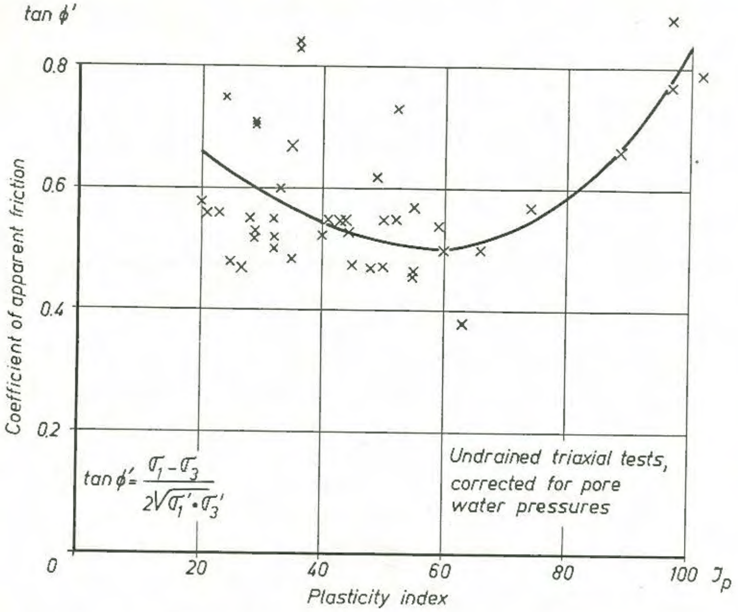


Fig. 10. Apparent angle of shearing resistance plotted against plasticity index, in undrained short-term triaxial tests, using effective stresses.

There may be many reasons for that. One is the all-around consolidation and the time effect, which must be of influence. Another lies in the pore water pressures set up, and also the manner of interpretation. Besides, the formula may not fit.

We will try the Skempton (1954) formula for pore water pressures on some extensively tested soils

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)] \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

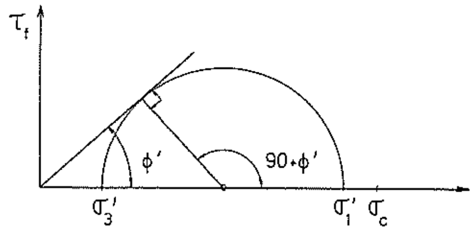
where $\Delta\sigma$ = increment of stress
A = constant $\quad B$ = constant

The following results are obtained. Firstly, in the case of performing the triaxial test by decreasing σ_3, keeping $\Delta u = 0$ at the same time, one gets

$$A = -\frac{\Delta\sigma_3}{\Delta\sigma_1 - \Delta\sigma_3} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6a)$$

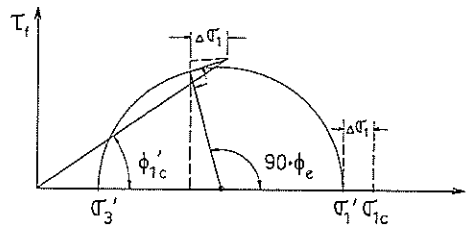
This case gives in some normal consolidated cases the following A-values. (Some additional data of the test specimens are given in Table II, the isotropic consolidation pressure σ_c, the major principal stress σ'_{1f} at failure, minor principal stress σ'_{3f} at failure, etc.)

a



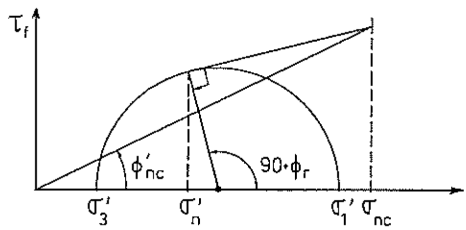
$$\tan\phi' = \frac{\sigma_1' - \sigma_3'}{2\sqrt{\sigma_1' \cdot \sigma_3'}}$$

b



$$\tan\phi_{1c}' = \frac{(\sigma_1-\sigma_3)\cos\phi_e + 2(\sigma_{1c}-\sigma_1')\tan\phi_e}{2\sigma_{1c} - (\sigma_1-\sigma_3)(1+\sin\phi_e)}$$

c



$$\tan\phi_{nc}' = \frac{(\sigma_1-\sigma_3)\cos\phi_r + 2\sigma_{nc}\tan\phi_r - [\sigma_1'+\sigma_3'-(\sigma_1-\sigma_3)\sin\phi_r]\tan\phi_r}{2\sigma_{nc}}$$

