STABILITY ANALYSIS OF SLOPES IN TERMS OF EFFECTIVE STRESS

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SYNOPSIS

Of the different types of analysing the stability of earth slopes the most convenient is known as the Swedish method, in which the circular slide surface can be commonly used. The so-called $\varphi=0$-analysis, based on the result of undrained shear tests, such as in situ vane tests or unconfined compression tests in the laboratory, is not appropriate for estimating the long-term stability, though it might give a correct result under fortunate conditions, when the stability of slopes just after a construction operation is treated.

This paper concerns with some important problems involved in another analysing method, called as c, φ-analysis, in which the shear strength of soils is expressed in terms of effective stress. The stability analysis is applied to a slide occurred along a small river in the Norwegian quick clay. Combining the results of borings, samplings, in situ tests and pore pressure measurements in the field, together with those of laboratory tests, it is concluded that the composed slide surface should be used, in stead of simple circular surface. As the cause of the slide it is assumed that considerably large amount of excess pore pressure in the clay layer might contribute to the occurrence of the slide.

In the conventional method of stability analysis, the failure envelope CB (shown as C_1B_1, C_2B_2, ...) in Fig. 1 (b) has been used in order to determine the shear strength parameters c'_0 and φ'_0 as the apparent cohesion and the apparent angle of internal friction, respectively, which have been supposed to be applicable to the stability calculation of overconsolidated soil. But it is wrong, because the shear strength of overconsolidated soil does not follow with the depth from the ground surface along the line CB. In another expression, the line CB is applicable to the field only at one point, B', whose abscissa is equal to the present overburden pressure $p_0=rH$. This can be shown as follows:

Let it be considered that two points are taken in the ground with the depth of H_1 and H_2, respectively. As the ground is overconsolidated, there exist such possibilities on the geological history of this ground that some height of soil mass has been eroded, or the ground water level has lowered down from the present elevation in earlier time. By means of oedometer tests performed on two specimens taken respectively from certain depths different from each other, we can find the exact amount of the precompression load p_{c1}, p_{c2}, respectively, on the void ratio-pressure diagram as shown in Fig. 1 (a). These points should accord with the break points B_1 and B_2 on the lines C_1B_1E and C_2B_2E, respectively, in the Mohr's diagram as Fig. 1 (b). The normal pressures acting on these points under the present

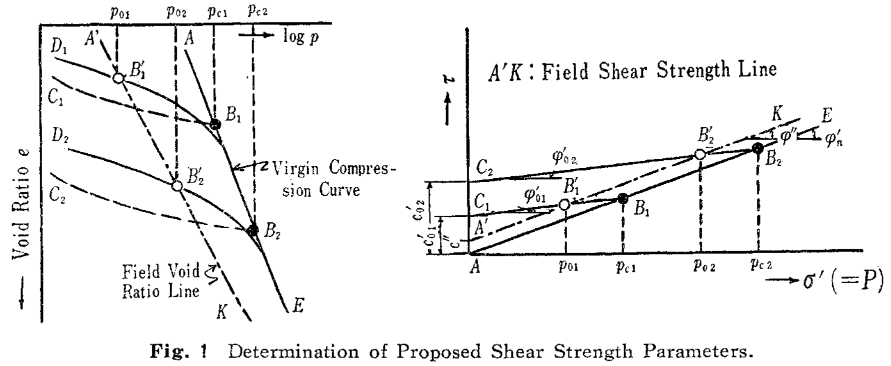

Fig. 1 Determination of Proposed Shear Strength Parameters.

geological condition are somewhat smaller than the precompression loads, and they can be estimated as p_{01} and p_{02}, according to the present overburden pressures. Thus the points B'_1, B'_2 in Fig. 1 (a) and (b) correspond to them, respectively, and as already described, these points are those, that should be applied to the field on the failure envelopes C_1B_1 and C_2B_2 in Fig. 1 (b). The locus of these Points, $A'K$-line, should be called as the field void ratio line in Fig. 1 (a) and as the field shear strength line in Fig. 1 (b). The latter, which is just useful for the stability calculation of slope whose slide surface might pass through various depths in the ground, has in general an angle of inclination of φ'' and an interception to the ordinate of the diagram as c''.

Since in the case of normally consolidated soil the field shear strength line $A'K$ accords with the failure envelope in normally consolidated state AE (Fig. 1 (b)), the angle φ'_n (with $c'_n=0$) can be used as the strength parameter, just as in the conventional method.

In situ vane tests, unconfined compression tests and cone tests on samples obtained in the slide area are performed to determine the shear strength parameters of clay. As a result it is found that the quick clay is normally consolidated in the part of river bank, whereas it is somewhat overconsolidated in the part of river bottom due to the erosion of the river. Thus the cohesion of the clay can be expressed as a function of eroded height of the ground.

If the soil approaches to the state of normal consolidation, the field apparent cohesion c'' defined by the author in Fig. 1 (b) decreases to zero, and there is no markable change in the apparent angle of field shear strength φ''. Since it is reasonable to consider that this change in c'' with depth from the original ground surface goes on in such a manner that it is proportional to the amount of eroded height, we can take $c''=0$ for the soil in the part of river bank and $c''=0.5\ \mathrm{t/m^2}$ for the river bottom, and $\varphi''=20°$ for all situations, from the result of triaxial tests.

Using the following formula proposed by Bishop for the circular slide surface:

$$F=\frac{1}{\Sigma W\sin\alpha}\cdot\Sigma\frac{c''b+(W-u_sb)\tan\varphi''}{\cos\alpha+\dfrac{\tan\varphi''\cdot\sin\alpha}{F}} \quad (1)$$

the minimum safety factor is as high as 1.06, whereas the formula recently proposed by Janbu for the non-circular surface:

$$F=f_0\frac{1}{\Sigma W\tan\alpha}\Sigma\frac{c''b+(W-u_sb)\tan\varphi''}{\cos^2\alpha+\dfrac{\tan\varphi''\cdot\sin\alpha\cdot\cos\alpha}{F}} \quad \cdots\cdots(2)$$

gives the safety factor of 1.03, with the correction factor $f_0=1.06$.

As the result of $\varphi=0$-analysis computed by the formula:

$$F=\frac{\Sigma sl}{\Sigma W\sin\alpha} \quad \cdots\cdots(3)$$

we obtain the safety factor as high as 1.24, even when the tension crack is taken into consideration. It can be concluded, therefore, that a stability calculation by means of $\varphi=0$-analysis gives a result on the unsafe side, if it is applied to a slide caused by the effect of excess pore pressure.

In conclusion, it is pointed out in this paper that the conventional $\varphi=0$-method based on the undrained test of soils, such as the vane test and the U-test, cannot provide a satisfactory factor of safety in the stability analysis of slopes, when one deals with their long-term stability. The c, φ-method, in which the strength of soils is expressed in terms of effective stress, is therefore recommended as more reasonable procedure in the analysis. Applying the latter to the stability of slopes, it is shown that the field shear strength line proposed here by the author should be used for determining the strength parameters of overconsolidated soils, instead of the apparent Mohr's failure envelope.