Analysis of self-boring pressuremeter (SBPM) and Marchetti dilatometer (DMT) tests in granite saprolites

F. Schnaid, J.A.R. Ortigao, F.M. Mántaras, R.P. Cunha, and I. MacGregor

Abstract: This paper presents the analyses of the results of the site investigation programme carried out at the Kowloon Bay site in Hong Kong. The tests consisted of self-boring pressuremeter (SBPM), Marchetti dilatometer (DMT), and laboratory tests carried out in a granite saprolite, which can be described as a lightly cemented sand. The purpose of this research project is to stimulate the development of methods to interpret data obtained from tests in residual soils. In particular, the work aims to evaluate the analyses of the SBPM data through a curve-fitting technique. Both the loading and unloading portions of the SBPM curve were analysed and the results compared with those from other tests. The advantage of this analysis technique is the possibility of constructing a theoretical curve that reproduces a pressuremeter test from which a set of fundamental parameters can be derived, namely the friction angle, cohesion intercept, lateral stress, and shear modulus. The DMT proved to be a reliable tool that yielded good soil parameters at a small fraction of the cost of the other in situ tests.

Key words: residual soil, in situ tests, pressuremeter, Marchetti dilatometer.

Résumé : Cet article présente les analyses des résultats du programme de l'étude de site réalisée sur le site de Kowloon Bay à Hong Kong. Le programme comprenait des essais de pressiomètre autoforeur (SBPM), de dilatomètre Marchetti (DMT) et des essais de laboratoire dans des saprolites granitiques qui peuvent être décrits comme un sable légèrement cimenté. Le but de ce projet de recherche est de stimuler le développement de méthodes pour interpréter les données obtenues par des essais dans des sols résiduels. Ce travail vise particulièrement à évaluer les analyses des données du SBPM au moyen d'une technique de lissage de courbes. Les portions de chargement et de déchargement de la courbe du SBPM ont toutes deux été analysées et les résultats comparés aux autres essais. L'avantage de cette technique d'analyse est la possibilité de construire une courbe théorique qui reproduit l'essai pressiométrique de laquelle un ensemble de paramètres fondamentaux peuvent être déduits, nommément l'angle de frottement, l'intersection de cohésion, la contrainte latérale, et le module de cisaillement. Le DMT s'est révélé être un outil fiable qui a donné de bons paramètres de sol, à une faible fraction du coût des autres essais *in situ*.

Mots clés : Sol résiduel, essais in situ, pressiomètre, dilatomètre Marchetti.

[Traduit par la Rédaction]

Introduction

Granite saprolites are a common occurrence in Hong Kong as in many other tropical and subtropical regions. These residual soils are a product of in situ weathering and decomposition of rocks in which fabric and structure are generally inherited. Under highly complex and heterogenic in situ conditions of residual soil formations, selection of appropriate strength and deformation parameters appears to be the most important step in the design of structures such as foundations, slopes, and excavations.

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F. Schnaid and F.M. Mántaras. Federal University of Rio Grande do Sul, Porto Alegre, Brazil.
J.A.R. Ortigao. Federal University of Rio de Janeiro, Rio de Janeiro, Brazil.
R.P. Cunha. University of Brasilia, Brasilia, Brazil.
I. MacGregor.[1] Dept. of Civil Engineering, Formerly City University of Hong Kong, Hong Kong, China.

[1]Present address: Dubai Central Laboratories, Dubai Municipality, P.O. Box 26840 Dubai, United Arab Emirates.

Recent research into the behaviour of granite saprolites has concentrated on laboratory triaxial and direct shear tests (Irfan 1988; Cheung et al. 1988; Massey et al. 1989; Julian and Fredlund 1996). The essential features of behaviour of the saprolites are the cohesive–frictional nature of soils due to cementation bonds and suction in unsaturated deposits. Bonding appears to be a significant component of the shear strength of undisturbed decomposed granite at low confining pressures. For saturated samples subjected to high normal stresses and strain values, the shearing resistance due to the frictional component of shear strength would govern. Finally, matrix suction increases the shear strength of samples taken from above the groundwater table.

Despite the fact that laboratory tests are still recognised as the most satisfactory means of studying the mechanical behaviour of soils, establishing the likely range of shear and deformation parameters in residual materials is complex because of their variability and spatial heterogeneity.

Laboratory specimens may not be representative of large particles and boulders present in the soil mass, sensitivity of cementation bonds to disturbance can affect results, and laboratory strength measurements can differ from the shear strength of in situ materials. Under these conditions, assessment of soil properties by in situ testing techniques is attractive because ground disturbance can be minimised. Pressuremeter tests are of particular interest, since they provide a measurement of the stress–strain response of soils in situ. Analysis of the pressuremeter test is a boundary value problem that can be modelled by cavity expansion theory.

There are two methods of interpreting the pressuremeter test: in the first method each parameter is assessed independently from one portion of the pressuremeter curve; in the second and more recent method advocated by many authors (Jefferies 1988; Ferreira 1992; Cunha 1994; Ortigao et al. 1996; Schnaid 1997; Mántaras 1999) the whole pressuremeter curve is taken into account. Both loading and unloading portions of the pressuremeter test can be analysed.

The purpose of this paper is to evaluate recently developed techniques to analyse pressuremeter and dilatometer tests in residual soils, in particular the parameters derived from the unloading portion of the self-boring pressuremeter (SBPM) test. A comprehensive site investigation programme was carried out at a site in the Kowloon Peninsula, known as the Kowloon Bay research site. The tests consisted of SBPM, Marchetti dilatometer (DMT), and laboratory tests. This paper presents a summary of the test programme, the analysis of the SBPM data, and a discussion of results.

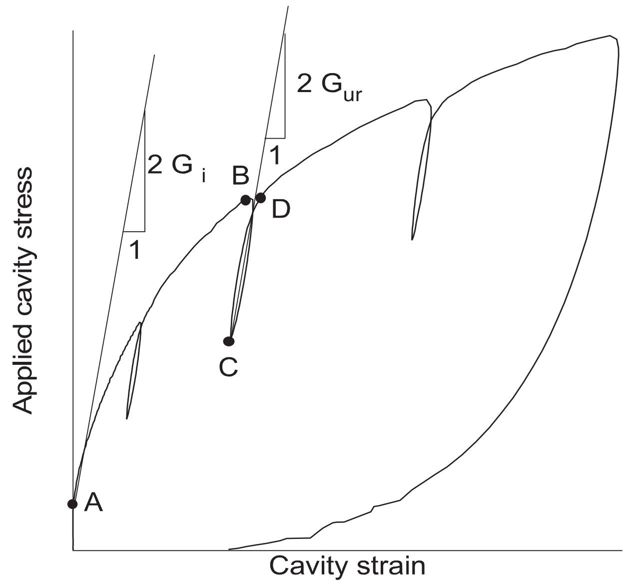
Background

Pressuremeters are cylindrical devices designed to apply uniform pressure to the wall of a borehole by means of a flexible membrane. Both pressure and deformation at the cavity wall are recorded and interpretation is provided by cavity expansion theories under the assumption that the probe is expanded in a linear, isotropic, elastic, perfectly plastic soil. Under this assumption the soil surrounding the probe is subjected to pure shear only. Acknowledging that the greatest potential of the pressuremeter lies in the measurement of modulus, it is a common practice to carry out a few unloading–reloading cycles during the test, as illustrated in Fig. 1. If the soil is perfectly elastic in unloading, then the unloading–reloading cycle will have a gradient of $2G_{ur}$, where G_{ur} is the unload–reload shear modulus.

Current references and interpretation methods of pressuremeter test data are related to either sands tested under fully drained conditions or clays under fully undrained conditions. There is scarce experience in materials other than clays and sands and interpretation is constrained to measurements of soil stiffness (Martin 1977; Rocha-Filho and Carvalho 1988). Ortigao et al. (1996), Schnaid (1997), and Schnaid and Mántaras (1998) have just recently shown that the framework developed for sands can be extended to nontextbook materials such as residual soils, which allows shear strength parameters to be derived from the test. This should give the pressuremeter broader acceptance and application.

Interpretation in residual soils is made by examining the suitability of cavity expansion–contraction analysis developed for tests in sand. It implies that all pressuremeter tests were carried out under drained conditions and volume changes around the cavity occur freely. Careful monitoring of pore-water pressures around the probe generally shows that no excess pore-water pressure is developed in the soil during pressuremeter expansion. As a consequence, any selected method of interpretation should take into account the effects of volume change due to drainage during the test.