Fig. 11. Explanatory figure showing different methods of interpreting triaxial tests. In method *a* one also may find solutions with other tangent or secant points. In method *b* (when used strictly) one corrects for volume changes due to $\Delta\sigma_1$.

Lilla Mellösa	Test No. 1 013	A =	1.01
Surte	1 830		1.15
	2 319		1.11
	4 148		1.03
	6 694		0.94
Strandbacken	25D		1.32
	25E		1.21

Thus $A = 1.1$ on an average. Testing the same soils with increasing σ, and reading the Δu-values, one gets

Lilla Mellösa	Test No. 1 024	B =	1.01
	1 054		1.07
	1 058		1.08

Table II. Data of triaxial tests with reduction of stresses and keeping the pore water overpressure $\Delta u = 0$, (performed in the apparatus, type NGI)

Identification No.	Depth m	Volume weight t/m³	w%	w_L	w_p	I_p	σ_c kg/cm²	σ_{3f}	σ_{1f}	$\tan \Phi'$	$\tan \Phi'_{1e}$	$\tan \Phi'_{ne}$	Soil classification
1013 F 12	3 5	1.43	112	133	44	89	3.0	0.86	2.98	0.662	0.656	0.425	Dark-grey clayey nekron mud
2319 K 5314	7.5	1.66	55	60	25	35	1.15	0.30	1.06	0.67	0.60	0.475	Grey clay
1830 K 5314	7.5	1.59	58	69	25	44	3.10	1.0	2.82	0.55	0.49	0.42	Grey clay
6694 K 5314	10	1.70	53	54	27	27	4.15	1.76	4.32	0.47	0.475	0.425	Grey clay
4148 K 5314	10	1.70	45	50	26	24	2.15	0.75	2.11	0.54	0.53	0.47	Grey clay
25 D K 5314	20.5	1.59	70	87	32	55	4.8	1.65	4.03	0.46	0.405	0.355	Grey clay, slight sulphide bands
25 E K 5314	20.5	1.59	70	87	32	55	6.8	2.48	6.06	0.46	0.425	0.37	Grey clay, slight sulphide bands

Other tests have shown that A and B under certain conditions (especially at overconsolidation) vary more than given above. The assumption of the pore pressure increments being the same as the stress increments is thus not in strict accordance to reality.

Nowadays, in theoretical connections, the Hvorslev (1937) formula of shear strength is often used

$$\tau_f = c_e + (\sigma_n - u) \tan \Phi_e \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

where c_e = »true» cohesion
Φ_e = angle of »true» friction

In this formula, one has

$$c_e = \varkappa \cdot p_e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (7a)$$

where $\varkappa$ = coefficient for working cohesion
p_e = equivalent consolidation pressure (belonging to void ratio at failure).

The pressure p_e is referred to the vertical pressure p_1 in the oedometer test in the following way

$$p_e = p_1 \cdot \mathrm{e}^{B'(e_1 - e} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7b)$$

where e = void ratio.
e = 2.718 . . .

