The Influence of the Colloidal Content on the Shear Strength of Clay

L'Influence de la teneur en colloïdes sur la résistance au cisaillement de l'argile

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SUMMARY

Soil coefficients determined by routine shear tests with clay specimens do not indicate the danger of a progressive failure in nature. For some years, a new method of shear testing has been in use in Vienna which reveals whether or not a clay or argillaceous rock is prone to slippage *in situ.* The method is based on repeated shear tests which are carried out alternately in reversed directions. In slippage-prone clays the friction angle in the shear plane drops to a very low final value φ_F. The smaller the value of φ_F, the higher the danger of a progressive failure. Test results prove that the colloidal content has a decisive influence on the shear property of clay and argillaceous rock.

SOMMAIRE

Les essais routiniers de résistance au cisaillement sur des échantillons d'argile ne révèlent pas le risque de rupture progressive du sol à échelle naturelle. Depuis quelques années, on utilise à Vienne une nouvelle technique de mesure de la résistance au cisaillement qui permet de prédire si une argile ou un rocher argileux sont sujets ou non au glissement *in situ.* Cette technique est fondée sur des essais répétés que sont exécutés alternativement en sens inverse. Pour les argiles sujettes au glissement, on observe que l'angle de frottement sur le plan de rupture diminue jusqu'à une faible valeur finale φ_F. Plus l'angle φ_F est petit, plus les risques de glissement sont grands. Les résultats d'essais démontrent que la teneur en colloïdes a une influence décisive sur les caractéristiques de cisaillement de l'argile et des roches argileuses.

AT THE FIFTH INTERNATIONAL Conference on Soil Mechanics and Foundation Engineering a report was given by the author (Borowicka, 1961) on comparative tests which had been carried out with sand and clay specimens in order to draw conclusions on the nature of internal friction and cohesion in clay. In order that new information about stress-strain relationships could be obtained, the specimens had to be subjected to stresses essentially different from those occurring in routine tests. This was achieved by shearing the samples several times under a reversed shearing direction. These tests showed that the stress-strain relationships are identical in principle for cohesionless sand and for disturbed clay, apart from immaterial differences, if positive and negative pore water pressures are excluded. For instance, the following observation was made with sand as well as with clay: It is well known that a soil specimen when first loaded undergoes a densification during a direct shear test carried out in the usual way. If the shear load is removed after the test without changing the vertical load one would suppose, according to the theory of elasticity, that an expansion of the sample would take place. On the contrary, however, further densification results in both initially loaded sand and clay. For this reason—namely, that initially loaded soil specimens behave in this case in the opposite manner from what would be expected according to the theory of elasticity—and for many others there can be no doubt that clay is a granular mass, having a soil particle skeleton with the property of internal friction.

Furthermore, tests have shown that the concept of cohesion in clay, i.e., of an intergranular shear strength which is acquired through preloading, is inconsistent with the test results. The stress-strain relationships existing in clay may be explained in a simple way only if an internal pressure p_k is assumed as being effective in the grain skeleton and which, interacting with internal friction, can produce the same effect as cohesion. The increased shear strength of preloaded clay may be explained therefore by means of an internal pressure that is effective in the grain skeleton after unloading.

Replacing cohesion with an internal pressure is not thought to be merely a formal matter. In fact, the internal pressure has to be considered a quantity which may be altered by any external influence acting upon the soil skeleton. For instance, not only a change in the state of stress but also in pore water conditions, or a temperature change, will exert an influence upon the value of the internal pressure. From its effects upon the shear strength of soils it may be concluded that the internal pressure is caused by forces acting at the contact points between the grains. It must be regarded therefore solely as a fictitious mean value, as is every stress existing in a granular mass. In this point it differs essentially from the pore water pressure which is a real hydraulic pressure and may therefore be measured directly.

The internal pressure must not be regarded as a soil coefficient inherent in the grain material. It is definitely not a soil constant but is a quantity that characterizes the state of the soil at any instant. The internal pressure may change with time, and may even drop to zero. Also, its value may depend upon the direction, and may change considerably from one point to another, and that within much wider limits than is generally assumed.