Fig. 1. Measured G values from SBPM test. G_i, initial tangent shear modulus.



The possibility of incorporating the cohesive–frictional nature of soils in cavity expansion analysis was first presented by Baguelin et al. (1978). An attempt was made to assess the stress distribution around the probe, but there was no formulation to express the stress–strain behaviour in a closed-form solution. Currently, there are several different analyses available for the interpretation of the test using the experimental data from both the loading portion of the pressuremeter curve (Hughes et al. 1977; Carter et al. 1986; Manassero 1989; Yu and Houlsby 1991; Cunha 1996) and the unloading portion (Houlsby et al. 1986; Withers et al. 1989; Yu and Houlsby 1995). Three of these methods have been considered more appropriate on the basis of the available geotechnical information at the site and the lack of experience on the analysis of weathered residual soils. The selected methods of interpretation are briefly described and discussed. The intention is not to compare the magnitude of soil parameters given from the different methods, but to look at the likelihood of assessing realistic parameters from interpretation of testing data in residual soil sites.

Method of Hughes et al. (1977)

The method developed by Hughes et al. (1977) for interpretation of the loading portion of the pressuremeter curve is based on a number of assumptions: the probe is considered to be of infinite length; the soil surrounding the expanding cavity is assumed to be deformed under conditions of axial symmetry and plane strain; and the soil is assumed to be isotropic, linear, and elastic until yield occurs when the

Mohr-Coulomb condition is satisfied. According to this method, an experimental curve of effective cavity stress (P') against cavity strain (ε) should be plotted in a logarithmic scale. In this space a linear plot should be obtained in the plastic section (usually in cavity strain ranges of 1–10%), and the gradient of this line (S_d) is related to the soil friction and dilation angles. The slope of the plastic phase is rarely clearly defined and requires a certain degree of subjectiveness in the interpretation (Mair and Wood 1987; Clarke 1995). Nevertheless, this method has been selected in this study for its wide acceptance on the interpretation of pressuremeter test data in sands.

Method of Houlsby et al. (1986)

Houlsby et al. (1986) proposed an analysis method for interpretation of the unloading section of the pressuremeter test in dilatant soils. This method is based on a small strain analysis and the assumptions are similar to those of the loading analysis proposed by Hughes et al. (1977). The analysis assumes an elastic – perfectly plastic Mohr-Coulomb model in which the elastic deformation in the plastically deforming zone has been ignored. The main conclusion from Houlsby et al. is that the plastic portion of unloading curve is a straight line if the effective cavity stress, ln P', is plotted against –ln ($\varepsilon_{max} - \varepsilon$), where ε_{max} is the maximum cavity strain. The slope of the unloading curve S_{ud} is expressed as a function of friction and dilation angles. This approach is used for the same reasons as those given for the method of Hughes et al. (1977).

Method of Yu and Houlsby (1991, 1995)

Yu and Houlsby (1991, 1995) proposed closed-form solutions for the stress and displacement fields in the soil during cavity expansion. The soil is modelled as linear elastic – perfectly plastic using a nonassociated Mohr-Coulomb yield criterion. The solution introduces dilation to the analysis of large strain expansion. In 1995, Yu and Houlsby presented a further analysis for the unloading portion of the pressuremeter curve in which once again a nonassociated Mohr-Coulomb yield criterion is used to account for dilation of the soil during shearing. Large strains are taken into account by adopting an appropriate strain definition within the plastically deforming region. It should be noted that it was not possible to present the pressure–strain relationship for cavity unloading in the form of a single formula. The final results are presented in such a way that the pressure–strain curve can be constructed by a procedure that involves the use of rapidly convergent series.

Bosch et al. (1997) demonstrated that the method of Yu and Houlsby (1991) for the loading portion of the pressuremeter curve gives results that are in close agreement with those obtained from the method of Carter et al. (1986). The use of the approach of Carter et al. would be redundant for the comparisons presented hereafter and will not be carried out or discussed.

A further modification of the theory was carried out by Cunha (1996), who incorporated elastic strains in the idealised "plastic zone" surrounding the cavity. The interpretation philosophy, however, differed from that of the original Hughes et al. (1977) method, since with the new theory of Cunha the fitting technique had to be adopted. The results of the method of Cunha are also in close agreement with those of the method of Carter et al. (1986) and thus will not be incorporated in the comparison herein.

The work presented in this paper aims to broaden the application of the theories developed for sand by evaluating the likelihood of assessing fundamental parameters in residual soils. For this purpose, a curve-fitting technique is proposed for interpretation based on the assumption that representative soil parameters will produce a match between the experimental and idealized model curves.

Interpretation procedure

Most interpretations of pressuremeter tests involve fitting curves to the test data (e.g., Clarke 1995). However, the concept of producing an image curve of the complete pressuremeter loading and unloading curve is relatively new. Jefferies (1988) proposed that fitting of the entire curve should be used to obtain a value for the initial horizontal stress. Software developed for personal computers performs a curve-fitting analysis from which soil parameters and the in situ horizontal stress can be assessed. Later, Ferreira and Robertson (1992) suggested that horizontal stress, initial tangent modulus, and shear strength could be obtained by fitting a hyperbolic function to the field data. Both methods have been developed and tested in clays.

In sands J.M. Hughes (personal communication, 1989) advocated the "image-matching" or curve-fitting procedure for the SBPM curve interpretation. Research in the early 1990s at the University of British Columbia was carried out on this subject, leading to the full establishment of the curve-fitting technique for the interpretation of SBPM tests in sands (Cunha 1994). Cunha and Campanella (1998) employed the fitting technique with undisturbed and disturbed SBPM testing curves from a well-documented granular deposit in Vancouver, British Columbia. They adopted the Cunha (1996) approach and carried out the interpretation analysis by matching field and idealised curves at distinct strain (or "fitting") ranges. They concluded that, despite the inherent disturbance of the testing curve, it is possible to obtain high-quality parameters if the fitting is conducted between approximately 5 and 10% cavity strain to avoid the disturbance at initial stages. In addition, due to the effects of length to diameter ratios, L/D, it is reasonable to assume that a pressuremeter of finite length would display a stiffer response than for a pressuremeter of infinite length which has some effect on interpretation at large strains (Houlsby and Carter 1993).

Ortigao et al. (1996) used a computer program based on the theory developed by Carter et al. (1986) to apply a curve-fitting technique to a cohesive–frictional, unsaturated, and tropical material, namely Brasilia porous clay. Pressuremeter tests were carried out above the water table and pressuremeter expansion was assumed to take place under drained conditions. Parameters derived from the analysis have been successfully compared with those from other laboratory and in situ tests.

Schnaid (1997) and Schnaid and Mántaras (1998) used a similar methodology to analyse weathered cohesive–frictional materials but employed the set of equations developed by Yu and Houlsby (1991) for loading and Yu and Houlsby (1995) for unloading. The analytical methods were implemented in a program in a mathematical package (MathCad). The program executes the calculations for loading and unloading equations and plots the data. The operator compares the quality of the match by visual inspection and modifies the input soil parameters until an adequate curve fitting is reached. The best fitting was evaluated by simple regression analysis (correlation coefficient r^2). It is interesting to note that input parameters are either kept within the limits defined by independent test data (effective internal friction angle ϕ' and dilation angle ψ measured or estimated by laboratory or in situ tests) or measured in the pressuremeter itself (shear modulus G and P'_o values).The parameters that produce an analytical curve which satisfactorily fits the experimental results are, in principle, representative of the soil behaviour.

The elastoplastic models implemented in the application software generally involve six variables: internal friction angle ϕ', cohesion c', horizontal stress σ'_{ho}, shear modulus G, Poisson's ratio υ, and dilation angle ψ. Adjustment of the pressuremeter curve is little affected by Poisson's ratio (estimated to be between 0.2 and 0.3). Cohesion is close to zero, as indicated by the triaxial data, which encouraged the use of theories developed for sands. Dilation is linked to friction angle through Rowe's theory (Rowe 1962). The expansion curve is therefore adjusted as a function of at least four variables: ϕ', ψ, σ'_{ho}, and G. To provide the engineer with guidance in the use of the fitting process, a brief description of the parameter selection process is presented.

Effective horizontal stress

Several methods have been developed to assist in the prediction of effective horizontal stress from SBPM curves in both clay and sands (e.g., Marsland and Randolph 1977; Wroth 1984). The selection of σ'_{ho} is subjective, since interpretation of the measured value depends upon the method of insertion of the probe into the ground. More importantly, there is no accumulated experience that can be applied to weathered materials.