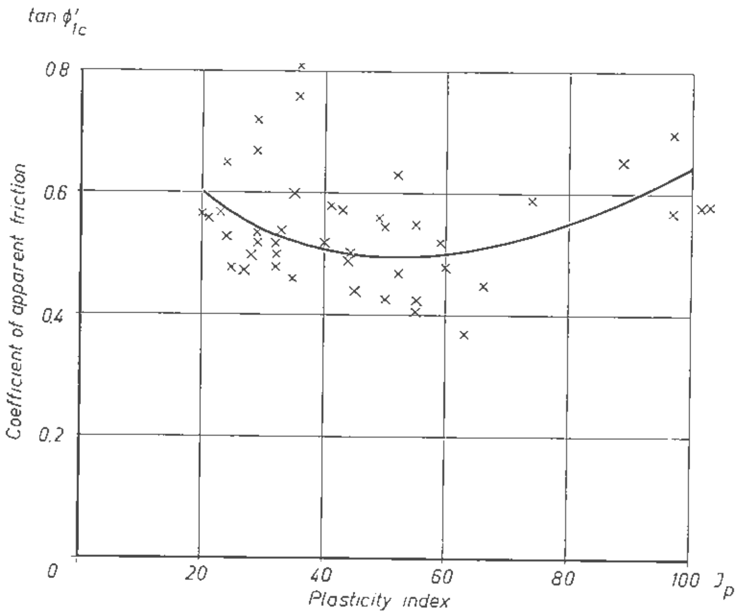


Fig. 12. Apparent angle of shearing resistance plotted against plasticity index, in undrained short-term triaxial tests, using effective stresses but corrected for changed σ'_1-stress.

The problem of the shear strength taken up from the Hvorslev formula is dealt with in the above-mentioned paper of Roscoe et al. as a problem in the space, the effective stress σ' and the void ratio e at failure being two of the axes.

It can be discussed whether the oedometer test in that connection really gives the proper values of the coefficients B' and $\varkappa$, which ought rather to be referred to a failure test. However, some proportionality is probable. Moreover, it seems reasonable that the particle shape and orientation, the quantity of organic matter, etc., should be of some importance.

Fig. 12 shows the same results as Fig. 10 but corrected for the differences between the major principal stress at consolidation and at failure, and for the direction of the failure surface (cf. Fig. 11b). This figure also shows a rather peculiar relationship between the coefficient of apparent friction, here called tan Φ'_{1c} and the plasticity index, and also some scattering of results.

Perhaps, the Hvorslev formula should not be used to interpret short-term tests. The problem of consolidation is obviously something more than the question of pore water pressures. One must discuss the cohesion due to secondary compression, etc.

Another formula in use is the Krey-Tiedemann formula (see Muhs 1957), which can be written

$$\tau_f = c_n + (\sigma_n - u) \tan \Phi_r \quad \ldots\ldots\ldots\ldots \quad (8)$$

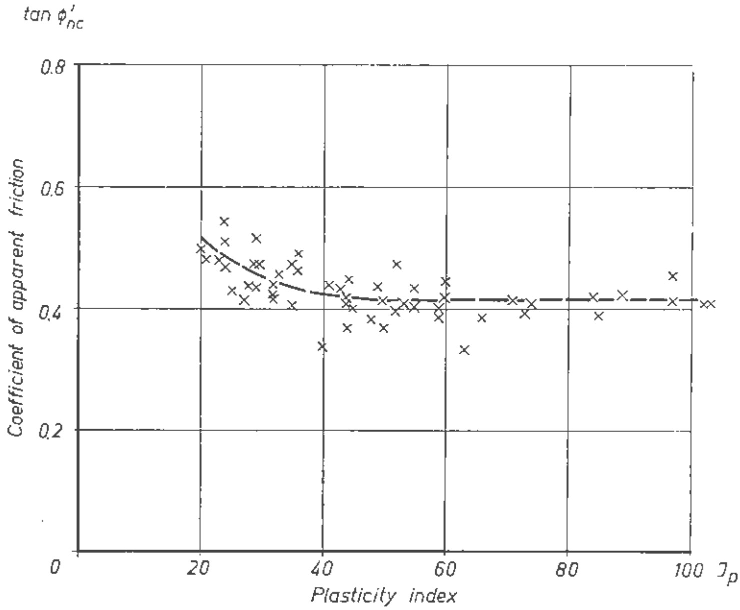


Fig. 13. Apparent angle of shearing resistance plotted against plasticity index, in undrained short-term triaxial tests, using effective stresses but corrected for changed σ'_n-stress.

where $c_n = \sigma_{nc} \cdot \tan \Phi_c$ . . . . (8a)

$\sigma_n - u$ = working pressure, normal to the failure surface

σ_{nc} = maximum working pressure, normal to the failure surface = = initial consolidation pressure

$\tan \Phi_c$ = coefficient of working cohesion

$\tan \Phi_r$ = » » » friction

The question of the accuracy of the Krey-Tiedemann formula is taken up by Hvorslev (1937), who points out the problem of swelling. This has also been mentioned by Muhs. In the case of clays of the Göta River Valley, however, the swelling is rather small.

