# CORRESPONDENCE

The Secretary,
The Institution of Civil Engineers.

DEAR SIR,

The Paper, "Fundamental shear-strength properties of the Lilla Edet Clay" by Bjerrum and Wu (*Géotechnique*, 10 : 3 : 101), brings to our attention that there are at present in use several methods of determining the fundamental shear-strength parameters, true cohesion $c_e$, and true angle of internal friction $\phi_e$. Because these methods do not necessarily give the same results, it is considered important that these procedures and their implications be reviewed.

The following list shows methods which have been generally used or advocated in the past for determining $c_e$ and $\phi_e$:

| *Reference* | *Test type* | *Failure criterion* |
|---|---|---|
| Hvorslev, 1937 | Drained | $(\sigma_1 - \sigma_3)$ max and $(\sigma_1'/\sigma_3')$ max give the same results. |
| Gibson, 1953 | Drained | |
| Bjerrum, 1954 | Drained | |
| Bishop and Henkel, 1957 | Undrained | $(\sigma_1 - \sigma_3)$ max |
| Simons, 1960 | Undrained | $(\sigma_1'/\sigma_3')$ max |
| Bjerrum and Wu, 1960 | Undrained | $(\sigma_1 - \sigma_3)$ max |
| Kenney, 1960 | Undrained | $(\sigma_1'/\sigma_3')$ max |

It will be seen that there are two different test procedures, namely, the drained test and the consolidated-undrained test with pore-pressure measurements, and there are two failure criteria used, namely $(\sigma_1 - \sigma_3)$ max and $(\sigma_1'/\sigma_3')$ max.

It has been shown (Kenney, 1959; Bjerrum and Simons, 1960) that the shear-strength parameters mobilized at failure in a drained test are very similar to those mobilized in an undrained test at the condition $(\sigma_1'/\sigma_3')$ max, but are not necessarily similar to those mobilized in an undrained test at the condition $(\sigma_1 - \sigma_3)$ max. In a recent study of the mobilization characteristics of $c_e$ and $\phi_e$ (Schmertmann and Osterberg, 1960), it has been shown that the mobilization-strain characteristics of $c_e$ and $\phi_e$ are not similar but that for isotropically consolidated samples, $c_e$ reaches a maximum value at much smaller strains than does $\phi_e$. These observations are illustrated in Fig. 1. If it is accepted that the values of $c_e$ and $\phi_e$ which are mobilized at failure in a drained test are similar to those mobilized in an undrained test at $(\sigma_1'/\sigma_3')$ max, it follows that in the case of soils which fail at very low strains, the values of $c_e$ and $\phi_e$ which are mobilized in an undrained test at $(\sigma_1 - \sigma_3)$ max may differ appreciably from those mobilized at $(\sigma_1'/\sigma_3')$ max. There is little doubt that the study of the mobilized parameters at the condition of $(\sigma_1 - \sigma_3)$ max in undrained tests is of much interest and importance. However, the Writer believes that the parameters measured at this condition are *not* fundamental properties of the soil but are only partially mobilized values of the true fundamental parameters $c_{\max}$, $c_e$, and $\phi_e$.

For their Paper, Messrs Bjerrum and Wu determined the mobilized value of $c_e$ at the condition of $(\sigma_1 - \sigma_3)$ max in undrained tests by using a relationship such as the following:

$$ce_m = \frac{\frac{1}{2}(\sigma_1 - \sigma_3)(1 - \sin \phi e_m) - \sigma_3' . \sin \phi e_m}{\cos \phi e_m}$$

where $ce_m$ and $\phi e_m$ denote the mobilized values. The Authors stated that a value of $\phi e_m = 20^\circ$ was used in the calculations, but it follows from the above discussion that if the fully mobilized

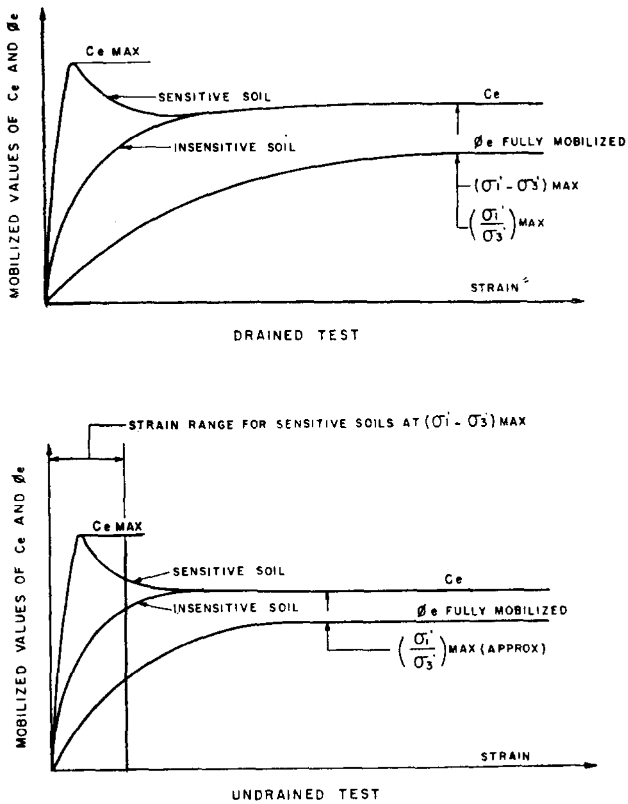


**Fig. 1. Mobilization of $c_e$ and $\phi_e$**

value of $\phi e$ is equal to 20° a somewhat lower value should have been used in the calculations to study conditions at $(\sigma_1 - \sigma_3)$ max in order to account for incomplete mobilization of $\phi_e$.