It is easily demonstrated from the various theories that information on the initial horizontal stress remains embedded in the value of limit pressure, in which case it can be assessed from the interpretation of the entire experimental curve (e.g., Baguelin et al. 1978; Clarke 1995). It is therefore suggested in the present analysis to use an alternative procedure to predict σ'_{ho} by adopting the lift-off pressure as an input reference value to perform a curve-fitting analysis. Once an image matching is achieved, a more reliable assessment of σ'_{ho} may be provided.

Shear modulus

The shear modulus adopted for the curve fitting varies within the range of values measured from unload–reload cycles performed in each individual test. Measured G values reflect the relevant mean stresses and strain amplitudes for a given test and should in theory provide a realistic fitting to the pressuremeter pressure–expansion curve.

Shear strength

Once the values (or range) of G and σ'_{ho} are selected, a theoretical curve is produced from which the friction angle is assessed. The relationship between the dilation angle and the friction angle is given by Schanz and Vermeer (1996) in accordance with the principles of Rowe (1962):

$$[1] \qquad \sin\psi = \frac{\sin\phi'_{ps} - \sin\phi'_{cv}}{1 - \sin\phi'_{ps}\,\sin\phi'_{cv}}$$

where ϕ'_{ps} and ϕ'_{cv} are the plane strain and critical state internal friction angles, respectively. For cohesive–frictional materials the values of the cohesion intercept c' must also be adjusted to meet the different degrees of cementation existing within a given soil profile. For the current testing programme the values of c' were taken as zero after careful examination of the laboratory data (discussed later in the analysis of results).

Other factors

The length to diameter ratio effects on strength parameters have to be considered in the analysis. A number of studies have been carried out to evaluate and quantify the finite pressuremeter length effect on pressuremeter test results (Yan 1988; Salgado and Byrne 1990; Cunha 1994; Schnaid et al. 1995; Houlsby and Carter 1993; Ajalloeian 1996). It was concluded that the length to diameter ratio of a pressuremeter probe has a significant effect on the strength parameters deduced from loading and unloading pressuremeter test results, mainly if the pressuremeter has a slenderness ratio (L/D) lower than approximately 6.

In the present work, the correction proposed by Ajalloeian (1996) and Ajalloeian and Yu (1998) is adopted, based on calibration chamber tests in sand:

$$[2] \qquad \frac{S_{d\infty}}{S_d} = 1 - \frac{D}{L} \qquad \text{for loading}$$

$$[3] \qquad \frac{S_{ud\infty}}{S_{ud}} = 1 - 0.67\frac{D}{L} \qquad \text{for unloading}$$

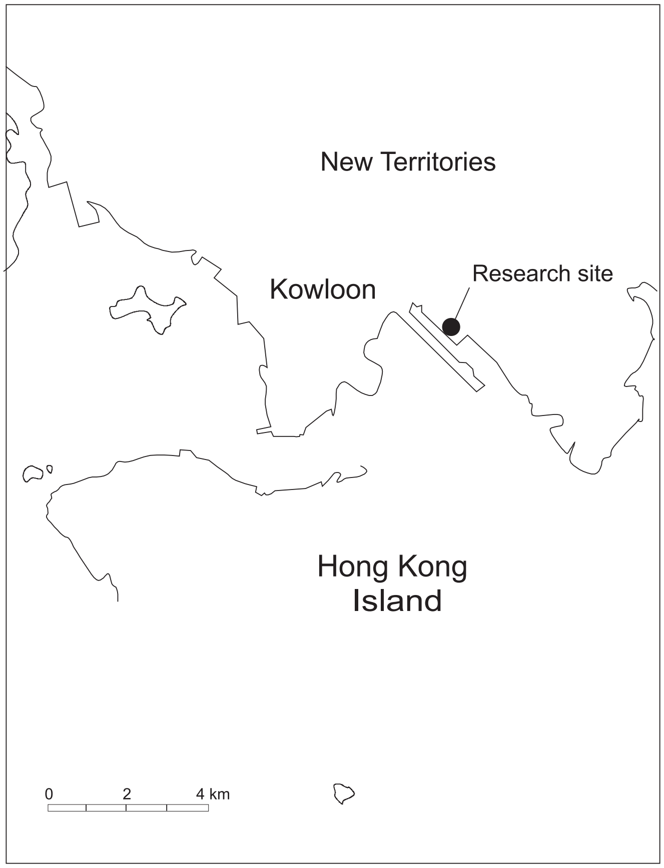
The formulation indicates that a trend exists of increasing angles of friction and dilation with decreasing L/D ratio. This correlation has been selected because it is the only one to directly correct the measured values of the pressuremeter slope S_d and therefore can be applied to every method presented here.

In situ test procedures

A brief description is given of the in situ test procedures used in the analyses. The standard penetration tests (SPT) were carried out in drillholes in accordance with British Standard 1377 (BSI 1990). The N value recorded is the number of blows required to drive the split-spoon sampler through 300 mm. The N values correspond to uncorrected values for SPT energy.

The self-boring pressuremeter tests (SBPM) were performed using a Camkometer probe and adopting standard strain-controlled testing procedures (e.g., Clarke 1995). The soil entering the tip of the probe is cut by a rotating cutter and then flushed to the surface. Drilling mud was used as the flushing fluid. A cylindrical membrane, fitted outside of the instrument, was expanded by gas pressure. The

Fig. 2. Test site.



expansion is sensed by electrical transducers which monitor radial displacements at three positions around the instrument and gas pressure. The electrical signals are relayed to the data-acquisition system, located at the surface, by electrical cables running inside the armoured tube that supplies the gas pressure. Transducers and the membrane were calibrated, the latter to account for stiffness and compliance.

When the probe reached the required test depth, a time of about 30 min prior to carrying out a test was sufficient to ensure full dissipation of excess pore pressures due to probe installation. This was followed by strain-controlled membrane inflation at a constant rate of strain appropriate for drained conditions. A small unload–reload cycle was included and the test continued until the maximum strain of the equipment of about 12% was reached. The subsequent deflation phase to zero total stress was monitored.

The dilatometer was developed in Italy by Marchetti (1980) and consists of a 14 mm thick, 95 mm wide, and 220 mm long flat blade which is driven or pushed into the soil. On one face there is a 60 mm diameter steel diaphragm capable of a lateral expansion of 1 mm under gas pressure.

The SBPM and DMT tests were carried out only in the residual soil layer; the overlying soil layers were predrilled.

The test site

The test site is located in the Kowloon Peninsula in Hong Kong, adjacent to Kowloon Bay, as shown in Fig. 2. The test programme consisted of a series of boreholes, sampling and laboratory tests, SBPM tests carried out by Soil Mechanics Ltd., and DMT tests carried out by the Hong Kong University.

The bedrock at this site, as given by geological maps and rock outcrops adjacent to the site, consists of fine- and medium-grained granite. The findings from the drilling are in accordance with the geological maps and are summarised as follows: (*i*) alluvium; (*ii*) grade VI/V rock or completely decomposed granite (CDG) (GEO 1987, 1988); and (*iii*) rock, consisting of medium-grained granite.

The soil profile at the test site was investigated by a series of nine boreholes, allowing soil characterization, sampling, and SPT, SBPM, and DMT tests to be carried out. In three of the boreholes SPT tests were performed at 1.5 m intervals. The borehole results were fairly consistent and despite the variability it was possible to describe a typical profile. The top of the CDG is fairly horizontal in all boreholes. A summary is shown in Fig. 3 and consists of the following: (*i*) a 10–15 m layer of fill, with N ranging from 5 to 20; (*ii*) a thin, up to 5 m, marine clay deposit, with low values of N, in the range 1–3; (*iii*) 15–20 m of alluvial silty clay; (*iv*) completely decomposed granite (CDG), consisting of extremely weak, light grey to reddish brown residual soil and saprolites, consisting of clayey silty fine sands with some gravel, with N values of 30 at the top of the layer but sharply increasing with depth up to 100 at about 40 m; (*v*) the water level was observed at about 2.6 m depth; and (*vi*) results of sieve analysis.

