Total stress approach for shear strength of unsaturated soils

L'approche de contraintes totales pour la résistance au cisaillement de sols non saturés

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ABSTRACT: Data is presented verifying the inertness of pore water pressures in stiff clays to stress changes as Skempton's pore pressure coefficient B drops below about 0.2. In such cases, a total stress analysis based on a direct measurement of strength through undrained tests, more easily applicable on a larger area, is shown to give more reasonable values of factor of safety for the short term stability of slopes in fissured and/or stony clays than effective stress analyses based on small specimens.

RÉSUMÉ: Des données sont présentées qui vérifient que les pressions interstitielles d'eau dans les argiles raides deviennent plus inertes quand le coefficient B de pression interstitielle de Skempton se baisse à moins d'environ 0.2. On montre que dans de tels cas une analyse de contraintes totales basée sur une mesure directe de la résistance au cisaillement à partir des essais non drainés, qui peuvent s'effectuer plus facilement sur une superficie plus grande, donne les facteurs de sécurité à courte terme des pentes dans les argiles fissurées et/ou pierreuses, plus raisonnables que des analyses de contraintes effectives basées sur de petits échantillons.

1 INTRODUCTION

Bishop (1966) quotes the results of in situ large shear box tests for studying the effect of sample size on the undrained shear strength of the overconsolidated fissured London Clay. This is a saturated deposit for which Skempton's (1954) pore pressure coefficient B is unity. For the unsaturated stiff fissured Ankara Clay, where local infiltration or a nearby perched water table exists as in the cases to be quoted, B is generally less than 0.1; where no such wetting has occurred, B is much less. Applying undrained tests on a larger area in fissured and/or stony clays seems to have an increasing advantage for a wide range of soils having B values between 1 and 0.1, but is certainly so for those with B less than those presented here. Here, previously published results are summarized and supported by field observations to substantiate this view.

2 IN SITU SUCTION MEASUREMENTS AT SIDE OF CUT

For the first three slips (Table 1) in the stiff fissured unsaturated Ankara Clay, Mirata (1980) had presented comparisons of the factor of safety F_s based on a total stress analysis using undrained shear strength parameters (c, ϕ) obtained using the in situ wedge shear test (*iswest*) (Mirata 1974) (initial shear plane area = 900 cm^2) and those based on effective stress analyses using triaxial tests on 36 mm and 102 mm dia. specimens (yielding c', ϕ') and values of pore water pressure u_w based on laboratory measurements. Shear strength was assumed to be governed by the difference between total normal stress and u_w in the latter analyses. To verify the laboratory estimates of u_w in situ measurements had been made (Mirata 1976) prior to and after a vertical cut at site E (Table 1), some 400 m south of Slip 2, using 2-bar air entry value ceramic cups, to the top of which were attached the components (Fig. 1a) enabling them to be flushed with de-aired distilled water. Measurements were made using a mercury manometer coupled with a null indicator, a valve block and a screw piston mounted on a portable frame.

In September 1975, three such piezometers were installed in the positions shown in Figure 1b, using the procedure employed by Vaughan & Walbancke (1973), close to a potential slip surface at the side of a proposed vertical cut. Water level in a nearby drainage trench stood at 3.5 m below the ground surface. Measurements using these piezometers before the cut was made and at various times after the cut are summarized in Table 2.

Table 1. Particulars of the slips analysed and the cut at site E

Site	Slip 1	Slip 2	Slip 3	Slip 4	E
1. Height of slope (m)	5.5	4.1	4.5	4.3	3.1
2. Average inclination (°)	90	66	90	76	90
3. Liquid limit (%)	92	77	79	76	55
4. Plastic limit (%)	32	29	26	28	18
5. Natural water content (%)	34	35	29	33	23
6. Clay fraction (%)	63	57	52	56	38
7. Degree of saturation (%)	93	93	97	95	91
8. Estimated p_c** (kPa)	1000	700	800	1000	600
9. Pore pressure coefficient B	.046	.242	.070	.021	.069
10. Estimated in situ u_w (kPa)	-40	-16	-45	-30	-40
11. c' from triaxial tests (kPa)	27.5	4.0	24.5/0.1*	41.9	42.8
12. ϕ' from triaxial tests (°)	26.4	23.3	28.9/33.1*	28.5	20.9
13. c from iswests (kPa)	15.5	6.0	13.5	...	...
14. ϕ from iswests (°)	36.6	35.1	42.7	...	...

* From 102 mm diameter specimens. ** Past preconsolidation pressure.

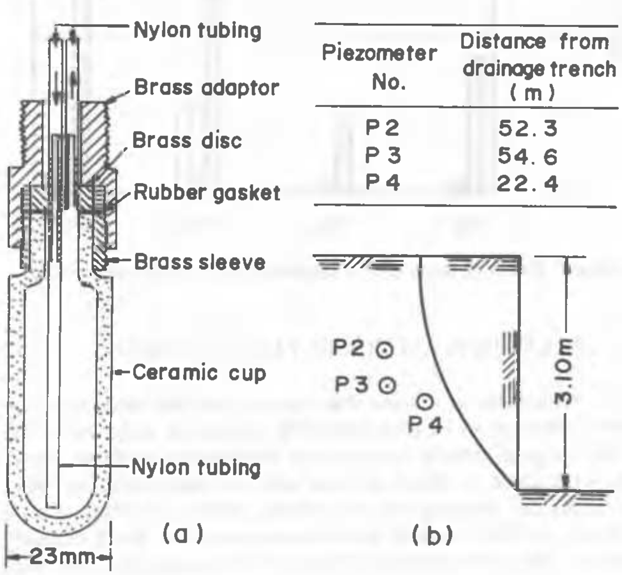

Figure 1. (a) Piezometer tip used; (b) Position of piezometers installed

Table 2 Pore water pressures recorded at various times after excavation

Piezometer No.	Depth (m)	u_w before & several days after excavation (kPa)			
		Before	1 day	4 days	8 days
P2	1.31	-55.3	-51.5	-53.4	-52.9
P3	1.76	-20.2	-19.8	-21.0	-24.6
P4	1.95	-20.2	-19.8	-20.2	-20.8

