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of rupture have the abscissae n_1 and n_2 respectively. By transferring these values into Fig. 9b we subdivide the normal stresses n on the surface of rupture AC_1 into three groups with an intensity between 0 and n, n_1 and n_2 and $> n_2$. For the stresses of each group the shearing resistance is determined by the corresponding sections of the broken line A_1 B C M_3 in Fig. 9c and the areas subject to the stresses of each group are shown on the horizontal axis of Fig. 9b. The equations for computing the shearing resistance for these sections are

Sections Aa and dC_1	$t_s = c_1 + n \tan \varphi_1$
" ab and cd	$t_s = c_2 + n \tan \varphi_2$
" bc	$t_s = c_3 + n \tan \varphi_3$

The remainder of the investigation is identical with the procedure described under the heading C). To make the tests a to c requires an intimate knowledge of soil-testing procedure. The only source of error on the unsafe side involved in the theoretical part of the method lies in the neglect of the excess hydrostatic pressure which may develop in the space between the unconsolidated core and the toe A of the fill on account of the drainage from this core towards the toe. However, under normal conditions this error is more than compensated by the opposite error associated with the assumption that the shearing resistance in the peripherical parts of the fill has the lowest conceivable values.

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No. G-7

STABILITY OF SLOPES OF NATURAL CLAY

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Explanation of terms and symbols.

w = water content of undisturbed sample taken at a depth z below the surface.
w' = water content of the same clay in a remoulded state after it was reduced from the liquid limit by consolidation under a pressure equal to the effective weight of the overburden at depth z.
A clay is said to be normally compacted if w is equal to or greater than w'.
Overcompacted clays are those for which w is appreciably smaller than w'.
w_p and w_l = plastic and liquid limits, respectively.
$I_l = \frac{w - w_p}{w_l - w_p}$, a ratio called the liquidity index. Its value indicates the consistency of the clay after remoulding it without changing the water content.
s = unit of weight of the clay, solid and water combined.
s_o = unit weight of the water.
t_s = shearing resistance
n = effective normal pressure (1)
c and tan φ = the constants in Coulomb's equation $t_s = c + n \tan \varphi$
c_m = average shearing resistance per unit of area of a surface of rupture computed from the data pertaining to a real slide.
Δn = compressive strength of the clay obtained from a simple compression test.
$z_o = \frac{\Delta n}{s}$ = greatest depth to which open vertical cracks in the clay can possibly extend.
z_1 = greatest depth of a surface of rupture below the surface of the ground.
All the numerical data are given in tons and meters.

Shearing resistance and secondary structural features. For the purposes of investigating the shearing resistance of undisturbed clay, this material may be classed into the following principal categories:
A) Soft, intact clays, free from joints and fissures. The results of the shearing tests are consistent with those obtained from triaxial compression tests.

B) Stiff, intact clays, free from joints and fissures. The results of shearing tests appear to be incompatible with those of triaxial compression tests.

C) Stiff, fissured clays. Like B), except for the presence of a network of joints and fissures. When it is dropped, a big chunk of such clay breaks into polyhedric, angular or subangular fragments with dull or shiny surfaces. The diameter of the fragments may range between less than one centimeter and more than twenty centimeters. In some cases, the jointing has been produced or intensified by preceding natural slides.

Soft, intact clays. In most cases the clays of this category are normally or slightly over-compacted and their liquidity index ranges between 0.5 and more than 1.0. A very common case of slides in clays of this type is shown in Fig. 1. A fill is constructed on a gentle slope A D underlain by the clay. Under the influence of the weight of the fill, the clay yields along a curved surface of rupture AA_1B. In order to obtain the smallest values of the effective normal pressure on the surface of rupture we assume that the application of the weight of the fill did not increase the effective normal pressures and that the weight of the soil in situ, located between AB and AA_1B is reduced by the full hydrostatic uplift.