Isotropically consolidated undrained triaxial tests with pore-pressure measurements were carried out on undisturbed samples obtained from 100 mm Shelby tubes taken at depths of 30, 32, and 36 m. Cementation produced a lightly bonded structure which enables specimens to be retrieved. Specimens were carefully trimmed in the laboratory, placed in the triaxial cell, and taken to saturation by applying a back pressure of up to 200 kPa, ensuring B values (Skempton 1954) of at least 0.97. Failure envelopes yielded cohesion intercepts close to zero and friction angles ranging from 36 to 41° as illustrated in Fig. 4.

A typical SBPM test is shown in Fig. 5 to illustrate the expansion and contraction curves and the unload–reload cycles carried out to measure the shear modulus. A total of eight high-quality field tests were evaluated in the present study. Three unload–reload cycles were carried out in each test, exhibiting characteristics similar to those of tests in sand. Cycles are highly nonlinear and the measured slope is considerably steeper than the secant or tangent to the pressure–expansion curve (e.g., Robertson and Hughes 1986; Bellotti et al. 1989). A procedure recommended by Houlsby and Schnaid (1994) which accounts for the effects of membrane stiffness and compliance was adopted to estimate the shear modulus. For calculating the slope, a single line was drawn between the two apexes of each cycle.

Analysis of the results

Soil parameters and in situ stresses were assessed from SBPM, dilatometer, and triaxial test data. Reference is made to the variation of soil stiffness, friction angle, and effective in situ horizontal stress with depth.

Fig. 3. (*a*) Soil profile. BH, borehole; GWL, groundwater level. (*b*) *N* values from SPT. (*c*) Sieve analysis.

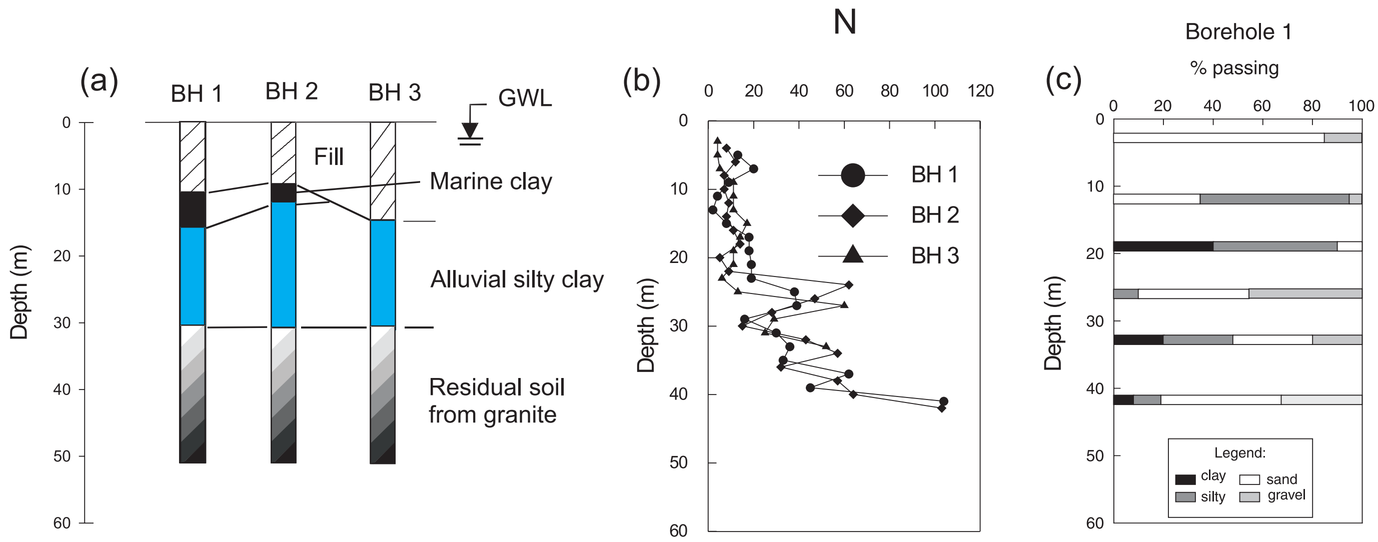
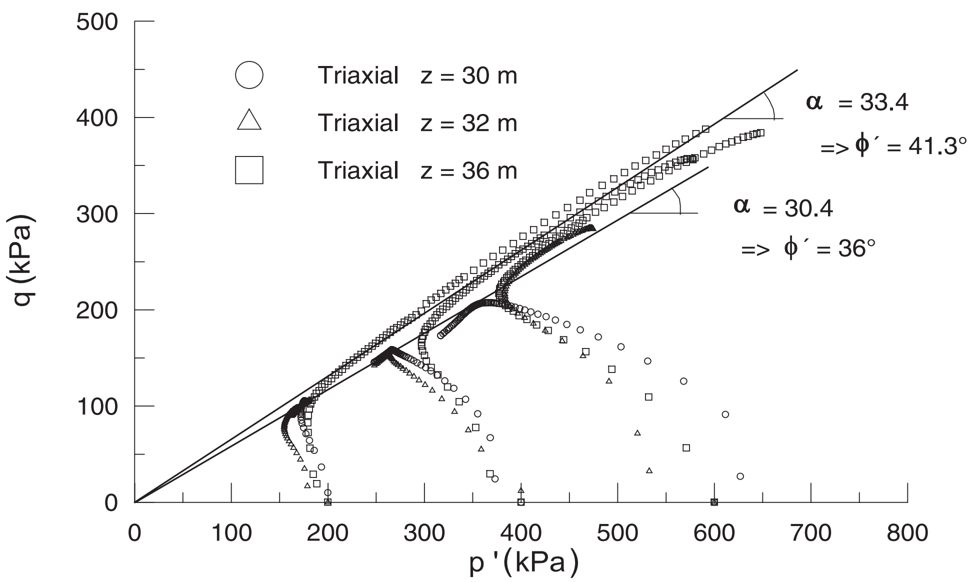


Fig. 4. Failure envelope for three sites of consolidated isotropically undrained (CIU) triaxial tests. *z*, depth.

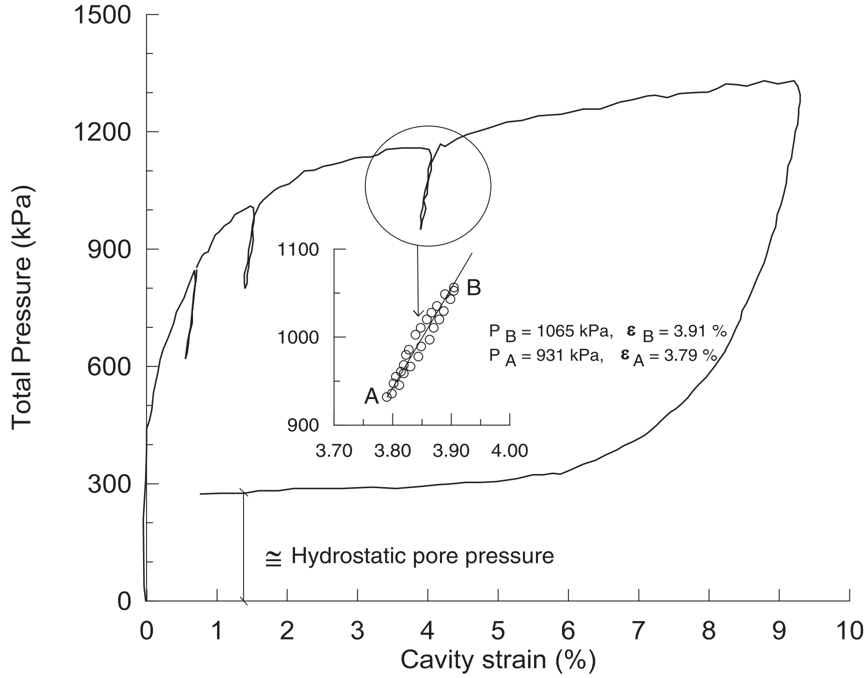


The implemented elastoplastic model in the applicative software is performed in a version that uses the six variables directly involved in the analytical development of the method: internal friction angle ϕ', cohesion intercept c', in situ horizontal stress σ'_{ho}, shear modulus G, Poisson's ratio υ, and dilation ψ. Poisson's ratio has little influence on the results and thus is estimated from general background (assumed to be between 0.2 and 0.3 for practical applications). The dilation is calculated from the correlation of Rowe (1962), as mentioned in the previous section. The values of G and σ'_{ho} were measured directly from pressuremeter data, whereas shear strength parameters were assessed from the proposed approach. A general guideline for the curve-fitting technique is summarized as follows:

(1) Horizontal stress — There is no compiled experience that can be applied to evaluate the feasibility of assessing the in situ horizontal stress in residual soils from the pressuremeter lift-off pressure. Field and laboratory data suggested that both could produce soil remoulding stress relieved effects and mechanical disturbance (de Mello 1972; Sandroni 1985, 1988). It is recognised from the various theories (Carter et al. 1986; Yu and Houlsby 1991) that information on the initial horizontal stress remains embedded in the value of limit pressure, in which case it can be assessed from the interpretation of the entire experimental curve. It is therefore suggested that σ'_{ho} be estimated from the lift-off pressure and that this point should be used as a reference value to perform a curve-fitting analysis that will refine the estimate of σ'_{ho} once an image matching is produced.

