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DISCUSSION

An analysis of pressuremeter holding tests

V. FIORAVANTE, M. JAMIOLKOWSKI and R. LANCELLOTTA (1994). *Géotechnique* **44**, No. 2, 227–238.

M. Fahey and A. Lee Goh.

The idea of using the self-boring pressuremeter (SBP) to measure the coefficient of consolidation (c_h) of soft clays has been around since the late 1970s, but surprisingly little information has emerged in that time about the use of the method in practice, and about the usefulness of the values obtained. The paper is therefore a welcome addition to the literature on the topic.

This contribution to the discussion is made on the basis of work on strain-holding tests (SHT) and pressure-holding tests (PHT) carried out at the University of Western Australia over the past 10 years. The most recent study of these tests is contained in the PhD thesis of Lee Goh (1994). This study involved a comparison between c_h values deduced from pressuremeter SHTs, PHTs, and piezocone dissipation tests.

The pressuremeter used in this work was manufactured by CAMBRIDGE INSITU, with modifications as described in details by Fahey *et al*. (1988). For the purposes of this discussion, the two significant features of the instrument are: first, the effective stress cells of the instrument have been replaced with minature Druck® pore pressure transducers; and, second, the main pressure membrane is protected from damage during drilling using a 'Chinese Lantern' supplied by CAMBRIDGE INSITU.

A non-standard piezocone, 40 mm in diameter, was used in the study. It is capable of simultaneously measuring total horizontal stress and pore pressure at two diametrically-opposite points on the shaft of the piezocone, at either $4D$ or $1{\cdot}5D$ behind the cone shoulder, where D is the diameter of the shaft. Miniature Druch® pore pressure transducers are also used in this instrument. The instrument may be used with a 60° tip or a 13° tip. In all the tests discussed here, the $13°/4D$ configuration was used. The decision to concentrate on this configuration was based on the assumption that a simple cavity expansion model would be more appropriate for this configuration than for a 60° cone, allowing more direct comparison between the piezocone and pressuremeter tests. Nevertheless, some tests were also conducted with the 60° cone, but the results are not as conclusive.

The clearest conclusion from this work is that the piezocone is a much better instrument for determining c_h that the SBP. This probably applies even to the standard piezocone, but it was certainly the case for the non-standard piezocone used in this research. Nevertheless, having high-quality pressuremeter tests from the same site give much more confidence in the interpretation of the piezocone tests, even if only standard expansion tests are carried out (i.e. even without holding tests).

The site at which most of the work was carried out is located on a meander peninsula (the Burswood Peninsula) of the Swan River in Perth, Western Australia. Below a sandy crust of some 3·5 m thickness, the soil consists of CH silty clay (w_l in the range 100–125; I_p in the range 70–95). Remarkably, the deposit is characterized by an absence of visible sand or silt lenses. Between 4 and 12 m depth, the vane shear strength is sensibly constant at about 22 kPa, and increases gradually below 12 m. The soil appears to be lightly overconsolidated down to about 12 m depth, with the OCR reducing with depth.

PRACTICAL ASPECTS OF HOLDING TESTS

Practical aspects of the instrumentation and test procedure (particularly the de-airing procedure) are crucial for successful pressuremeter holding tests, and perhaps the authors might like to give some more details on some of these aspects.

For any SBP tests in soft clay (and perhaps this also applies to stiffer clay), it is very instructive to monitor the pore pressure at all stages of drilling and testing. Our practice is to start by casing a hole to the top of the water table, and fill this with water. The pore pressure transducer de-airing jig (see Fahey *et al*. 1988) is not removed until the transducers are below the water in the casing. It is found that excess pore pressures are almost always generated during drilling. The end-of-drilling values are shown versus test depth in Fig. 20. In earlier tests, it was found that improper drilling