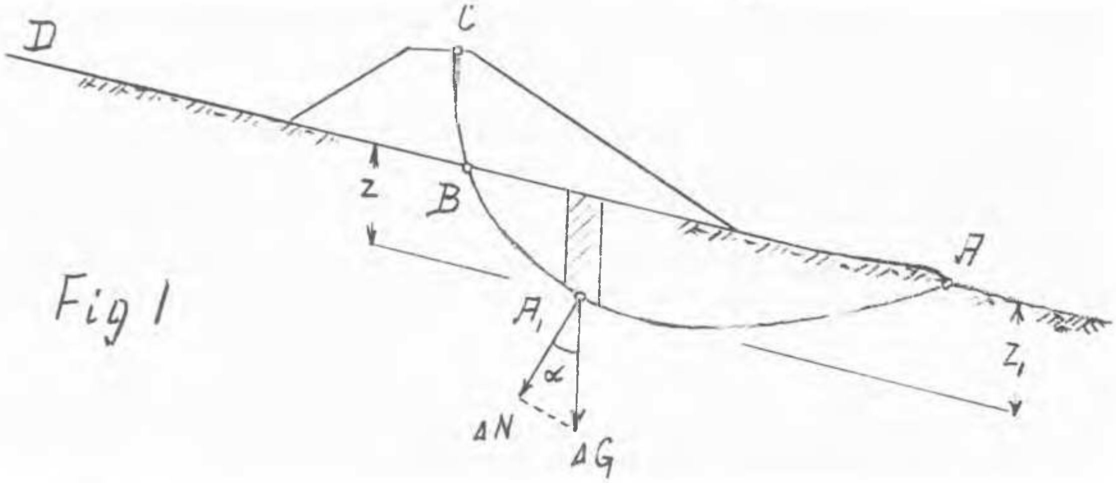

Fig 1

On this assumption the effective normal pressure on the surface of rupture at any arbitrary point A_1 is $z(s - s_o) \cos^2\alpha$ and the total shearing resistance along AA_1B with the total length ℓ is

$$c\ell + (s - s_o) \tan\varphi \int_0^\ell z \cos^2\alpha \, d\ell = c_m \ell \qquad (1)$$

wherein c_m is the average shearing resistance along AA_1B. For clays the value φ ranges between 20° and 30°. Assuming the average values $\varphi = 25°$, $\tan\varphi = 0.532$ and $s - s_o = 1.0$, we obtain

$$c_m = c + \frac{0.532}{\ell} \int_0^\ell z \cos^2\alpha \, d\ell$$

The value of the second term increases with z_1 because with increasing depth below the surface the frictional resistance produced by the weight of the soil also increases. Hence we can write

$$c_m = c + f(z_1) \qquad (2)$$

Under average conditions such as those shown in Fig. 1, the value $f(z_1)$ is approximately equal to 0.25 tons/sq.m for each meter of z_1. This means that for each 10 meters of z_1 the value c_m should increase by about 2.5 tons per sq.m.

In order to find out whether this conclusion agrees with general experience, Table I was compiled.

TABLE I

SHEARING RESISTANCE OF SOFT, INTACT CLAYS

Liquidity index	(a)		(b)			(c)			(d)
	No.	c	No.	z_1	c_m	No.	z_1	c_m	c
	1	1.9-2.0	4	6	1.4	8	15	0.8	1.5 to
ca. 1	2	1.5-3.2	5	5	2.0				3.6 inde-
			6	35	2.1				pendent of
ca. 0.5	3	5.5-6.4	7	10	1.3	9	10	5.0	depth

(a) determined by laboratory tests, (b) estimated from slides, (c) from catastrophic subsidence of buildings and (d) from pulling tests on piles. c and c_m in tons/sq.m and z_1 in meters.

Description of cases:

1: Fresh-water clay from depth of 10 to 15 m Bregenz, Austria.

2: Glacial clay from depth of 15 to 20 m, Lynn, Mass.