(2) Shear modulus — It is relatively simple to obtain G values from the measured pressure versus radial strain of an expanding cylindrical membrane at a given mean stress. It is therefore suggested that the curve fitting should be initiated using stiffness values from unload–reload cycles, G_{ur}. For the unloading portion, the value of G is directly calculated from the fitting process by drawing a single line between the

Fig. 5. Typical SBPM test result at a depth of 32.60 m in borehole BP 3, Kowloon Bay. P_A, total stress at the end of unloading; P_B, total stress at the closure of cycle; ε_A, shear strain at cavity wall at the end of unloading; ε_B, shear strain at cavity wall at the closure of cycle.



point that defines the end of loading and the representative point of the theoretical plastic reverse of the experimental unloading curve.

(3) Strength parameters — The value of the cohesion intercept measured from laboratory tests was close to zero and was thus disregarded in the present analysis. The values of ϕ' and σ'_{cv} were assessed independently from complementary laboratory or in situ tests and adopted as input data in the curve fitting. The theoretical curve was then constructed and the parameters tuned to obtain the best fit.

In summary, with the fitting technique a coupled set of six parameters should be used to describe the soil behaviour. To reduce ambiguities, some constraints must be imposed to the analysis: the estimate of σ_{ho}' is limited to the range of values of lift-off pressures measured by the three strain arms. The pressuremeter modulus can vary only within the range of measured unload–reload loops G_{ur}. The friction angle ϕ' is allowed to vary during the fitting process.

Typical SBPM pressure–expansion curves in residual granites are presented in Fig. 6, in which the average of the three strain arm measurements is shown. The unload–reload cycles have been intentionally removed from the curves to avoid distracting from the comparisons between measured and predicted behaviour. The visual comparison obtained for the SBPM test at 29.60 m clearly indicated that a good fit (r^2 = 0.9976) has been achieved between theory and data over the whole curve for both the loading and unloading analysis. There are cases such as the test at 30.60 m in which the curve fitting does not reproduce the measured behaviour with the same degree of accuracy (r^2 = 0.9808). However, it is necessary to bear in mind that the magnitude of G adopted as an input parameter in the analysis is restricted to the range of values actually measured in the pressuremeter test. The constrains imposed to input data introduce some limits to the quality of the comparisons but enforce the consistency among all parameters.

Friction angle

Triaxial friction angles (ϕ'_{tx}) were measured in tests carried out on undisturbed samples retrieved from Shelby tubes. It is necessary to convert these values into plane strain friction angles to facilitate comparison with in situ testing results. A few alternative expressions may be used to convert ϕ'_{tx} to ϕ'_{ps}, and the simple relationship proposed by Lade and Lee (1976) was adopted in the present analysis:

$$[4] \qquad \phi'_{ps} = (\phi'_{tx} \times 1.5) - 17$$

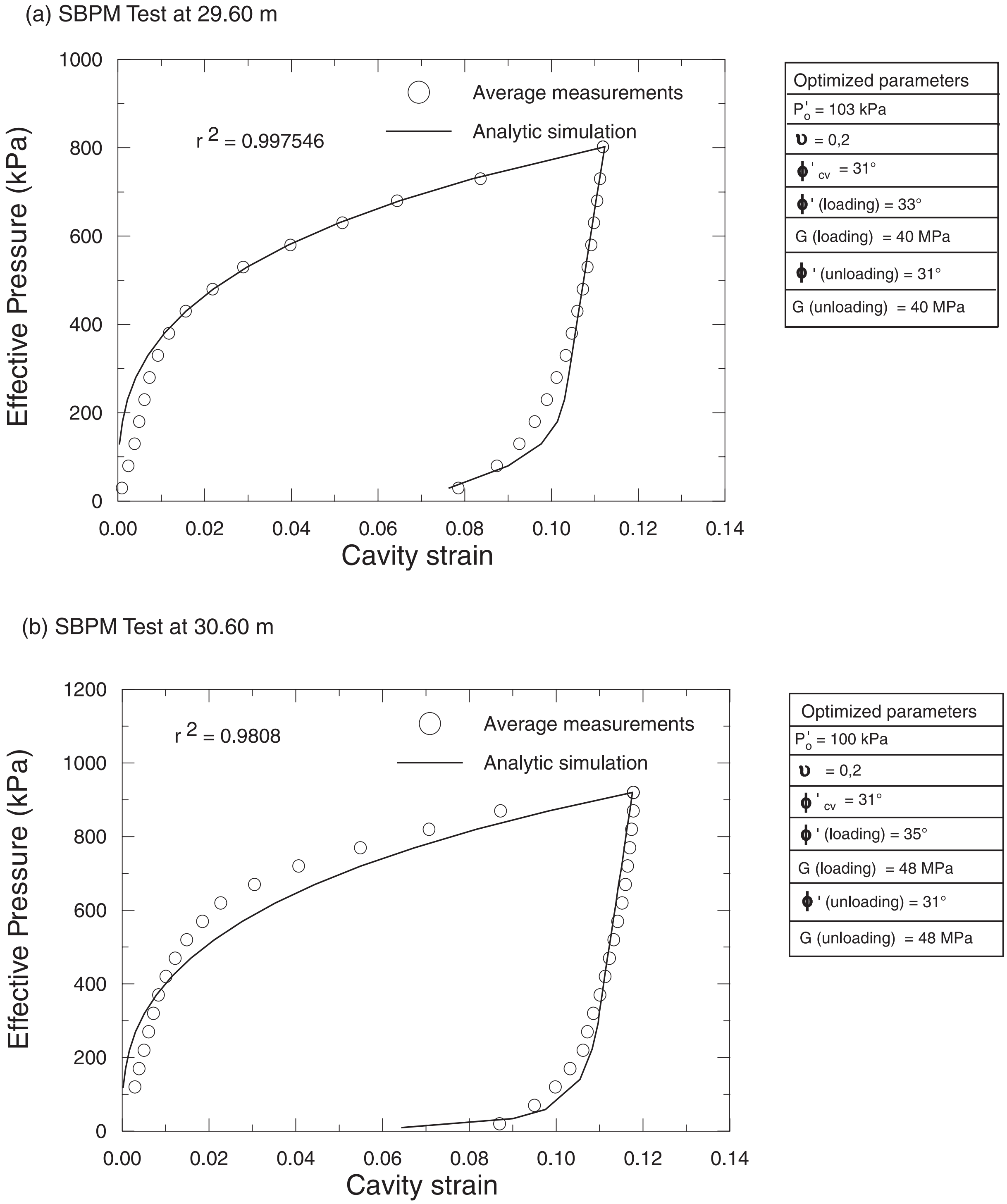
Estimating friction angle in sands is one thing that DMT can reasonably accomplish at present (Marchetti 1997). Existing correlations have been proposed for noncemented sands, however. A proposal is made in this study to apply a well-established correlation developed for sand to a lightly cemented material in an attempt to assess a lower bound estimate for the friction angle:

$$[5] \qquad \phi'_{ps} = 28 + 14.6 \log K_D - 2.1 \log^2 K_D$$

where K_D is the DMT horizontal stress index. The penetration of the DMT blade represents a plane strain condition, and therefore its interpretation produces a prediction of a plane strain friction angle ϕ'_{ps} (Briaud and Miran 1992).

With the SBPM, different methods of analysis give different ϕ' values with considerable data scatter as indicated in Table 1 and Fig. 7. Difficulties were experienced with the

Fig. 6. Typical comparisons between theoretical and experimental pressure–expansion curve.



