Recording and interpreting soil macrofabric data

A. McGOWN, A. MARSLAND, A. M. RADWAN and A. W. A. GABR (1980) *Géotechnique* **30**, No. 4, 417–447; *Géotechnique* **31**, No. 4, 577–580

Mr R. Harris, Harris & Sutherland (Far East)

I would like to clarify a matter regarding the spatial arrangement of joints referred to by the Authors in reply to Mr Koo's comments raised in discussion.

I determined the spatial arrangement of discontinuities within the *in situ* weathered soil by the mapping of joints using a discontinuity compass and my own detailed geological description. The data were gathered from a trial pit programme consisting of widely spaced, elongated trenches which were further excavated, where required, to ensure that orientation of the trial pits was an independent variable.

All the data were constantly checked to prove their validity both in terms of information within and between the trial pits. Nevertheless, few data were excluded from the overall analysis and the joint régime of the soil was believed to have been mapped accurately.

The data for the stereographic and spacing analysis were gathered from an excavated slope, although in some cases the slope was within 2 m of

the original ground surface. Evidence of stress relief processes was seen by the infrequent occurrence of persistent, clean discontinuities, whereas most of the relict joints were stained and coated with a relatively hard oxide deposit.

Stochastic methods based on inadequate data must be open to question, as are deterministic models relating to ill-conceived assumptions. However, in this case the discontinuity data within the soil were accurately measured with an eye to further analysis and it was believed that probability was the best solution to the analysis of these heterogeneous and troublesome soils.

Authors' reply

The clarification of the techniques used by Mr Koo to obtain his data on joint patterns is most helpful. It serves to illustrate the necessity of carefully reporting the joint measurement techniques adopted and the problems of separating out discontinuities induced by the investigation technique and those inherent to the soil or rock mass studied.

Self-weight consolidation of soft soils: an experimental and theoretical study

K. BEEN and G. C. SILLS (1981). *Géotechnique* **31**, No. 4, 519–535

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The Authors have extended the work of Gibson, England & Hussey (1967), taking into account the self-weight of the soil and large strains. Several workers, including Koppula (1970) and Gibson *et al.* (1981), have formulated equations governing the consolidation of a sedimenting soil, that consider finite strains and self-weight of the soil.

Although closed-form analytical solutions for the various cases reported in the Paper are useful,

it seems that the numerous simplifying assumptions made are at best idealistic. The assumption of constancy of the coefficient of consolidation c_F in a sedimenting soil fails to recognize the dependency of c_F on depth as well as on time; typically c_F decreases with depth and with time. To make c_F a constant, linear relationships between void ratio–effective stress and void ratio–permeability were assumed. These assumptions are too restrictive and do not necessarily reflect the behaviour of a sedimenting soil.

Koppula (1970) used the power laws

$$de/d\sigma' \propto (\sigma')^p \quad (1)$$

$$k/(1+e) \propto (\sigma')^q \quad (2)$$

to describe the behaviour of a sedimenting soil. The value of the exponential p in equation (1) may range from -1 to zero. When $p = -1$, $de/d\sigma'$ varies inversely as the effective stress and a plot of e against $\log \sigma'$ yields a straight line, thus representing a normally consolidated material. A value of zero for p makes $de/d\sigma'$ a constant, denoting the behaviour of a linearly elastic material and the behaviour of an overconsolidated soil in a limited stress range. Thus the limits of -1 to zero on p represent a soil spectrum at one end of which lie normally consolidated deposits and at the other end of which lie overconsolidated soils. Equation (2) shows that a change in $k/(1+e)$ for a given change in e approaches zero as σ' becomes large (i.e. as e becomes small) and that the rate at which $k/(1+e)$ changes decreases continuously. A value for q may be obtained from laboratory tests and/or field measurements.

Lee & Sills (1981) took permeability to be proportional to $(1+e)$, i.e. $k/(1+e)$ was considered to be constant with $q = 0$ in equation (2). This is clearly inappropriate in a sedimenting soil where the coefficient of permeability decreases with e and depth.

Based on the power law relationships a generalized one-dimensional finite strain consolidation theory was developed by Koppula (1970) in a form suitable for evaluating the progress of consolidation in deltaic sediments off the coast of Louisiana. The rate of deposition is believed to be the main contributing factor most closely associated with the observed consolidation characteristics (Koppula & Morgenstern, 1982). The Authors assumed instant deposition and that the deposit is uniform in terms of void ratio. These relationships are rarely found in practice where sedimentation involves a moving boundary and sediment thickness is decreasing.