3: Boston blue clay.

4: Slide due to weight of a fill acting on a slope consisting mostly of clay underlain by gravel, Vita Sikudden, Sweden.

5: Partial displacement of clay with horizontal surface due to weight of fill, 6 m high, Järna, Sweden.

6: Slide in clay, Stigvergskai, Gotenburg, Sweden. (Fellenius).

7: Slide in big pocket of glacial clay, Södertalje, Sweden, (Fig. 3a)

8: Catastrophic subsidence of structure, 38 by 15 m into bed of soft, marine clay, Tunis, North Africa.

9: Catastrophic subsidence of grain-bin into bed of stiff clay, Winnipeg, Canada.

Although the experience expressed by the contents of this table is still entirely inadequate to justify final conclusions, the table contains no evidence of an increase of the average shearing resistance c_m with the depth z_1.

Last fall the author had an opportunity to examine very extensive slides of the type illustrated by Fig. 1 in an unusually homogeneous bed of intact glacial clay with a liquidity index somewhat smaller than unity. From the results of exploration by means of shafts and borings, it appeared that the depth of z_1 was equal to at least 10 meters. The width A B, Fig. 1, exceeded 50 meters in some places. Although the laboratory investigations are not yet terminated, there is ample evidence that c is approximately equal to 2 tons/sq.m. As indicated by the discussion following equation (2), the value of c_m should be equal to at least 4.5 tons/sq.m. In reality it is slightly smaller than 2.0 tons/sq.m.

Hence the computation of the stability of slopes on soft, intact clays still involves a vital element of uncertainty. For the time being it does not seem justifiable to assume that the shearing resistance along the surface of rupture is appreciably greater than c regardless of what the value of z_1 may be. In this connection it should be mentioned that the natural water content of soft intact clays seldom shows any tendency to decrease with depth, any variation in the water content being quite erratic. Similarly, the skin friction on piles driven into such clays does not tend to increase with depth. No vital information regarding this subject can be obtained except from thorough investigations of slides combined with laboratory investigation of undisturbed samples secured at the site of the slides.

Stiff, intact clays. No experience is available concerning clays of this type, because they are rather rare.

Stiff, fissured clays. The characteristics of this group prevail among intensely overcompacted clays with a liquidity index close to zero. In exceptional cases they are also found among clays which have retained a high liquidity index in spite of the fact that they carried a heavy overburden for some period of their geological history. The compressive strength Δn of the fragments of these clays is usually far in excess of 20 tons per sq.m. Yet the clays of this type are likely to represent the seat of very troublesome slides on gentle slopes regardless of the value of Δn.

In order to visualize the mechanics of such slides it suffices to consider that the compressive strength Δn of the unweakened material permits fissures to remain open from the surface down to a depth $z_0 = \frac{\Delta n}{8}$. For $\Delta n = 50$ tons per sq.m, which is by no means unusual, the depth z_0 would be approximately 25 m.

As long as the clay remains in its natural state, the existing joints have no mechanical consequences because, owing to the intense lateral pressure in the clay, they are almost closed, as shown in Fig. 2a. However, as soon as a cut is made, the state of the stress in the clay adjoining the cut is altered. Owing to the removal of the lateral support along the slopes the clay becomes subject to very unequal lateral expansion involving the opening of the joints. If the depth of the cut is appreciably smaller than z_0 this process involves both the slopes and the bottom. As a consequence, the pressure produced by the weight of the overburden becomes concentrated on a small fraction of the original bearing areas as shown in Fig. 2b. During rainstorms water accumulates in the fissures and the clay swells under zero pressure along the walls of the open cracks, although these cracks may be located at an appreciable depth below the surface. The process of non-uniform swelling weakens the fragments and new cracks are formed, like those indicated in Fig. 2b by dotted lines. Once the displacement along these cracks begins, the lubricated sides of the walls of the joints come into contact with each