Fig. 13 shows the same results as Figs. 10 and 12 but corrected for the differences between the initial consolidation pressure normal to the failure surface and the working pressure at failure, and for the direction of failure surface (cf. Fig. 11c). This figure, which also includes a few additional results, shows a rather smooth relationship between the coefficient of apparent friction, here called $\tan \Phi'_{nc}$, and the plasticity index.

At the latter calculation the present author has used a method suggested in his paper of 1960 to interpret the results of triaxial testing which, for the sake of simplicity, is graphic and approximate to be a simple complement to the usual pore water pressure correction in the Mohr circle version. Using the method just named the scattering of results of $\tan \Phi'_{nc}$ could be kept moderate for similar clays. When determining Φ'_{nc}, values of Φ_r are taken up from the

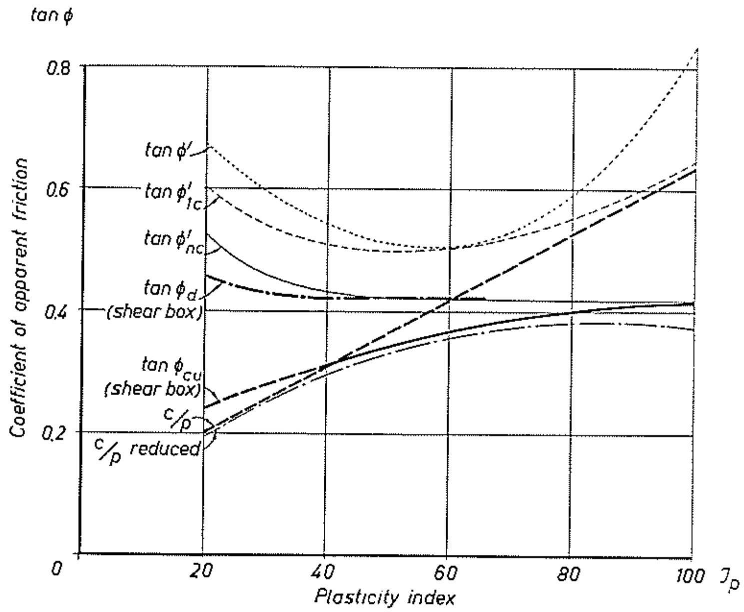


Fig. 14. Apparent angle of shearing resistance plotted against plasticity index, in normally consolidated tests or recalculated into effective stresses and normal consolidation, according to various interpreting methods.

Gibson (1953) diagram of Φ_r, which, however, should correspond to the Hvorslev angle Φ_e. The error thus introduced cannot be great.

The shear strength parameters were often earlier, when correction was made only for pore water pressure, derived in a way which will not hold true in the case of tests on Swedish clays. The most important reason for the deviations is probably prestresses, already mentioned by A. Casagrande and Wilson (1953), which may be allowed for according to the method of the above-mentioned paper of the present author.

These questions are most important when the consolidation pressures are higher than the actual pressures, the clay thus being in the range of overconsolidation. The isotropic consolidation in the laboratory may influence test results, a question touched upon by Bishop and Henkel (1953).

It seems also necessary to regard the influence of the swelling on the cohesion intercept.

Finally, the *Cohesion method* is also used. We know that it can be applied in the undrained case. We may for instance on the same points as were used for Figs. 10, 12 and 13 use the formula

$$\tan \Phi_c = \frac{\sigma_1 - \sigma_3}{2\sigma_c} \qquad (9)$$

where σ_c = the all-around consolidation pressure.