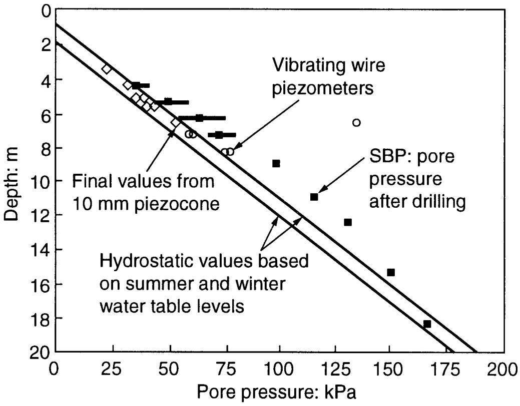

Fig. 20. Pore pressures after drilling in SBP, compared with in situ values

procedures produced significantly greater excess pore pressures, so that we now consider the magnitude of the excess pore pressure to be a measure of the drilling quality.

Ideally, these excess pore pressures should probably be allowed to dissipate before commencing the expansion phase, but this is often impractical due to the time required for dissipation.

One conclusion from the SBP work at this site is that the Chinese Lantern protection sleeve had to be abandoned if meaningful holding tests were to be performed. It appeared that the steel strips were providing a path for reduction of excess pore pressure as measured by the pore pressure transducers. In some cases, the pore pressure would reach a peak during the expansion phase, and then start to reduce even while expansion was still in progress. This type of behaviour was never observed in tests conducted without the Chinese Lantern. There are, of course, practical difficulties in testing in this manner, because of the frequency of rupture of the membrane during drilling in the absence of the Chinese Lantern. Applying a vacuum to the membrane during drilling reduces the frequency of rupture (but this has negative consequences when rupture does occur). It would be interesting to know if the authors observed similar problems with the Chinese Lantern, and, if so, how they managed to overcome these problems.

At the point of 'liftoff' of the membrane in overconsolidated soil, FE analysis using Cam Clay shows that the pore pressure remains constant until the yield surface is reached. This is because no change in mean stress occurs during cavity expansion in a linear elastic soil, and hence Cam Clay predicts no pore pressure increase. This is shown in the vertical stress paths below the yield surface plotted by the authors in Figs 14 and 15. However, Fahey & Carter (1993) showed that for a nonlinear elastic soil, the mean stress increases right from the start of cavity expansion, and hence, even in the absence of shear-induced pore pressures, a nonlinear elastic soil would show pore pressure increase right from the start of expansion. In practice, this is what we always observe (although it is accepted that most of our experience is in lightly-overconsolidated clays). Thus, for OCR > 1, the pore pressure response during initial expansion is always underpredicted by FE modelling with Cam Clay.

A very important practical aspect of the SHT is that it is very difficult to suddenly impose a constant cavity size if gas is used to inflate the instrument, particularly if the cavity is expanded fast enough to ensure that there is no drainage during expansion. In order to maintain an absolutely-constant cavity size, the pressure must be reduced very quickly in the first few seconds of the strain-holding phase. This necessary drop in total pressure is not shown in FE modelling of the test using the Cam Clay soil model. We believe that this is the result of creep (or rather relaxation) effects in the soil. The very rapid reduction in total stress results in an equal reduction in measured pore pressure. Hence, the initial rapid drop in measured pore pressure is not the result of pore pressure dissipation due to radial drainage. Interpretation of the subsequent dissipation phase of the test must take this into account in calculation of $u_{\max}$ and hence in determining the t_{50} value, otherwise the true t_{50} time will be underestimated.

It is not clear from the paper if the authors also experienced this relaxation problem. However, their Fig. 17 shows that the total pressure response in the holding phase of a SHT was poorly modelled by Cam Clay, and this has also been our experience. Similarly, we have found it difficult to model the strain increase which occurs during the constant pressure phase of PHTs, presumably also due to creep effects.

NUMERICAL MODELLING OF HOLDING TESTS

Even given the limitations of FE modelling of holding tests with the Cam Clay model, this type of modelling is very enlightening in interpretation of the tests, as shown by the authors.

In our study, Cam Clay parameters obtained from the soil from the test site were used in numerical modelling of SHT and PHT dissipation tests. In addition, the piezocone tests were modelled by assuming that expanding a cylindrical cavity from 0·1 r_c to r_c was an appropriate way of modelling the piezocone insertion, where r_c is the radius of the shaft of the piezocone.