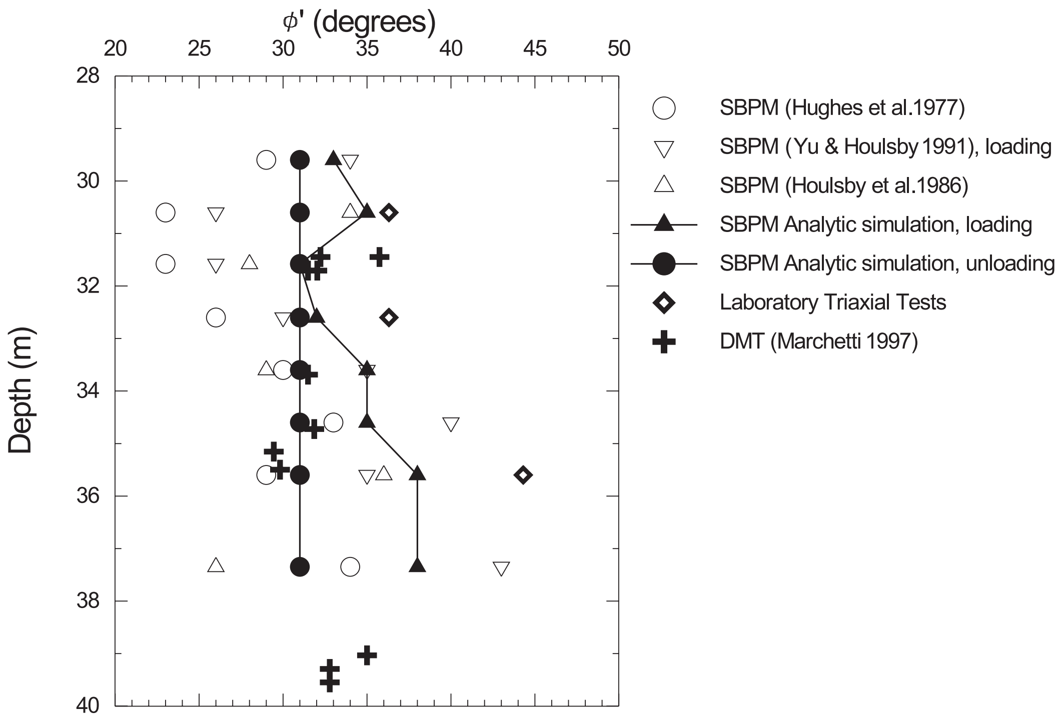
application of the analytical formulations by Yu and Houlsby (1991) and Hughes et al. (1977), since the experimental data did not produce a straight line required to predict a friction angle. The same problem is frequently observed in noncemented sands (e.g., Clarke 1995), which suggests that most soils are not perfectly plastic and therefore the gradient of the line varies depending upon the strain level. As a consequence, ϕ' values in the depth range 29–34 m are too low, below the predicted critical state values reported for the decomposed granite of Hong Kong (e.g., Cheung et al. 1988; Julian and Fredlund 1996). Therefore, these methods were no longer considered.

The curve-fitting technique applied to the loading portion of the SBPM test gave results which are quite consistent, above the assumed critical state values and increasing with depth. This pattern is compatible with evidence provided by *N* values of strength continuously increasing with an increase in depth. The unloading portion of the SBPM curve

Table 1. Friction angle from self-boring pressuremeter (SBPM), dilatometer, and triaxial tests.

		Pressuremeter φ′ loading			Pressuremeter φ′ unloading			
Test No.	Depth (m)	Hughes et al. 1977	Yu and Houlsby 1991	Curve fitting	Yu and Houlsby 1991	Curve fitting	Dilatometer	Laboratory φ′ (triaxial) plane strain
7	29.60	29	34	33	31	31		
8	30.60	23	26	35	34	31		37
9	31.58	23	26	31	28	31	31–36	
10	32.60	26	30	32	31	31		37
11	33.60	30	35	35	29	31	35	
12	34.60	33	40	35	—	31	31	
13	35.60	29	35	38	36	31		45
14	37.35	34	43	38	26	31		

Fig. 7. Friction angles.



yielded friction values close to the critical state friction angle, a trend already suggested by Houlsby et al. (1986). Great reliance has been placed on these results, as the unloading portion yielded φ′ values of 31° that are constant with depth; this may indicate that φ′ values are not being affected by void ratio and the stress state.

Cavity contraction theories applied to derive fundamental soil parameters in a curve-fitting process are attractive because the unloading curves are insensitive to the well-known disturbance generated during insertion of the SBPM, as well as cone pressuremeter and DMT insertion. Houlsby et al. (1986) were the first authors to introduce the cavity contraction theory to analyse the unloading stage of SBPM tests in sands. Despite the high potential of this model, preliminary interpretation of SBPM results in the United Kingdom revealed inconsistencies for the derived loading friction angle. This led Houlsby et al. to comment on the necessity of a large strain formulation to improve their model. Withers et al. (1989), after the analysis of several pressuremeter tests in sand, concluded that simple cavity contraction models are not suitable for deriving strength parameters from either the core pressure meter test (FDPM) or SBPM tests. This is due to the extremely complicated behaviour of the cavity during unloading, with possible stress reversals and arching phenomena. On the interpretation of the current set of data, it is also possible that the whole analysis is in error because of the analytical assumptions embedded in the formulations and the possible effects of arching during the unloading phase of the pressuremeter test. Comparisons between the parameters obtained from the present analysis and those obtained from other in situ and laboratory tests reported for decomposed granites are encouraging, however (e.g., Cheung et al. 1988; Julian and Fredlund 1996).

It is noteworthy that peak friction angles from the DMT lie between the lower and upper boundaries of φ′ given by the curve-fitting technique applied to the unloading and loading portions, respectively. Additionally, the DMT φ′ values lie close to the lower bound, which is expected from a relationship

Fig. 8. In situ stress ratio K_o.

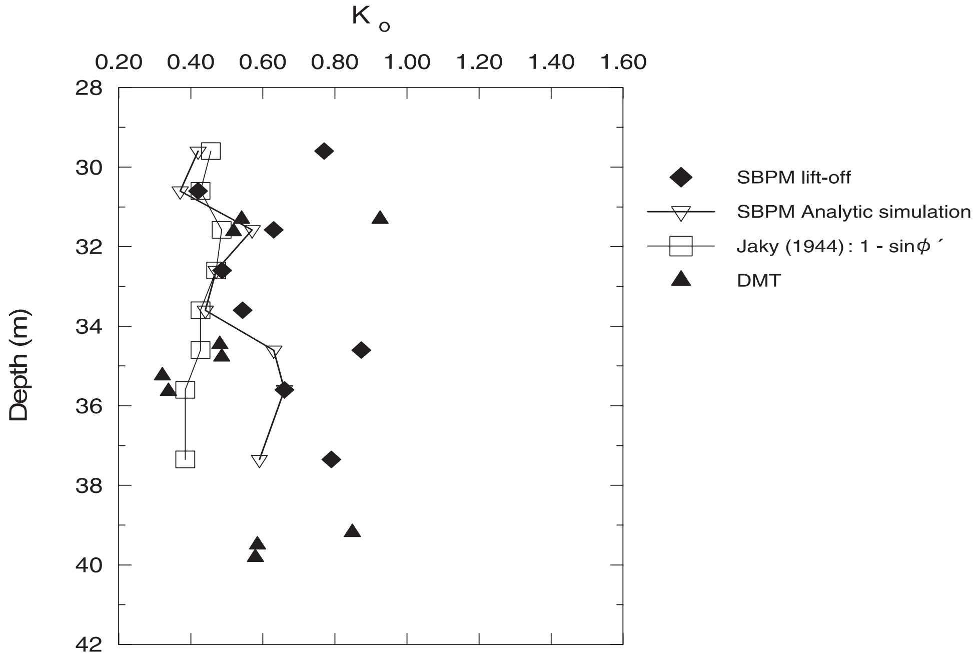


Table 2. K_o from SBPM tests.

Test No.	Depth (m)	Arm 1	Arm 2	Arm 3	Average	Curve fitting
7	29.60	0.93	1.00	0.38	0.77	0.42
8	30.60	0.39	0.38	0.49	0.42	0.37
9	31.58	0.64	0.58	0.67	0.63	0.57
10	32.60	0.54	0.43	0.49	0.49	0.47
11	33.60	0.44	0.61	0.58	0.54	0.44
12	34.60	0.66	1.36	0.67	0.90	0.63
13	35.60	0.75	0.69	0.54	0.66	0.66
14	37.35	0.70	1.06	0.61	0.79	0.59

that yields slightly conservative values. Laboratory plane strain friction angles ϕ' ranging from 36° to 44° were determined and also showed a trend of increasing with an increase in depth. There is a reasonable match between the triaxial test results and those of other in situ tests.