It would be appreciated if the Authors would comment on this point and also if they would express their views concerning the definitions and the methods of measuring the fundamental shear strength parameters of $c_e$ and $\phi_e$.

Yours faithfully,
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1 December, 1960.

REFERENCES

Bjerrum, L., 1954. "Theoretical and experimental investigations on the shear strength of soils." *Norwegian Geotech. Inst. Publ. No.* 5. 112 pp.

Bjerrum, L., *and* N. E. Simons, 1960. "Comparison of shear strength characteristics of normally consolidated clays." *Paper presented to Conf. on Shear Strength, Boulder, Colorado, U.S.A. Norwegian Geotech. Inst. Publ. No.* 35: 13–22.

Bjerrum, L., *and* T. H. Wu, 1960. "Fundamental shear strength properties of the Lilla Edet clay." *Géotechnique*, 10 : 3 : 101–109.

Bishop, A. W., *and* D. J. Henkel, 1957. "The measurement of soil properties in the triaxial test." *Arnold, Lond.*

Gibson, R. E., 1953. "Experimental determination of the true cohesion and true angle of internal friction in clays." *Proc. 3rd Int. Conf. Soil Mech.*, 1 : 126–130.

HVORSLEV, M. J., 1937. "Über die Festigkeitseigenschaften gestörter bindinger Böden" ("On the physical properties of disturbed cohesive soils"). *Ingeniørvidenskabelige Skrifter, A*45. 159 pp.

KENNEY, T. C., 1959. *Discussion* on Paper "The influence of rate of strain on effective stresses in sensitive clay" by C. B. Crawford. *ASTM Spec. Tech. Publ. No.* 254 : 49–58.

KENNEY, T. C., 1960. *Discussion* on Paper "The compressive strength of remoulded Niagara clay" by D. J. Bazett and S. W. Smotrych. *ASTM Proc. Ann. Gen. Mtg.*

SCHMERTMANN, J. H., *and* J. O. OSTERBERG, 1960. "An experimental study of the development of cohesion and friction with axial strain in saturated cohesive soils." *Paper presented to Conf. on Shear Strength, Boulder, Colorado, U.S.A.*

SIMONS, N. E., 1960. "Comprehensive investigations of the shear strength of an undisturbed Drammen clay." *Paper presented to Conf. on Shear Strength, Boulder, Colorado, U.S.A. Norwegian Geotech. Inst. Publ. No.* 35: 23–33.

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DEAR SIR,

In their Paper "A study of the consolidation characteristics of a clay" (*Géotechnique*, 10 : 2 : 62), Messrs Newland and Allely make the observation that the slope of the "secondary tail" is independent of a wide range of factors. In particular, it was concluded from tests on Whangamarino clay that this slope is independent of the total void ratio change during a given increment (and therefore of the total pressure and pressure increment).

While an *apparent* constancy in the slope of the secondary tail is occasionally observed in other clays, extensive series of consolidation tests on a wide range of soil types (Ramiah, 1959; Girault, 1960; Altschaeffl, 1960) have demonstrated that this is not generally the case. Results from a series of tests on undisturbed samples of Mexico City clay are shown in Fig. 1(a); similar data for an artificially sedimented limestone residual clay are presented in Fig. 1(b). It is evident that the slope of the secondary tail is not even approximately constant. Furthermore, there exist systematic relationships between the rate of secondary compression, the total pressure, and the pressure increment, as illustrated in Fig. 2 ($R_s$ is the secondary compression per cycle of log time).

The coefficient of consolidation ($C_v$) is much more sensitive to test variables than the compression index (Leonards and Ramiah, 1959). In fact, the applicability of the Terzaghi theory can be rigorously examined only if the rate of pore-pressure dissipation is measured and the effects of side friction are either eliminated or accounted for. Investigations of this nature (including studies on the mechanism of secondary compression) are being reported elsewhere (Leonards and Girault, 1961).

Messrs Newland and Allely recommend the use of large-pressure increment ratios to accelerate consolidation testing. While this procedure has little effect on the straight-line portion of the $e - \log p$ curve, it does obscure the determination of the effective preconsolidation pressure. On the basis of laboratory tests, it has been suggested (Leonards and Ramiah, 1959) that a normally consolidated clay will not reduce in volume until a load increment of sufficient magnitude has been applied to break the bonds developed by long-term secondary compressions and thixotropic action (the effective or quasi-preconsolidation pressure). Tests on artificially sedimented clays (Altschaeffl, 1960) and field evidence (Hansbo, 1960) are corroborating this conclusion. The quasi-preconsolidation pressure can be determined from laboratory test data provided small pressure increments are used. In many cases, particularly for deep and/or highly compressible clay deposits, this factor has an important bearing on the accuracy of the settlement forecast. For Mexico City clay, where the use of net pressures only slightly in excess of the quasi-preconsolidation pressure ($p_c$) would result in catastrophic settlements,