In carrying out the tests described by the author (Borowicka, 1961), a new routine shear test was developed in the soil mechanics laboratory of the Vienna Technical University. This test has been in use there for five years and has given satisfactory results. It may be carried out with disturbed as well as with undisturbed samples. Most suitable for the test is a shearing apparatus in which the lower part of the shear box translocates horizontally and which permits, in addition, shearing in opposite directions, as is the case with Krey's shearing apparatus.

Details of the test procedure have been described by the author (Borowicka, 1963a, 1963b). During slow primary shearing the volume of the specimen is kept constant by varying the vertical load while increasing the shear load in order to avoid excess pore water pressures in the specimen. The relation between the vertical load and the shear load at constant volume is greatly influenced by the previous history of the specimen. During the following quick removal of the shear load the volume of the specimen is kept constant by diminishing the vertical load. In the subsequent repeated shear tests, which are carried out alternately in opposite shearing directions, the vertical load is kept constant for simplicity. The shear tests are repeated until a constant value of the shear strength is measured.

Through the Vienna method of shear testing a shearing property of clay is revealed which is of paramount importance. In fact, there are two groups of clays. When tests are made with disturbed specimens of the first group, to which the silty clays belong, the angle of internal friction determined in primary shearing results in more or less of a constant soil coefficient which does not change under repeated shearing. As may be seen from Fig. 1, the shear strength

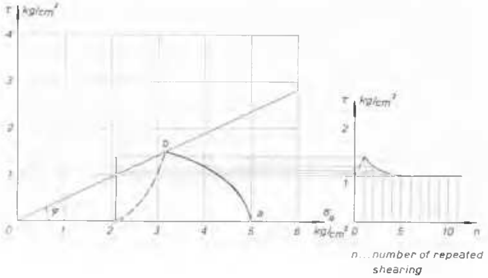

FIG. 1. Repeated shear tests with normal clay.

occasionally even rises slightly in second shearing but after repeated shearing drops again to the value corresponding to primary shearing. The transitory rise of the shearing strength in second shearing is due to the effect of the pore water.

In clays belonging to the second group, which often have a high colloidal component, the angle of internal friction determined in primary shearing continuously drops in repeated shearing until it reaches a very low final constant value (Fig. 2). This decrease of the friction angle is apparently caused by the fact that the scale-like and flake-shaped colloids adjust themselves towards the shear plane until they form a continuous, shiny shear surface (slickensided clays). This group of clays is particularly prone to slippage and slides because a progressive failure may occur in them, with the greatest part of the shear strength disappearing, while only the low final value of the friction angle in the shear plane will remain.

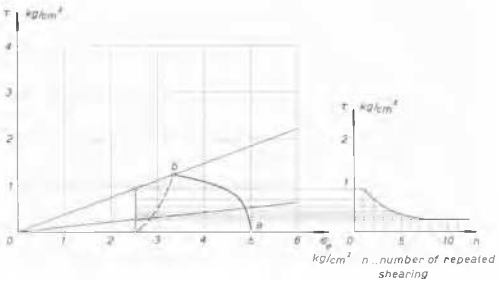

FIG. 2. Repeated shear tests with slippage-prone clay.

From a general point of view, the behaviour at failure of any granular mass may be characterized by three coefficients, viz.: φ, angle of internal friction in primary shearing; φ_F, final value of friction angle after repeated shearing; p_k, internal pressure at failure. In the special case, $\varphi = \varphi_F$, the soil has normal friction properties as they are assumed in every stability analysis applied in soil mechanics. In this case the angle of internal friction may be regarded approximately as a soil constant which accordingly applies for a disturbed as well as for an undisturbed soil. Actually, there are only a few types of soils for which the angle of internal friction is constant. Internal friction is caused mainly by an interlocking effect between the grains. As the position of the grains is changed during repeated shearing, the interlocking effect and consequently the angle of internal friction, may decrease within certain limits. Therefore, a drop by some degrees of φ at repeated shearing is quite common and a soil showing such behaviour is to be included in the group of soils with normal friction properties.

