For permanently submerged organic soils in south Louisiana, it was found that the specific gravity, pH, and natural moisture content could be used in a given small area as preliminary indicators of organic content and compressive strength (Figs 2–4).

Because the Writer's samples contained fibrous organic material with carbon-14 ages ranging from 370 to 2180 years (Table 1), he could hardly expect to have total agreement with the specific characteristics of soils used by the Authors. However, it is refreshing to find that the Authors have undertaken the investigation of a much neglected subject, the properties and mechanics of organic soils.

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EXPERIMENTAL AND THEORETICAL INVESTIGATIONS OF A PASSIVE EARTH PRESSURE PROBLEM

(JAMES, R. G. & BRANSBY, P. L. (1970). *Géotechnique* **20**, No. 1, 17–37; Discussion (1971) **21**, No. 2, 173–175)

R. G. James and P. L. Bransby

Dr Meyerhof suggests that earlier passive earth pressure investigations on sand (Rowe and Peaker, 1965) and observations of the inclinations of rupture surfaces in triaxial compression tests on sand indicate that the observed rupture surface directions agree fairly well with the predictions based on the average Coulomb friction angle on the rupture surfaces.

Unfortunately, conventional triaxial tests with rough end platens are likely to produce complex stress and strain distributions within the sample, even at small strains, and therefore any rupture surface observed on the boundaries of the sample is likely to be a poor reflection of the curved rupture surfaces within the sample, especially as these curved rupture surfaces may well have different inclinations to the vertical at different points within the sample. Furthermore, accurate measurement of the apparent inclination of a rupture surface can only be made with confidence after extreme deformations of the soil sample, and it is in this state that there is least knowledge about the local principal stress directions.

In contrast, plane strain tests using lubricated end platens are likely to provide more reliable data of rupture plane inclinations. King and Dickin (1970) observed that the rupture surface formed in a typical plane strain test (PB6) on dense sand was inclined at 36° to the major principal stress direction. The corresponding observed angle of dilation, ν, was 14°, and thus the zero extension direction was at $(45°-\nu/2)=38°$ to the major principal stress direction, in good agreement with the observed rupture plane. The observed peak angle of friction $\phi=45°$ indicates that the stress characteristics were inclined at $(45°-\phi/2)=22\frac{1}{2}°$ to the major principal stress direction.

The observations of rupture surfaces by Rowe and Peaker (1965) in their passive earth pressure tests on sand do not entirely support the concept that rupture planes form along stress characteristics. For example, Rowe and Peaker state on page 76 of their paper, when discussing data from tests on loose sand, 'It was found repeatedly in the region near the sand surface remote from the wall where Rankine passive states are normally assumed to occur, that the slopes of the slip paths were much steeper than for the case of $\phi'_{\max}=34°$. The slopes conformed more with $\phi'_m=10°$.' The rupture surfaces must therefore have emerged at 40° to the horizontal, which corresponds to $\nu=10°$, a reasonable value for loose sand at large strains. The stress characteristics would be at $(45°-\phi_m/2)=28°$ to the horizontal.

Some of the difficulties of comparing theoretical and observed rupture surfaces are illustrated by the complicated pattern of ruptures which is observed, both when the wall rotates about its toe (James and Bransby, 1970), and when the wall translates (Roscoe, 1970). These data indicate that the formation of the rupture surfaces is a progressive phenomenon and is especially complex when multiple rupture surfaces form. Furthermore, there is no guarantee that the additional surfaces which are activated at large wall movements are the same shape or in the same location as those which are relevant at peak load on the wall. A further difficulty in the comparison is that the inclinations of the rupture surfaces at the wall will be governed by the local angle of wall friction. Although it is usually assumed that the local angle of wall friction is constant down the wall and equal to the overall angle of wall friction, data quoted in the Paper and in earlier work of James (1965) indicate that there is a considerable variation of the angle of wall friction down the wall for two modes of wall rotation.

Further experimental evidence for the observation that rupture surfaces outcrop at $(45°-\nu/2)$ to the horizontal is given by Hansen (1953), Balakrishnamurthy (1968) and the photographs of Narain *et al.* (1969); but see Bransby (1970).