Horizontal stress ratio

Figure 8 and Table 2 summarise the results of the horizontal stress ratio K_o. The classical method for obtaining the horizontal stress from the lift-off pressure of the SBPM arms produced rather scattered data, but apart from a few spurious results it appears that K_o increases with an increase in depth.

It is well established from the interpretation of pressuremeter data that the average pressure–expansion curve, rather than measurements of individual arms, is more appropriate for estimating the horizontal stresses (e.g., Mair and Wood 1987; Clarke 1995). The averaging of the output of the strain arms helps in the compensation of errors as a result of differential disturbance and other effects imposed in the horizontal plane during insertion, averages the anisotropic soil response along the horizontal testing plane by assuming beforehand that the soil is approximately isotropic, and compensates the differential results at each arm caused by an eventual translation of the center of the SBPM. Studies carried out with the six strain arms SBPM (Cunha 1994) demonstrated that such combined effects can, indeed, lead to reasonably distinct testing curves at different arms (at the same testing depth). On the other hand, however, the use of a mean value of σ'_{ho} at each depth would not change significantly the spread of data presented in Fig. 8. It should be understood that the intention here is only to highlight the difficulties in estimating a reliable measurement of the magnitude of K_o from the lift-off pressure in residual soils.

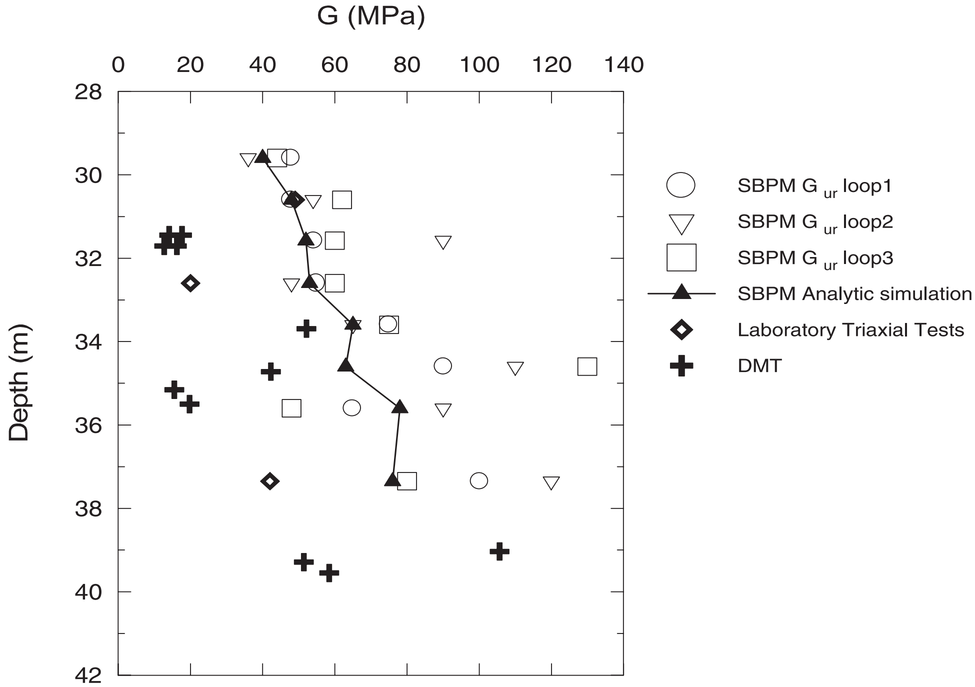
The curve-fitting technique presents a mean value, with much less scatter, as pointed out in the philosophy behind this approach. In general, there is good agreement between the DMT and SBPM results, despite the scatter observed between 34 and 38 m depth.

K_o values were computed through the well-known Jaky (1944) equation established for noncemented sands ($K_o = 1 - \sin \phi'$) employing ϕ' values from the SBPM curve-fitting technique, and its prediction is also plotted in Fig. 8. Despite the lack of reliability usually associated with the assessment of K_o in sands, it is surprising to observe that both methods of in situ testing produced values of the same order of magnitude and comparable with those obtained using Jaky's equation for normally consolidated soils. The observed trend is even more unexpected for a lightly structured material in which the bond effects are not fully known.

Shear modulus

The unload–reload moduli from small cycles gave G_{ur} values that increase with depth, from about 40 to 100 MPa with some scatter. This scatter is considerably decreased through the curve-fitting analysis, which averages the unload–reload values. Also, the curve-fitted G values agree with the unload–reload data, which gives consistency to the

Fig. 9. Shear modulus.



curve-fitting method of analysis. The results are summarised in Fig. 9 and Table 3.

The DMT gives the one-dimensional compression modulus *M* values which were transformed through the theory of elasticity into *G* values. Their range of values is about 50% lower than that of the SBPM data, but with a similar trend of increasing with an increase in depth.

G moduli from triaxial tests were obtained from a strain level of approximately 0.05% shear strain and are plotted. The choice for a 0.05% strain is arbitrary and was selected here to illustrate a possible comparison between laboratory and field data. At about 30 m depth, the agreement is good. The value at the intermediate depth is somewhat lower than the SBPM data range. The lower depth also agrees well and shows a clear trend of increasing *G* with an increase in depth.

To understand the difference between *G* obtained from different test types it is necessary to examine the shear strain and mean stress level effects. Therefore, an attempt was made to express the shear moduli data as a function of shear strain.

G_{ur} from SBPM were corrected to account for the stress level according to Bellotti et al. (1989) through the following equation:

$$[6] \qquad G_{ur}^{c} = G_{ur}\left(\frac{s_o'}{s_{avg}'}\right)^n$$

where G_{ur}^{c} is the corrected modulus; *n* is the modulus exponent, which generally ranges from 0.3 to 0.5 (e.g., Bellotti et al. 1989); s_o' is the initial mean effective stress calculated by

$$[7] \qquad s_o' = \frac{\sigma_r' + \sigma_\theta'}{2} = \sigma_{ho}'$$

Table 3. Shear modulus from SBPM.

		G_{ur} (MPa)			G (MPa)	
Test No.	Depth (m)	Loop 1	Loop 2	Loop 3	Loading curve fitting	Unloading curve fitting
7	29.60	48	36	44	40	40
8	30.60	48	54	62	48	48
9	31.58	54	90	60	52	52
10	32.60	55	48	60	53	53
11	33.60	75	65	75	65	65
12	34.60	90	110	130	63	63
13	35.60	65	90	48	78	78
14	37.35	100	120	80	76	76

and is equal to the in situ horizontal stress σ_{ho}' in the elastic range, where σ_r' is the effective radial stress, and σ_θ' is the effective in situ horizontal stress; and s_{avg}' is the average effective in situ stress. In the elastic domain, $s_{avg}' = s_o'$. However, in the plastic domain s_{avg}' is given by

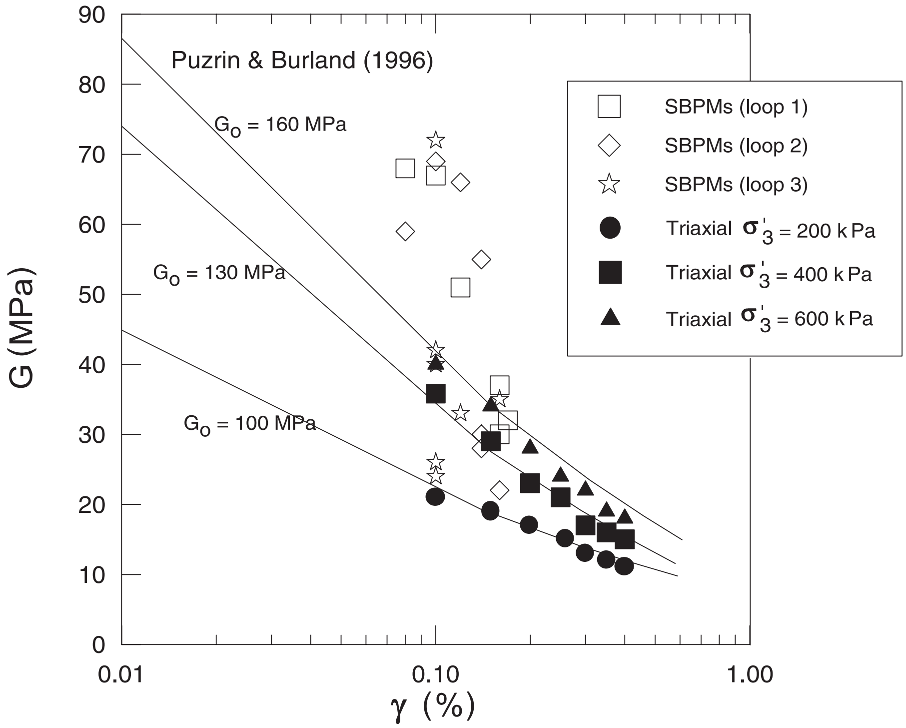
$$[8] \qquad s_{avg}' = \frac{\sigma_r' - c'\cos\phi'}{1+\sin\phi'}$$

The limit stress between the elastic and plastic zones is given by

$$[9] \qquad \sigma_p' = \sigma_{ho}'(1 + \sin\phi') + c'\cos\phi'$$

Despite the fact that c' is close to zero in this particular residual soil site, the equations presented by Yu and Houlsby (1991) have been selected because they are valid for cohesive frictional materials and can be generally adopted to the interpretation of tests in residual soils.