All soils for which the final value φ_F of the angle of friction after repeated shearing is much smaller than the angle of internal friction φ at primary shearing must be considered slippage-prone. To this class of slippage-prone soils belong especially clays of high colloidal content (slickensided clays) and some marls and micaceous soil types. The smaller the value of φ_F, the higher the danger of a progressive rupture in the soil.

It must be assumed that the drop of φ after repeated shearing is caused by the re-orientation of the scale-like individual grains which eliminates the interlocking effect in the shear plane. In a disturbed sample of clay, the individual grains are oriented at random. Orientation of the scale-like grains starts during the first shearing so that the angle of internal friction, φ, will in general be smaller in slickensided clays than in silty ones. In most cases, after completion of the test, the soil sample has disintegrated into two halves and the surface of the shear plane has acquired a shiny, slick appearance. In tests with disturbed samples the final value of the friction angle φ_F characterizes the absolute minimum value to which the shear strength may drop in the shear plane. After the test has been carried out an even, smooth shear plane has been formed. For a slippage-prone soil φ_F obtained from tests made with a disturbed sample may therefore be considered as a characteristic soil coefficient.

If the Viennese routine shear test is carried out with undisturbed samples, the strength in primary shearing will be higher due to an existing internal pressure. Owing to the variation of internal pressures from specimen to specimen the results for primary shearing will often be so greatly scattered that the angle of internal friction cannot be determined with sufficient accuracy, particularly when the internal pressure is high. In a slippage-prone clay the angle of friction must lie between the limits φ and φ_F for disturbed samples, corresponding to the actual orientation of the grains *in situ*. In undisturbed specimens the final value φ_F is always several degrees higher than in disturbed ones because no smooth shear plane will ever form due to the irregular internal pressures, so that a certain interlocking effect remains.

Some indication of the value of the internal pressure p_k existing in clays may be obtained from the unconfined compression strength, q_c, if the negative pressure, p_w, possibly existing in the pore water is also known:

$$p_k + p_w = (q_c/2)\ [(1 - \sin\varphi)/\sin\varphi]. \qquad (1)$$

Knowledge of p_k alone, however, is not very important in most practical cases. For calculating the instantaneous stability, it is sufficient to know $p_k + p_w$ whereas for the calculation of the long-term stability it must be considered that not only p_w but also the internal pressure, p_k, may drop to zero and therefore cannot make a reliable contribution to the shear strength. This is all the more true for slippage-prone soils where p_k can vary quickly thus causing rapid disintegration.

Furthermore, the fact must be pointed out that equation (1) is valid for the small specimen only on which the compressive strength, q_c, is determined. It must not be supposed that when a soil sample is removed from its natural environment and is prepared for a test that the internal pressure will remain unaltered. Such an assumption is approximately admissible for a low internal pressure only. Taking a sample at a high internal pressure may lead to a reduction as well as to a rise of this high internal pressure. The cause lies in the inherent variability of the internal pressure which is influenced by many effects.

The concept of a granular mass and of an internal pressure effective in it is not limited to soils but can be applied to rocks also. In fact, the same basic laws are valid for both soils and rocks. For both Coulomb's equation holds, in both the water pressure becomes fully effective in the water-saturated state, and in both excess water pressures may occur. Even the distinction between normal and slippage-prone rocks is equally as necessary as for soils. Not only a mass of rock split up by faults and cracks represents a granular mass but also a single, compact rock on which no fissures can be observed with the naked eye. Actually, however, it has been decomposed by fine fractures or structure and lattice faults so that it must be regarded as a granular mass in which an internal pressure is effective. Its strength properties differ from those of a soil *in situ* quantitatively but not qualitatively.

Since most slides and failures occur in clays and argillaceous rocks which consist of a slippage-prone basic material, it is of the highest practical interest that this property can be identified in the laboratory. It is certain that the way in which friction decreases in a soil or rock after failure depends on a number of effects, and many more tests will have to be made in order that all those effects can be investigated and determined.