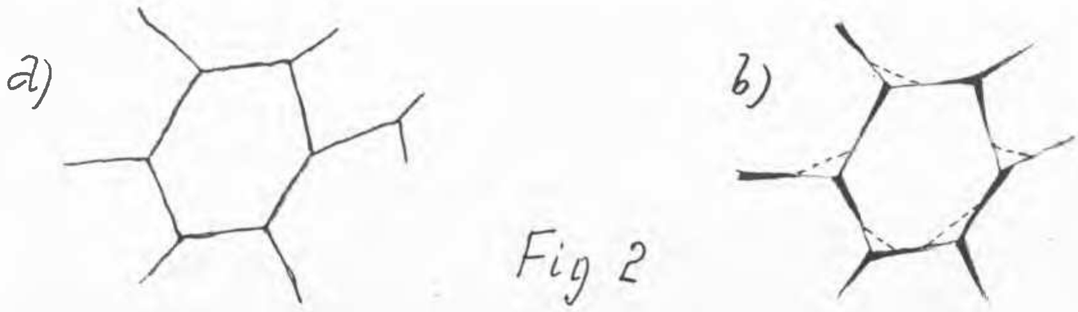

Fig 2

other. Since the excess water in the softened material along the walls has no opportunity to escape, the shearing resistance along the contacts is reduced to the value it would have under zero pressure in the laboratory.

This picture is strikingly confirmed by experience. Fig. 3a shows a slide in a clay of the soft, intact type and Fig. 3b one in a clay of the stiff, fissured type. In both profiles the depth z_o is marked by a dash-dotted line. In Fig. 3a the depth z_1 is several times greater than z_o, while in Fig. 3b it is only a small fraction of z_o. These two figures disclose at a glance the fundamental difference between these two slides. The slide shown in Fig. 3a represents a failure due to stresses in excess of the full resistance of the material, while the slide of Fig. 3b represents failure along zones of local weakening within a very deep zone of potential disintegration.