Since the coefficient of consolidation is not an input parameter in the model, one-dimensional oedometer tests were also modelled to ensure that the coefficient of consolidation deduced from the output was consistent with the input parameters (k, λ, κ, G) and the OCR. In this case, the ratio of c_{voc} to c_{vnc} was equal to λ/κ (i.e. 8·2 in this case). Since isotropic conditions were assumed, the c_v and c_h values are directly comparable.

The dissipation behaviour obtained from the numerical modelling was interpreted using the method of Clarke *et al.* (1979) to give values of c_h. These results are shown in Fig. 21. The values are normalized between 0 and 1 (in a manner analogous to liquidity index) such that

$$c_{\text{normalized}} = \frac{c_h - c_{hnc}}{c_{hoc} - c_{hnc}}$$

Note that in a standard oedometer test, in which the load is doubled for successive increments, the measured value of c_v lies between the normally consolidated and the overconsolidated value for OCR between 1 and 2. For the pressuremeter PHT, the deduced c_h value is very close to c_{hnc} for OCR of 1, and gradually increases with increasing OCR. Note also that the deduced c_h value depends on the strain at the start of the pressure holding phase.

For the pressuremeter SHT, the deduced value of c_h is much less sensitive to OCR in the range considered. While the c_h value depends on the strain at which the tests is carried out, this dependence reduces with increasing strain, so that the difference between values at strains of 10 and 15% is very small. Similarly, there is not very much difference between these values and that obtained from the simulated piezocone test (where the strain is approaching infinity). In the paper, the authors appear to have carried out the SHTs at very low values of strain; we believe that this is not a good idea, as the deduced c_h value is sensitive to the starting strain at these low values of strain.

From these results, a tentative correction to get c_{hnc} and c_{hoc} values from the c_h value deduced from SHT and piezocone tests in clays with OCR up to 2 in might be

$$c_{hnc} \approx 2\frac{\kappa}{\lambda}c_h \quad \text{and} \quad c_{hoc} \approx 2c_h$$

The c_h values from the PHT are more influenced by OCR, and such a simple correction is not possible.

EXPERIMENTAL EVIDENCE

Laboratory piezocone dissipation tests were carried out in samples of kaolin to verify the findings from the numerical work. Unfortunately, it was not possible to carry out laboratory pressuremeter holding tests, because a sufficiently-large calibration chamber was not available.

The kaolin was consolidated from a slurry in a large consolidometer. Removable ports in the piston of the consolidometer allowed a small piezocone (10 mm diameter) to be inserted at various stages of the consolidation. As in the 40 mm diameter cone, the cone was equipped with a 13° point, and the pore pressures were measured at a point 4D back from the cone shoulder.

The c_h values deduced from these tests at OCR of 1 and 2 are shown in Fig. 22. Also shown are the values of c_v deduced from the observed settlement-time behaviour in the consolidometer during virgin loading (OCR = 1) and during unloading at OCR of 2. As in the numerical ‘experiments’, the piezocone values for the normally consolidated soil lie between c_{vnc} and c_{voc} values. At OCR of 2, the piezocone value is still less that the OC value.

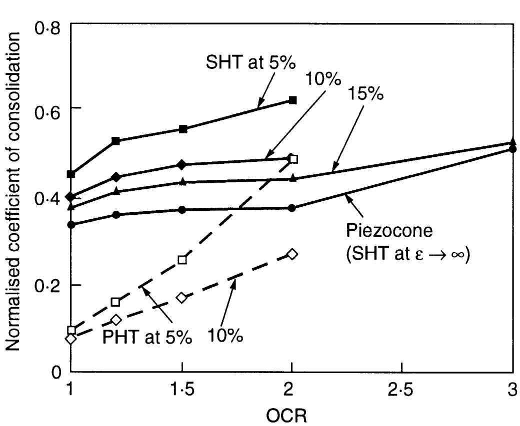

Fig. 21. Normalized coefficient of consolidation from cavity expansion modelling of SHT, PHT and piezocone tests