It is obvious, however, that apart from its mineralogical character, the quantity of the very fine fraction (size less than 2μ) has a decisive influence upon the shear properties. In order to demonstrate this influence, normal Viennese clay was separated into a coarse and a fine fraction and specimens of various grain size distributions were prepared from them by renewed mixing. With the artificially produced specimens the soil coefficients shown in Table I were obtained.

In Fig. 3 the friction angles φ and φ_F are plotted against the fine grain content below 0.002 mm. It may be seen from Fig. 3 that for a colloidal content of more than 43

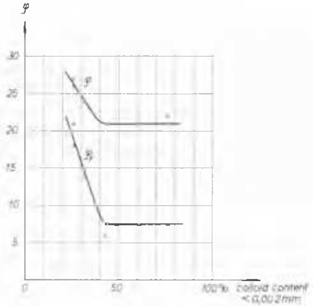

FIG. 3. Influence of the colloidal content on the shear strength of clay.

per cent the friction angles φ and φ_F are constant, apart from scattering of the plotted points. In that range the silty particles are apparently embedded in a basic mass of colloidal clay so that they do not exert any influence upon the shearing properties. During shearing a slickensided shear plane is formed which separates the specimen into two halves.

If the colloidal content of the specimen is less than 43 per cent, both φ and φ_F increase but their difference, $\varphi - \varphi_F$, decreases. After shearing, the two parts of the specimen stick to each other in the shear plane and only a part of the shear plane is slickensided. This is obviously the cause of the increase in φ and φ_F. There are not enough colloidal particles to form a slickensided area which covers the whole shear plane. The interlocking effect of the coarse-grain fraction becomes more effective with decreasing colloidal content.

The test results show clearly the decisive influence of the colloidal content on the shear property of clays, and

TABLE I. SOIL COEFFICIENTS OF ARTIFICIALLY PRODUCED SPECIMENS

Sample no.	Colloidal content, grain size less than 0.002 mm (per cent)	Plasticity index (per cent)	φ	φ_F
1	76	52*	21.5*	7.5
2	61	52.5	21	7.5
3	44	40.5	21	7.5
4	43	29.5	20	6
5	32	25	24	14
6	26*	24	26.5	24

*Average taken from two tests.

TABLE II. SOIL COEFFICIENTS OF HOMOGENIZED SPECIMENS

Sample no.	Colloidal content, grain size less than 0.002 mm (per cent)	Plasticity index (per cent)	φ	φ_F
1	16	8.5	32	32
2	16	9.5	29	29
3	18	11	27	18
4	16	11	31	16
5	21	13.5	30	13
6	21	12	28	13
7	14	7	30	11
8	23	12.5	27	10
9	28	17	22	7

especially on their behaviour after rupture. It must be taken into account that the clay minerals were identical, the specimens having been prepared artificially. Therefore, the test results are valid only for this special case and must not be generalized.

In order to show to what extent the shear properties *in situ* may scatter, nine samples were taken from one stratum of reddish-coloured hard marl. With the naked eye no difference could be observed between the individual samples. The soil coefficients obtained from the homogenized samples are shown in Table II. One would have expected that the samples would have given at least approximately equal composition and equal shearing properties. Actually, however, the final values of the friction angle φ_F comprise the whole range found in these rock types. In order to make a correct estimation of a soil stratum, therefore, one will always have to examine a great number of samples. Although a general dependence of the final value φ_F upon the colloidal content may be observed in these tests also, the scattering is rather wide (specimens 3, 4, and 7). From this it follows that in addition to the colloidal content other causes, such as mineral and chemical composition, must play a role. For the plasticity index similar considerations are valid.

On the basis of the tests carried out to date, of five years' experience with the Vienna method of shear testing, and by comparing the test results with the behaviour of the soil or rock *in situ* it may be stated that the final value of the friction angle, φ_F, offers a measure of the danger of slides. Without knowing φ_F, no judgment can be made on the actual stability of a slope.

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