The values of tan Φ_c will then be in the region of 0.325 at $I_P \approx 40$ and 0.35 at $I_P \approx 100$, and the scattering will be on an average about 10 per cent, or nearly the same value as when using the Krey-Tiedemann principles of strength formulas in the version of the present author.

In Fig. 14 a compilation is made of average values of the above-mentioned results of the strength tests. In all cases the tests used are checked to be performed at stresses above the preconsolidation (at natural conditions) stress by control tests, usually 3—4 tests. The values plotted are regularly those from the tests with the highest load.

In the diagram a curve of the values of c/p is also plotted, reduced in a way similar to that used by Olsson. In most laboratory tests the strengthening effect of the secondary consolidation is left unsolved. For that reason, the curves fitted are rather conditional.

It may be asked which of the curves plotted in Fig. 14 should be used. The tan Φ'-curve seems obviously wrong. The tan Φ'_{1c}-curve may be too high, the interpreting method more suited to long-term tests. Of the curves from triaxial results, the minor scattering is on the Φ'_{nc}-curve which is also more conservative than the others, and which may be used when interpreting quick tests. The Φ_d-values for drained tests are also arrived at in a reasonable way.

Of the curves for undrained strength, the tan Φ_{cu}-curve at high values of I_p and the reduced c/p-curve are most conservative. The tan Φ_{cu}-values at low I_p do not seem to be performed on fully undrained samples, and are not to be used.

The choice of the test method should be made with due regard to the specificity of the actual problem. Time influences and manner of consolidation (and possible decrease in effective stresses) seem thus to be most important effects and these will be more carefully investigated later on. A final solution of the problem demands extended physico-chemical considerations, as for instance to thixotropic effects.

Natural conditions

Above, the interpretation of strength results, the influence of time, and particle orientation were discussed in the case of laboratory testings. Such influences can also be observed in natural conditions.[1]

Fig. 15 for instance shows some results of measurements on electrical resistance in the soil, performed vertically and horizontally on samples taken. As might be imagined, the latter resistance is smaller, the plane of most particles being orientated in a horizontal direction. This fact may arise initially at sedimentation or be caused later on by overburden pressures. The same effect can be studied on permeability tests, performed in both directions.

In Sweden, visible deformations can often be observed in the upper regions of slopes. This phenomenon may, however, be rather harmless, the outer parts

[1] B. Fellenius (1955) has pointed out creep settlements of piles in clay.

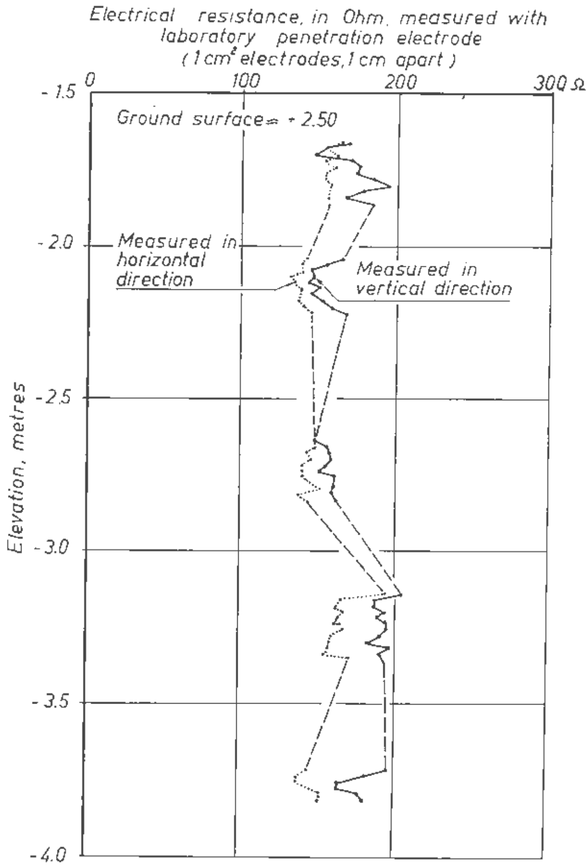


Fig. 15. Electrical resistance in Ohms, measured with laboratory penetration electrodes (1 cm² surfaces. 1 cm. apart), in Skå-Edeby clay.

of the slopes being in the state of slight creeping, but may also depend on a state of stress very near to failure.