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Authors' reply

There are two alternative approaches to the modelling of soil behaviour. One possibility is to simplify the governing equation by the maximum acceptable idealization, as described in the Paper. The disadvantage of this approach is that the necessary assumptions are frequently unrealistic, as in the present case. The second approach is the one put forward by Dr Koppula, in which care is taken to model the soil behaviour (e.g. stress-strain-permeability relationships) by fitting an appropriate curve. The resulting governing equation is always more complex and often requires substantial computer effort for its solution.

Each approach has certain advantages. The analytical method leads more easily to a solution. The restrictive assumptions can often be improved by solving the simplified equation using a numerical technique which allows different values of the soil parameters to be input at different stages of the consolidation process. In this way, a complex relationship can be modelled as accurately as is required. However, in many field situations, the soil parameters are not known in detail and measurement may yield only approximate values. In such cases, a complicated equation would not be justified. In some circumstances, the analytical solution may appear to be more realistic than the individual idealizations would indicate (perhaps because effects tend to cancel each other out) and it may then be acceptable to use this analytical solution to predict behaviour of the soil in similar circumstances. This seems to be the case with the present results.

The numerical approach should produce a more realistic governing equation with a wide general application. However, the solution of the equation often requires numerical manipulation that can introduce convergence problems and errors.

Whichever approach is chosen, it is important that the idealizations are examined carefully. Some assumptions fundamentally affect the nature of the governing equation, so that, for example, a decision to restrict strain magnitudes prohibits the application of the model to situations of large strain. Similarly, if body forces such as gravity are ignored, self-weight loading cannot be considered. The significance of the work presented by Gibson, England & Hussey (1967) for sedimentation and self-weight consolidation lies in

the acceptance of finite strain and the inclusion of self-weight.

The extension of any model to an environment different from that in which it has been tested should be undertaken with great care. In this case, a simple model has provided good correlation with accurate laboratory measurements, but an extension to field conditions is likely to require greater flexibility with regard to the soil parameters. This

could be achieved with either approach.

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In situ measurement of horizontal stress in overconsolidated clay using push-in spade-shaped pressure cells

P. TEDD and J. A. CHARLES (1981). *Géotechnique* 31, No. 4, 554–558

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The Authors present in situ measurements of horizontal stress at a site in Essex made using push-in pressure cells and the Camkometer. We have been engaged in a laboratory study of the soil properties at this site. In particular, careful measurements of the capillary pressure p_k' have been made which can be used to determine the horizontal stress using the method suggested by Skempton (1961).

The samples were 105 mm in diameter and 600 mm long and were obtained by thin-walled push sampling. They were stored in the steel tubes with waxed ends. Each sample was trimmed to 102 mm in diameter and 204 mm in height. Pore pressure was measured using a high air entry porous disc and a transducer. Contact between sample and porous disc should be avoided during setting up and this was achieved by standing the sample on two semi-circles of fuse wire (0.864 mm in diameter). The small amount of air trapped by this procedure would be dissolved under the high cell pressures that were used. During the setting up period the pore pressure in the measuring system

was monitored and no significant drop in pressure was observed. Filter paper drains were not used as it has been found that these substantially affect initial sample suction.

After mounting and jacketing the sample the cell was rapidly filled with water and a cell pressure of 800 kN/m² applied. On application of the cell pressure the fuse wire was forced into the sample base and the sample came into contact with the porous disc and the pore pressure measuring system. The sample was then left for about two days so that any pore pressure gradients within the sample could equalize. A small increment of cell pressure was then applied in order to check the value of the pore pressure parameter B . The difference between the measured pore pressure and the applied cell pressure was taken as the initial suction p_k' of the sample.

The results of four tests from the Essex site are given in Table 1. The values of K_0 have been derived from the expression

$$K_0 = \frac{p_k'/\sigma_v' - A_s}{1 - A_s}$$

where A_s has been taken as $\frac{1}{3}$ and $\frac{1}{2}$. A value of

Table 1. Measured values of capillary pressure p_k' and associated values of K_0 and σ_H

Depth: m	Soil type*	Overburden pressure σ_v : kN/m ²	Pore pressure u : kN/m ²	p_k' : kN/m ²	B	$A = \frac{1}{3}$		$A = \frac{1}{2}$	
						K_0	σ_H : kN/m ²	K_0	σ_H : kN/m ²
3	CB	60	0	65	0.972	1.12	67	1.16	69
8.4	LC	168	52	155	0.975	1.51	227	1.68	247
15.1	LC	302	117	231	0.991	1.35	370	1.47	391
19.7	LC	394	162	347	0.971	1.75	567	1.99	624

* CB Claygate beds, LC London clay.