Meyerhof remarks upon the success of methods of analysis based on stress characteristics and the average Coulomb shearing strength. Surely the important point is that these methods are only successful if the correct stress boundary conditions and the correct shearing strength are used in the analysis? The displacement boundary conditions of a specific problem play a vital role in fixing the relevant stress boundary conditions (e.g. the angle of wall friction in passive earth pressure problems) as well as in controlling the variation through the soil of the mobilized shearing strength. Of course, once a prediction has been made of the stress boundary conditions and of an average value of ϕ_m, the conventional methods of analysis should give realistic predictions of peak load, but the prediction of the complete load–displacement behaviour requires a detailed understanding of the pattern of soil deformation.

The Authors would like to thank Professor Lee for his wholehearted support of the major hypotheses presented in their two Papers (James and Bransby, 1970, 1971). However, two minor points require comment. The Authors consider that the interpretation of results obtained by upper and lower bound calculations requires caution. For, although it may be possible to obtain unique solutions satisfying both upper and lower bound types of calculation, these solutions are only relevant to the idealized, perfectly plastic material. Quite apart from theoretical arguments about the relevance of these solutions to soils, the practical fact remains that for certain problems (e.g. passive earth pressure problems) gross deformations are required to mobilize the so-called lower bound collapse loads. These deformations are so large that for many engineering purposes failure could be said to have occurred at loads well below the lower bound load.

Finally, it is appropriate to mention that the concept of non-coincidence of 'shearing planes' (stress characteristics) and 'slip lines' (velocity characteristics) is a concept of the theory of plasticity relevant to materials with a non-associated flow rule (Davis, 1968). The precise directions of particle movement are, of course, determined by the directions of the slip lines and the velocities along them; these velocities are in turn dependent upon the velocity boundary conditions. It is of interest to note that although the stress dilatancy theory gives unsatisfactory predictions of rupture surface inclinations (King and Dicken, 1970) it is, however, successful in predicting the observed relationships between stress ratio (R) and the dilatancy factor (D) for cohesionless materials, when suitable values of ϕ_f are inserted into the theoretical relationships.

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THE CORRELATION OF CONE SIZE IN THE DYNAMIC CONE PENETRATION TEST WITH THE STANDARD PENETRATION TEST

(MOHAN, D., AGGARWAL, V. S. & TOLIA, D. S. (1970). *Géotechnique* **20**, No. 3, 315–319; Discussion (1971) **21**, No. 2, 184–190)

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It is noticed that Professor Meardi, in his technique for dynamic cone penetration tests, uses a 48 mm dia. casing in which a 51 mm dia. cone with 34 mm dia. shaft is made to slide. The Authors are not surprised that the Writer is getting very erratic behaviour in clayey and silty soils since their own experience is similar. It is quite reasonable to expect a good correlation of the SPT with a 51 mm dia. cone in sandy soils *provided a casing is used* which would eliminate the side friction on the shaft carrying the cone.

Professor Meardi has missed the main point in the Paper, which mentions that *no casing* is used with the cone. In this case the side friction on the shaft becomes important and this would increase with depth. However, up to 9 m depth the Authors obtain a fair relationship with SPT. For design of shallow foundations the normal depth of exploration does not normally exceed 9 m. The Authors have also found that with the standard A rods (41·5 mm dia.), out of all the cone series varying between 43·75 and 75 mm dia., the cone of 62·5 mm dia. gave the most consistent results. It should also be conceded that driving a cone without casing is simpler and more economical than driving one with casing.

For soil exploration with the dynamic cone to a greater depth, a technique where friction on the shaft is eliminated by circulating the bentonite slurry has already been suggested (Mohan and Sen Gupta, 1970). The Authors have found that this technique is also simpler than using a casing pipe for eliminating side friction.

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THE DYNAMIC PENETRATION TEST: A STANDARD THAT IS NOT STANDARDIZED

(IRELAND, H. O., MORETTO, O. & VARGAS, M. (1970). *Géotechnique* **20**, No. 2, 185–192; Discussion **20**, No. 4, 452–456 and (1971) **21**, No. 2, 183–184)

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Mr Serota discusses the current British methods of carrying out the so-called standard penetration test. The Writers would agree with him that the practices adopted in Britain for many years, involving the use of casing having a minimum diameter of 6 in. and bailers or shells in drilling through granular soils, depart from the American practice of carrying out site investigation work, in which the SPT has its origins.