If one dares to draw any conclusions from the theories of the above, creep will start at shearing stresses higher than those of final settlements of a level terrain. The plastic deformations will thus be limited to some high-stressed regions.

The velocity of creep may be in a certain relation to the amount of over-exertion. Fig. 16 shows a slope, where in a plastic region it is supposed that a slip failure will occur along a surface (or rather a zone), which is kinematically possible, taking into account volume changes that occur. As the sliding body is supported in a highly statically indeterminate way, the distribution of subgrade reactions must be regarded as rather arbitrary.

One often uses a definite shear strength value as a criterion on the resistance against sliding. That must, however, be made with some caution. In

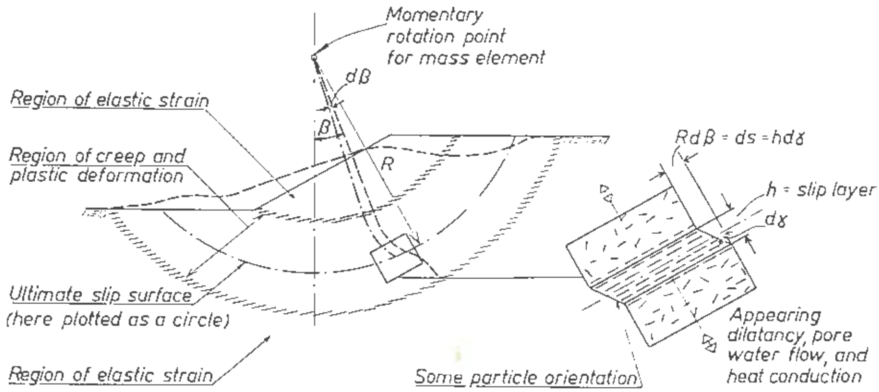


Fig. 16. Theoretical behaviour of high-stressed slope and principles of failure. Failure zone h in thickness.

this connection, see Peterson, Jaspar, Rivard and Iversen (1960), who state that the stability of embankments on slightly overconsolidated, very plastic clays is overestimated according to both the "total stress" method and the "effective stress" method, and recommend reduction of laboratory test results to avoid the calculated, too high safety factors.

The concept of a certain strength is also rather primitive in reality. As can be seen immediately, sand in bulk, for instance, has no cohesive strength, but it shows resistance to shear, mainly arising from grain contact forces which, during the shear, form works when the grains travel, see Osterman (1959).

For an accurate estimation of the shearing resistance of a slope one should really make an extensive study of the stability of the system from the thermodynamic point of view. Normally, however, one must confine oneself to get an idea of the main amounts of energy mobilized at a real or virtual displacement, arising during an estimated time, and to find out if that event will occur under drainage.

Thus, in the figure one may study the loss of potential energy of the sliding body, when it is turning at an angle of $d\beta$ (or in other cases travelling a certain distance). Strain-work appearing in the body proper can often be ignored.

The study consists also in estimating the work occurring in the slip layer h when it is sheared at an angle of $d\gamma$. The latter energy mainly comes from shearing and outer dilatancy.

At the deformation some energy may be dissipated by thermal effects during the time dt and pore water flow in a direction normal to the slip surface. Of special interest in this connection is any transfer between intergranular stress and pore water pressure arising from inner dilatancy, and changes in frictional resistance emanating from such transfer.

In simple practical cases the calculation must thus be confined to a study of the equilibrium conditions, with regard only to the main situation of drainage in the actual case. In extreme or theoretical cases it seems as if a thermo-dynamical investigation should be worth while. The principles of the mathematical treatment are, however, rather intricate, and will not be discussed here.

Acknowledgements. In making these studies various matters were discussed with Mr. G. Lindskog and Mr. D. Almstedt.

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