Cavitation occurred in the lines for P2; the corresponding suction values are an underestimate. At P3 and P4, practically no change in u_w occurred. This is in accordance with Skempton's (1954) equation correlating pore water pressure change with principal stress changes: low values (e.g. Table 1, row 9) of B, which also enters the expression for deviator stress change as a multiplier, result in an overall small change in u_w.

So it is not surprising that the F_s values based on iswests, in which the order of normal stresses are close to those in a critical slope of the soil being tested (Mirata, 1974), are much closer to the true value of 1.00 than those obtained from effective stress analyses based on tests on small specimens (Fig. 2); this figure shows the comparisons presented earlier (Mirata 1980) for the three slips, plus that for a fourth slip (Mirata 1976), 34 m from, and on the opposite side of the same excavation as for Slip 1, analysed using the iswest data for slip 1, despite the differences in the index properties (Table 1, rows 3-7) and the orientation of the failure planes. It seems possible that effective stress analyses based on triaxial tests on larger sized specimens might give estimates of F_s, closer to unity, but it must be remembered that each of the triaxial tests even on 102 mm diameter specimens took as long as 168 hours to complete in order to ensure 95 % equalization of pore pressures at failure, whereas an iswest can be completed in 4 to 6 hours depending on the stiffness of the soil: Cascini (1992) reports testing times of 2.5 hours.

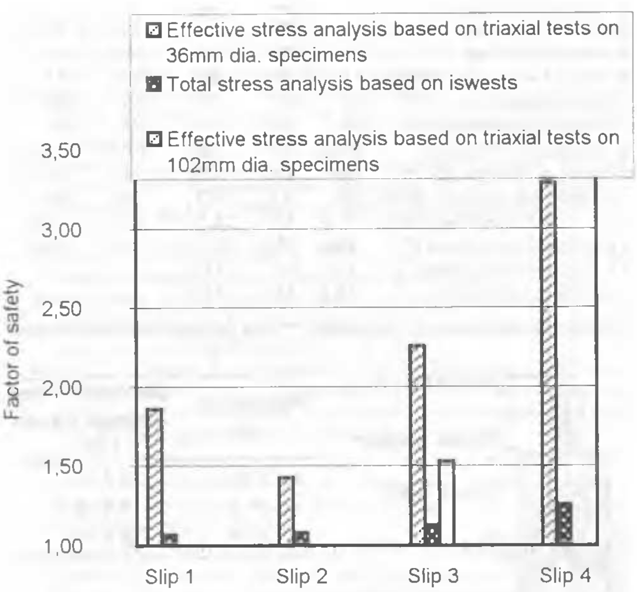

Figure 2. Factor of safety of four slips estimated by different methods

3 DEDUCTIONS AND RECENT DEVELOPMENTS

It is reasonable to assume that for such and all drier soils, the pore pressures in an undrained test, conducted under about the same range of normal stresses as in the stability problem, would be very close to those in situ, and so, measuring the shear strength and carrying out the stability analysis in terms of total stresses is likely to yield correct estimates of the factor of safety for the short term stability of slopes. This possibility is of particular value in fissured and/or stony clays, which require a larger area to be tested than in the usual laboratory tests. Another implication of the low response of pore pressures to stress changes in unsaturated clays is that, because the contribution of pore pressures to the effective stresses and hence the shear strength will be practically constant under the normal stresses at which such response is low, the angle of friction in terms of total stress will be very close to, but somewhat lower than, the angle of friction in terms of effective stress, and so, by neglecting the effective cohesion, can be used for conservative estimates of the factor of safety of slopes in fissured and jointed clays, where surface water infiltrating into the cracks and fissures may exert full hydrostatic pressures in such discontinuities, while suctions may continue to exist in intact lumps of the clay (e.g. Esu, 1966).

The iswest has in recent years been adapted to be performed using a portable frame, either on undisturbed or compacted cylindrical specimens of clay or sand with particles finer than 10 mm, or on prismatic specimens of gravel or clay with particles up to 40 mm (Mirata 1991, 1992). Recent applications and comparisons with the results of established tests are presented by Mirata et al. (1998, 1999) and Tosun et al. (1999). Full details of all three versions of the wedge shear test, including the working drawings of the special devices, instructions for the use of two appended computer programs for the detailed evaluation of the tests, and examples of calculator programs for the simplified evaluation, are given in a recent manual (Mirata 2000).

4 CONCLUSIONS

In stiff fissured unsaturated clays, the advantage of testing a larger area seems to grossly outweigh any possible effects of a misrepresentation of the contribution of suctions to the shear strength. The ideal is to be able to carry out such analyses in terms of effective stresses. If this proves to be impractical or expensive, the use of undrained tests and a total stress type of analysis seems to be a good compromise. The wedge shear test is a practical means of implementing the latter approach.

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