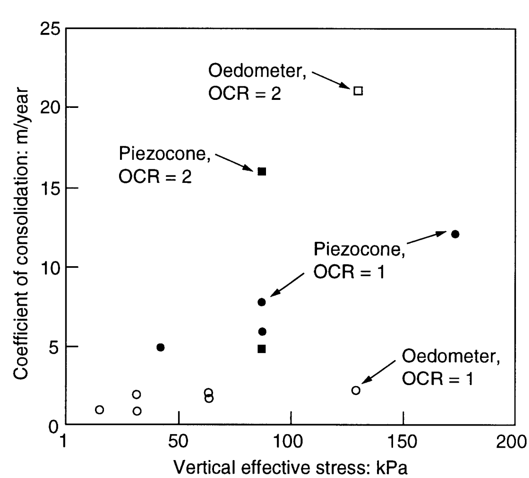

Fig. 22. Coefficient of consolidation from oedometer and piezocone tests on kaoline

Although the soil could still exhibit anisotropic properties, the results tend to confirm the conclusions from the numerical modelling.

A similar comparison was made for the data obtained from SHTs, PHTs, are piezocone tests (using both 40 mm diameter and 10 mm diameter instruments) at the Burswood Peninsula site. The results shown are average values at each depth from each instrument. These are compared to c_v values obtained from laboratory consolidation tests in undisturbed samples from the site. The results of this comparison are shown in Fig. 23. The in situ values are higher than the laboratory values, but this is to be expected, because the permeability of this natural alluvial soils is likely to be anisotropic. Note that if c_h is taken to be 10 times c_v (a common assumption), then the laboratory c_{vnc} and c_{voc} values would bound the in situ c_h values, as expected from the numerical modelling.

From this, we have concluded that the c_h value obtained from a pressuremeter SHT or a piezocone dissipation test can give a very good approximation of the true c_h value of the soil in situ. Although this conclusion is based on tests with a non-standard piezocone with $13°/4D$ configuration, the work with other configurations tend to broadly support it. Without correction, the value will lie between the 'normally consolidated' and 'overconsolidated' values, but a simple correction such as that suggested may be capable of giving a good estimate of both of these values. We would be interested in the authors comments on these conclusions, in light of the data from their tests.

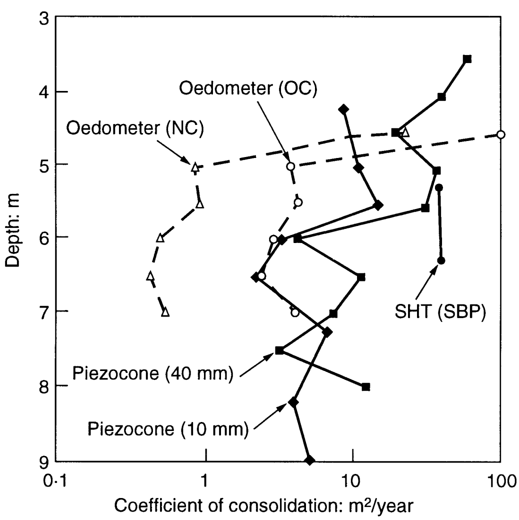

Fig. 23. Average in situ values of *c* from SHT and piezocone (40 mm diameter and 10 mm diameter) compared to *c* values from laboratory tests on undisturbed samples

Author's reply

The writers are grateful for the discussion by M. Fahey and A. Lee Goh. It confirms that the interpretation methods used to obtain c_h from the PHT and SHT are oversimplified. This situation leads, in some circumstances, to an incomplete and unclear understanding of these tests and the related parameters as can be inferred from Fig. 2, presented by the discusser.

The writers attempted in the paper to investigate some of the peculiar aspects of these tests with a numerical tool (CRISP FEM program with Modified Cam Clay model) which gave, in spite of its limitation, valuable insights into the interpretation of holding tests.

