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**DISCUSSION**

# Interpretation of moduli from self-boring pressuremeter tests in sand

R. BELLOTTI, V. GHIONNA, M. JAMIOLOWSKI, P. K. ROBERTSON and R. W. PETERSON (1989). *Géotechnique* **39**, No. 2, 269–292

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One of the most important recent developments in soil mechanics has been the realization that the stress–strain response of soil is non-linear, very high stiffnesses being exhibited at low strain levels. Sophisticated local strain instrumentation has been developed to measure this effect accurately in laboratory triaxial tests (e.g. Brown *et al.*, 1980; Jardine *et al.*, 1984) and such instruments are now used in many commercial laboratories.

Current thinking concerning the pressuremeter also places a proper emphasis on the low-strain behaviour of soils. The Authors are rightly concerned that low-strain stiffnesses should be accurately measured and they present a detailed procedure for processing raw pressuremeter data. Following Hughes (1982) and Wroth (1982) the Authors adopt the technique of performing small unload–reload cycles. As they point out in their conclusions, however, this technique should be used with caution. They also recommended that 'considerable attention to probe design is required for reliable strain measurements'.

For materials such as sands which cannot be sampled without serious disturbance, instruments such as the in situ self-boring pressuremeter offer, *in principle*, an important technique for measuring horizontal stiffnesses. However, there is some concern about the accuracy of the low-strain stiffnesses measured by the pressuremeter. Until local-strain techniques were developed, a similar lack of confidence also existed in relation to small-strain moduli determined in laboratory element tests (Burland, 1987). Do the lessons of the laboratory not suggest that an attempt should be made to develop local-strain instrumentation for the pressuremeter? Fig. 21 shows a possible conceptual design. A prototype would, no doubt require a good deal of mechanical ingenuity, but it should be by no means impossible. In Fig. 21 the deformation measuring devices emerge radially from the instrument housing half way up the pressuremeter body, and are located in the surrounding soil using mini-augers, or other devices depending on the soil type. The relative displacement of the diametrally opposed auger heads may be monitored by LVDTs or other high resolution devices.

The essential principle is that diametral deformations are measured by two independent devices, each measuring across different diameters, $D_1$ and $D_2$ so that an annulus of soil between $D_1$ and $D_2$ becomes the test element. Some of the possible advantages are

(*a*) measurement points are intimately connected with the soil so that no corrections are required for membrane stiffness
(*b*) a fairly thin annulus of soil is tested so that the use of a single value of strain becomes more meaningful than hitherto
(*c*) the soil under test is removed from the pressuremeter so that the installation disturbance has less effect than using the currently-used arrangement; while some additional disturbance is occasioned by the installation of the auger-heads this is likely to be fairly localized and the behaviour of the overall test element will dominate deformation readings
(*d*) a full stress–strain plot may be produced unlike the current arrangement where the first part of the curve must generally be discarded because of installation disturbance effects.

There are, of course disadvantages associated with such an arrangement both practical (additional time, cost, inconvenience and so on) and conceptual (including stress discontinuity around the instrument housing and the use of a single set of readings at discrete points which may not be representative). However, on balance, such a system may offer substantial advantages over the current arrangement. In the light of the observations made in the Paper, what are the Authors' views on the suggested adaptation?

**Authors' reply**

The Authors agree with Mr O'Reilly's comments, and specifically that

(*a*) in sands, the shear modulus as obtained from a small unload–reload loop represents a measure of horizontal soil stiffness of practical interest for designers