Fig. 10. Shear modulus versus shear strain amplitude.



The shear strain level γ for each loop was corrected according to a semiempirical correlation for granular materials to take into account the variation of the shear strain γ into the soil annulus around the cavity:

[10] $\gamma^{c} = \beta \Delta \gamma$

where γ^{c} is the corrected shear strain, $\Delta\gamma$ is the measured shear strain amplitude of the cycle at the cavity wall, and β is an empirical coefficient that takes the value of 0.5, as suggested by Robertson (1982), Robertson and Hughes (1986), and Bellotti et al. (1989).

The triaxial data have been corrected taking into account the mean effective normal stress p' and the shear strain γ given by the following equations:

[11] $p' = \dfrac{\sigma_1' + 2\sigma_3'}{3}$

[12] $\gamma = \dfrac{2(\varepsilon_a - \varepsilon_r)}{3}$

where σ_1' and σ_3' are the major and minor normal effective principal stresses, respectively; and ε_a and ε_r are the axial and radial normal strains, respectively.

The results of the corrected G moduli versus γ are plotted in Fig. 10. The trend of G reducing with increasing γ is clear, but significant scatter is observed. An attempt is made to reproduce the pattern of reduction on shear stiffness with shear strain amplitude. Among the several methods that can be used to match experimental data (Hardin and Drnevich 1972; Seed et al. 1986; Puzrin and Burland 1996), the logarithmic formulation proposed by Puzrin and Burland (1996) was selected because the stress–strain function is expressed by a small number of constants that are easy to derive and have physical meaning.

[13] $\dfrac{E_s}{E_{max}} = 1 - \alpha[\ln(1+x)]^{R}$

where E_s and E_{max} are the secant and maximum deformation moduli, respectively; α and R are parameters obtained from the experimental curve; and

[14] $x = \dfrac{\varepsilon_a E_{max}}{q_u}$

where q_u is the deviatoric stress at failure. Since E_{max} has not been directly measured, its magnitude was assessed by extrapolation of triaxial data. Later, E_{max} was converted to G_o assuming a Poisson's ratio of 0.5 (undrained triaxial tests). Typical G_o values were assumed to be within the range of 100–160 MPa. This formulation fits the pattern given by the triaxial data (see Fig. 9); SBPM moduli are scattered and produce an upper bound to existing laboratory data.

To plot the DMT results on the same graph one must estimate the equivalent shear strain and the mean stress. The DMT does not yield this value directly, but experimental and numerical works suggest that the DMT moduli are obtained at much large strain levels than ordinary SBPM tests. Campanella and Robertson (1991) indicated that the 1 mm total expansion of the dilatometer membrane is quite significant and can represent 14% equivalent cavity strain for pressuremeter expansion. A cavity strain of 12% is a typical maximum expansion for many SBPMs, as illustrated in the results presented in the early part of the curve in Fig. 6. Comparisons between SBPM and DMT data also suggest that the slope of the DMTs that characterize the soil stiffness represents a measurement of the elastic–plastic response of the soil, and therefore yields a measurement of stiffness lower than those obtained from SBPM small unloading–reloading cycles (Campanella and Robertson 1991). Bellotti et al. (1989), Lutenneger (1988), and Hryciw (1990) quoted that the typical shear modulus of the DMT is equivalent to a secant Young's modulus at 25% of strength mobilization, which also explains lower G values in Fig. 9. In conclusion, the modulus predicted from the test should be associated with large shear strains producing a lower bound to other in situ and laboratory testing data.

Conclusions

This paper presents a case history in a residual soil site, with emphasis on the interpretation of pressuremeter data using cavity expansion theories developed and tested on sedimentary granular soils. The method of analysis employed here for the SBPM results clearly showed the advantages of the curve-fitting technique which employs the whole SBPM curve. Since the resulting parameters K_o, G, ϕ', and ϕ'_{cv} are interdependent, the possibility of a concentrated error in the assessment of one single parameter is avoided. The results obtained through analyses yielded values of strength and stiffness which are representative of the decomposed granite from Hong Kong. Estimated strength values suggest that in a lightly structured material the *loading portion* of the pressuremeter curve provides an estimation of the peak shear strength parameters, whereas the *unloading portion* is associated with the critical state behaviour.

The DMT gave results that compared well with SBPM data, except for G moduli. The DMT stiffnesses were lower than those from the SBPM, which is due to different shear strain amplitudes in these tests, and emphasises the need for further studies in this area, principally for structured, unsaturated tropical soils.

The following constitutive parameters were obtained for the residual Hong Kong granite saprolite testing site (and are recommended for design) based on the studies presented herein: (*i*) G values range from 40 to about 100 MPa in a low-strain range (γ = 0.01–0.1%); for higher shear strain amplitudes, G should be selected within the range of 22–40 MPa; (*ii*) the peak plane strain friction values ϕ' are in the range of 33–40°, varying with depth; (*iii*) the critical state friction angle ϕ'_{cv} is close to 31°; and (*iv*) K_o is in the range of 0.4–0.6, which is typical of the normally consolidated behaviour of noncemented, cohesionless soils.

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List of symbols

c' cohesion intercept
D diameter of pressuremeter membrane
E_{max} maximum deformation modulus
E_s secant deformation modulus
G shear modulus
G_i initial tangent shear modulus
G_o maximum shear modulus
G_{ur} unloading–reloading shear modulus
$G_{ur}{}^c$ corrected unloading reloading shear modulus
K_D dilatometer horizontal stress index
K_o in situ stress ratio
L length of the pressuremeter membrane
M one-dimensional compression modulus
n modulus exponent
N SPT blow count
p' mean effective stress
P cavity pressure
P_A total stress at the end of unloading
P_B total stress at the closure of cycle
P'_o effective horizontal in situ stress
q axial pressure, q is $(\sigma'_1 - \sigma'_3)/2$
q_u deviatoric stress at failure
R parameter from the experimental curve
S_d pressuremeter loading slope in logarithmic plot
$S_{d\infty}$ pressuremeter loading slope in logarithmic plot corresponding to L/$D = \infty$
S_{ud} pressuremeter unloading slope in logarithmic plot
$S_{ud\infty}$ pressuremeter unloading slope in logarithmic plot corresponding to L/$D = \infty$
s'_{avg} average effective in situ stress
s'_o in situ mean effective stress on horizontal plane
α parameter from the experimental curve
β empirical undimentional factor
ε cavity strain
ε_a axial strain in triaxial compression test
ε_A shear strain at cavity wall at the end of unloading
ε_B shear strain at cavity wall at the closure of cycle
ε_{max} maximum cavity strain
ε_r radial strain in triaxial compression test
γ shear strain
$\Delta\gamma$ measured shear strain amplitude
γ^c corrected shear strain (average shear strain amplitude around the cavity)
υ Poisson's ratio
ϕ' effective internal friction angle
ϕ'_{cv} critical state internal friction angle
ϕ'_{ps} plane strain internal friction angle
ϕ'_{tx} triaxial internal friction angle
σ'_{ho} effective in situ horizontal stress
σ'_p plastic cavity pressure
σ'_r effective radial stress
σ'_θ effective circumferential stress
σ'_1 major principal effective stress (triaxial test)
σ'_3 minor principal effective stress (triaxial test)
ψ dilation angle