In fact after the expansion phase (which in the field might occur in a partially drained condition), in case of PHT the cavity stress is maintained constant, hence the cavity strain increases owing to the consolidation of the surrounding soils. On the other hand, during the SHT, after the expansion phase, the cavity stress is appropriately reduced in order to hold the cavity radius constant. As a result, the dissipation phase of the test occurs generally under conditions of unloading with respect to the state of the effective stress. Consequently, in SHT the calculated c_h reflects the response of an OC soil even if the deposit is close to NC state.

Moreover, the amount of cavity strain attained during the expansion influences both Δu_{max} and the extension of the plastic zone around the probe. Therefore, the value of c_h obtained using the simplified method adopted by the discussers is conditioned by the level of cavity strain reached. If this is the case a reference value of cavity strain should be adopted in order to have the tests which are comparable among them. Anyhow, it is preferable to perform tests at cavity strains as small as possible to reduce the extension of the plastic zone which influences soil permeability.

In the following, the writers attempt to answer the points raised by the discussers.

COMPARISON WITH THE PIEZOCONE

The soil disturbance caused by the insertion of a piezocone is much larger than that linked to the installation of a self-boring probe. Since, the hydraulic conductivity of the ground depends not only on the void ratio but also on the structure of the soil, the induced disturbance influences in a different manner the results of two tests. In addition, it must be recognized that the dissipation around the CPTU; up to 50–60% of the consolidation, occurs in condition of unloading, therefore values of c_h are proper to the over consolidated range.

In view of what is stated above it appears obvious that any possible coincidence between the flow and the consolidation properties obtained from PHT or SHT and those resulting from the CPTU dissipation might depend on the fact that the discussers have used the same method for the interpretation of the both holding and CPTU tests and to the compensation of errors linked to the simplified assumptions made.

PRACTICAL ASPECTS

The procedure adopted at the Fucino site in 1987 for the saturation of the porous stone was the one suggested by Battaglio & Bruzzi (1987). It consisted of the following steps;

(1) cleaning and saturating the filter in laboratory,
(2) assembling the filter on the SBP probe on the site while the probe was immersed in de-aired water and subsequently flushing the cavity with water, using a syringe, and
(3) inserting the probe into the hole with a casing having its shoe well below the static GWL and filled with water.

The Camkometer used at the Fucino site had two differential pore pressure transducers mounted in it which were also used to monitor the pore pressure during insertion as stated by the discussers. This reflects, to some extent, the quality of the self-boring process. At the Fucino site, the Δu experienced during the insertion of the probe was of the order of 20–50 kPa.

In any case, all tests carried out at the Fucino site involved a rest period before commencing the expansion phase, in order to dissipate completely the excess pore pressure caused by drilling. Therefore, the holding tests were not affected by extra excess pore pressures.

The Chinese Lantern was not used at the Fucino site.

From a practical point of view, a precise and constant cavity size (SHT) can be achieved after the expansion phase, by means of a electro-pneumatic servo-controlled system which automatically adjusts the applied pressure nearly instantaneously.

NONLINEARITY OF SOIL BEHAVIOUR

In modelling soil nonlinearity, the Cam Clay model has the limitations shown in Fig. 17, as its use results in an underestimation of the Δu caused by the expansion phase in an overconsolidated media as mentioned by the discussers.

Nevertheless, its correct use leads to valuable indications concerning the interpretation of PHTs and SHTs, as shown in the paper, such that soil nonlinearity significantly influences the interpretation of SHTs (where both applied load and pore pressure are in transient conditions) rather than the PHTs (where only pore pressure changes during the test). The state of strain caused by consolidation is different in the two tests and so the soil deformability is different in the two tests. In the PHT, cavity strain increases (soil voids reduce near the probe, Fig. 16) due to consolidation (soil hardening); in the SHT the increments of cavity strain due to consolidation are compensated by the reduction of applied stress which influences the consolidation: the test is performed under unloading conditions.

The general effect is a faster dissipation but it is not enough to correct the dissipation curve (as shown in Fig. 8 and as suggested by the discussers) in the evaluation of c_h due to the soil stiffness changes.

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