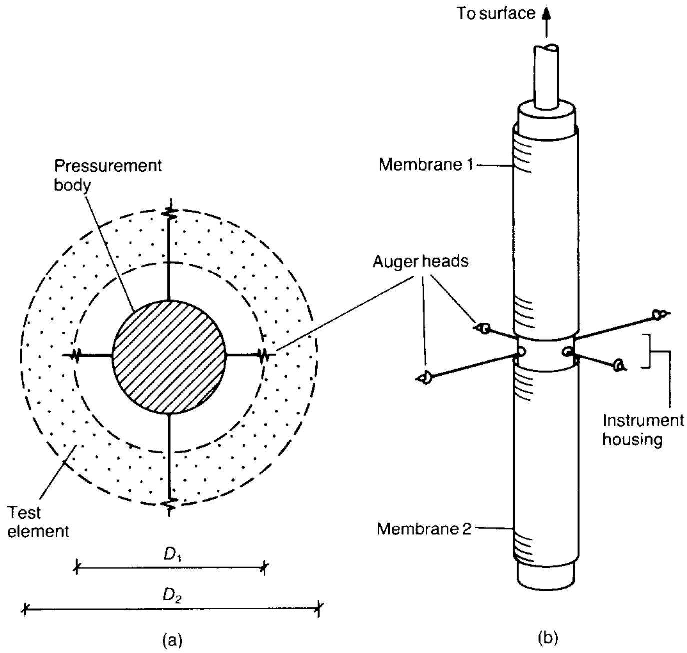


**Fig. 21. Modified pressuremeter with instrument augers extended: (a) plan; (b) isometric view**

(*b*) the reliability of the measured stiffness might be hampered by the compliance of the strain measuring system of the SBP probe.

The latter problem, especially important in very stiff soils ($G > 50$ MPa) has been recognized by many researchers during an international symposium on pressuremeters held at Oxford University (e.g. Fahey & Jewell, 1990; Campanella *et al.*, 1990). Therefore, one can hope that both manufacturers and users of the SBP will make every effort in the future to improve the accuracy of the strain measurement system in order to increase the reliability of the measured lift-off pressure and unload–reload shear stiffness. O'Reilly's suggestion of measuring the local strains around the expanding pressuremeter probe internally is certainly stimulating but may not be practical at the present stage of development of the use of the SBP test. In fact this test, as far as execution and interpretation are concerned, still appears to be too complex to encourage its use by practitioners. In this circumstance we believe that it is necessary to promote the use of the SBP test in design practice before introducing further sophisticated modification of the test equipment and procedures.

Among the different possible applications of the SBP test in everyday design practice, that related to the use of the elastic unload–reload stiffness to compute the displacements of structures found in sands is one of major interest. In order to achieve this it is necessary to consider

**Table 6. Elastic anisotropy of pluvially deposited dry Ticino sand**

| $D_R$: % | $\frac{\sigma'_v}{\sigma'_h}$ | $\frac{G_{HH}}{G_{VH}}$ | $\frac{M_H}{M_V}$ |
|---|---|---|---|
| 40 | 0·7 | 1·00 | 1·37 |
| | 1·0 | 1·15 | 1·15 |
| | 2·0 | 1·30 | 0·95 |
| | 3·0 | 1·27 | 0·79 |
| 90 | 0·7 | 1·00 | 1·23 |
| | 1·0 | 1·10 | 1·29 |
| | 2·0 | 1·33 | 1·18 |

From the measured shear wave velocities:
$G_{HH}$ shear modulus in the horizontal plane
$G_{VH}$ shear modulus in the vertical plane.
From the measured compression wave velocities:
$M_H$ constrained modulus in the horizontal plane
$M_V$ constrained modulus in the vertical plane.
$\sigma'_v$, $\sigma'_h$ consolidation stresses applied to the CC specimen.

the following aspects of the problem, which were treated in the Paper but are repeated here in order of importance:

(*a*) the non-linearity of the soil stress–strain relationship which renders the $G_{UR}$ dependent on the $\Delta\gamma$ of the loop
(*b*) the hypoelastic nature of the soil which means $G_{UR}$ is influenced by $p'_B$
(*c*) anisotropy of the soil which makes the $G_{UR}$ dependent on the direction in which the soil is loaded: the values of shear moduli obtained from the SBP tests correspond to the horizontal soil stiffness.

With respect to the latter problem a series of recently completed seismic tests performed in the CC yielded indications of small strain ($\gamma < 10^{-4}\%$) elastic anisotropy of the pluvially deposited dry Ticino sand, which is summarized in Table 6. To what extent these indications are applicable to the values of sand stiffness evaluated at higher strains, such as the $G_{UR}$ discussed here, is just a conjectural matter and needs to be clarified by further research.

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