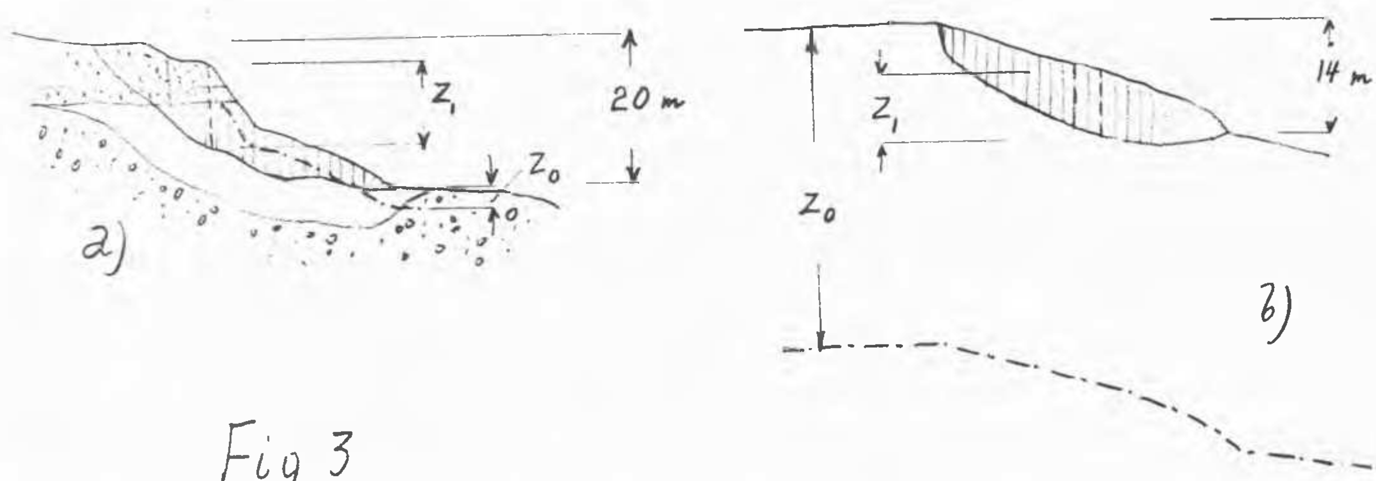

Fig 3

Table II contains a set of data pertaining to slides in stiff, fissured clays. In three cases, (1), (2) and (3), succesful shearing tests were made and in each case the value c_m was found to be approximately equal to c, although in each case the value of c was extremely small compared to what one should have expected from the results of the simple compression tests. However, our experience is still too limited to permit a decision as to whether this was more than a mere coincidence. Attention should be called to the fact that many of the values of c_m for the clays in Table II are hardly twice as great as those for the clays in Table I, while the compressive strength of the clays in Table II was from ten to twenty times greater.

TABLE II

SHEARING RESISTANCE OF STIFF, FISSURED CLAY COMPUTED FROM SLIDES

h = height of slope in m, i = average slope (height:base), c_m in tons per sq.m.

Case No.	h	i	c_m
1	7	(1:5)	3.0
2	14	1:2.4	2.5 - 3.0
3	14	1:3	3.1
4	32	1:3	10.0
5	10	1:3 to 1:1.7	2.5 - 3.4

Description of cases:

(1) Tertiary clay, I_l ca. 0, fissuration mostly due to old slides. Slide produced by weight of fill, 5 m high, on slope 1:5.

(2) Glacial clay, I_l ca. 1.0, n = 40 to 80 tons per sq.m. Surface of deposit previously loaded with 50 to 80 tons per sq.m by superimposed strata. St. Lawrence River, Canada. Shown in Fig. 3b.

(3) Heavy, rust-brown, residual clay with weathered pieces of porphyritic rock. I_l = 0. Railroad cut, Costa Rica.

(4) Intensely overcompacted, very hard clay, broken up into very small, angular fragments with polished surfaces. Railroad cut, Rosengarten, Germany.

(5) Intensely overcompacted, Jurassic and Cretaceous clays, Mittelland Canal, Germany.

Methods of stabilizing the slides. Slides in soft, intact clays can only be stabilized by reducing the slope. Slides in stiff, fissured clays can be remedied by injecting cement grout under low pressure. The hardened cement forms a network of veins which increase the value c_m and counteract a further disintegration of the fragments located between the joints. This method was successfully used on the Mittelland Canal in Germany.

Effect of chemical and other factors on the stability of clays. Every clay will slide when excavated at a slope greater than tan φ provided the excavation is made deep enough and for the same slope in the same clay the critical height depends on the average water content, on the conditions of jointing and on the difference between the real and the theoretical shearing resistance of natural clays illustrated by Table II. Therefore the effect of chemical and other factors on the stability of clays can only be expressed by their effect on the values c and φ, as obtained in the laboratory under well-defined conditions (see Ref. 1) and from their effect on the value c_m obtained from data pertaining to actual slides.

According to Kirchhoff (2) the degree of stability of clays depends on the value of the product M of the percentage of soil particles greater than 0.005 times the (CaO + MgO) content of the soil, both in per cent of the dry weight. For stable clays in the region investigated by Kirchhoff the value of M was greater than 360. However, this author failed to investigate the influence of the factor M on c_m. Since the publication contains neither any profiles of the slides nor the results of shearing tests and of stability computations, no practical conclusions of general validity can be derived from the results of his observations. The same comment applies to the empirical fact that the troublesome slides in the Dutch Indies seem to occur exclusively in clays whose sticky limit is below the liquid limit (3). The value of this important observation would also be increased by the results of shearing tests and a computation of c_m from data pertaining to actual slides.

According to K. Endell and his co-workers the values of c and φ represent some function of the mineralogical composition of the clays and the nature of the absorbed kations, but the nature of these relations is not yet known in detail (4).

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