R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

Recent Developments in In-situ Testing of Soils
Dévelopements Récents dans les essais In-situ des Sols

XI ICSMFE
SAN FRANCISCO (1985)

SYNOPSIS This report presents comments and data from current research in the area of in-situ testing of soils at UBC. Results and comparisons include the CPT, research DMT, seismic-cone, and cone-pressuremeter.

INTRODUCTION

In-situ test methods can be divided into two basic groups (Campanella and Robertson, 1982);

i) logging methods, and
ii) specific test methods.

The logging methods are usually penetration type tests and are usually fast and economic, and usually provide qualitative estimates, based on empirical correlations, of various geotechnical parameters. Specific test methods are usually more specialized and, therefore, often slower and more expensive to perform than the logging methods. The two basic groups are usually complimentary in their use. The logging method is best suited for stratigraphic logging and preliminary evaluation of soil parameters. The specific test methods are best suited for use in critical areas, as defined by the logging methods.

New test methods, such as the cone-pressuremeter and the seismic cone combine these groups to produce tests that can both log the soil profile as well as measure specific soil parameters in one operation. These newer tests, which will be discussed in later sections, offer particular advantages in areas such as off-shore geotechnical investigations where cone equipment is already in use and where the need for better specific soil data is also required.

The following sections will outline some of the recent developments made in the area of in-situ testing of soils at UBC.

CONE PENETRATION TEST (CPT)

The piezometer cone penetration test is becoming increasingly more popular as an in-situ test for site investigation and geotechnical design. As a logging tool this technique is unequalled with respect to the delineation of stratigraphy.

Recent publications by de Ruiter (1982) and Campanella and Robertson (1981, 1982) have highlighted the importance of equipment design and procedure related to accuracy and repeatability of results obtained using the CPT. One of the major problems with cone designs has been the difficulty in combining ruggedness with adequate sensitivity for penetration testing in soft fine grained soils.

Little has been published concerning the accuracy of various cone designs. De Ruiter (1982) gave data on the observed accuracy of Fugro cones with typical values of about 1% accuracy of full scale capacity. Therefore, for a 1000 bar (10 tonf.) cone, the accuracy is about 10 bar. Most normally consolidated fine grained soils to depths of 30 m have cone bearings less than 10 bar. This does not necessarily mean that all data below a bearing of about 10 bar is not meaningful, since the hysteresis, non-linearity and repeatability error are very much less than they are at full scale. However, at low load the zero offset error is most critical. Therefore, the size of the zero offset error must be confirmed as minimal by recording the zero load at the start and end of each sounding to have meaningful measurements in soft clays and silts.

A major cause of zero load error at the end of a sounding can be caused by temperature effects. The initial zero before a sounding is usually carried out at room or air temperature, which may be significantly higher than the temperature in the ground. Good temperature compensation can limit the variation of load cell output with changes in temperature to as little as 0.05% of full scale output over the normal expected temperature range. However, the temperature sensitivity of load cells can vary widely for different cones, even for cones from the same manufacturer. The best way to accommodate the temperature zero shift is to calibrate the cone against temperature and to continuously monitor temperature in the cone and to correct all data as a function of temperature. These corrections are easily accommodated in a computer based data acquisition system.

Campanella et al. (1982) have suggested that all cone data be corrected for unequal end area effects and reported as total stress tip resistance, as follows:

R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

$$q_T = q_c + u_T(1-a) \qquad (1)$$

where q_T = total stress tip resistance,
u_T = total pore pressure during penetration, measured behind the tip,
a = net area ratio.

In sand, the corrections for both temperature and pore pressure effects are usually insignificant (i.e. less than 0.5% of the measured values). However, in soft fine grained soils, these corrections can be significant (greater than 30%).

These problems of accuracy and pore pressure effects for cone testing in soft, fine grained soils may explain the large scatter in published data concerning the cone factor (N_k) for the determination of undrained shear strength, c_u.

The N_k values based on existing experience will change somewhat as all cone resistance values become corrected to q_T.

An alternative approach for the calculation of c_u is the use of the excess pore pressure (Δu) generated during cone penetration. During cone penetration in soft, fine grained soils, the pore pressures generated can be very large. Typically, the pore pressure may represent 80% of the measured tip resistance. The pressure transducer is recording pore pressures close to its design capacity and thus, the pore pressure measurements are often very accurate (i.e. better than 1% of measured value). Therefore, estimates of soil parameters, such as c_u, will inherently be more accurate using the pore pressure data, as opposed to the tip resistance.

It is possible to estimate c_u from the excess pore pressure (Δu) using cavity expansion theories. The excess pore pressure is defined as follows:

$$\Delta u = u_T - u_o \qquad (2)$$

where u_T = total pore pressure,
u_o = equilibrium pore pressure (often assumed as hydrostatic).

The measured excess pore pressure is dependent on the location of the pore pressure element on the cone as well as the stiffness ratio (G/c_u) and the sensitivity of the soil. Low values of stiffness ratio apply to highly plastic clays (PI > 80) which tend to generate lower pore pressures during cone penetration. High values of stiffness ratio apply to low plastic clays and silts (PI ≈ 15) which tend to generate high pore pressures. The excess pore pressure tends to increase with increasing soil sensitivity and decrease with increasing overconsolidation ratio. A semi-empirical solution was proposed by Massarch and Broms (1981) based on cavity expansion theories but including the effects of overconsolidation and sensitivity by using Skempton's pore pressure parameter at failure (A_f). Charts illustrating this approach are given in Fig. 1 for excess pore pressures behind the cone tip and on the face. Approximate values for A_f can be estimated from the following:

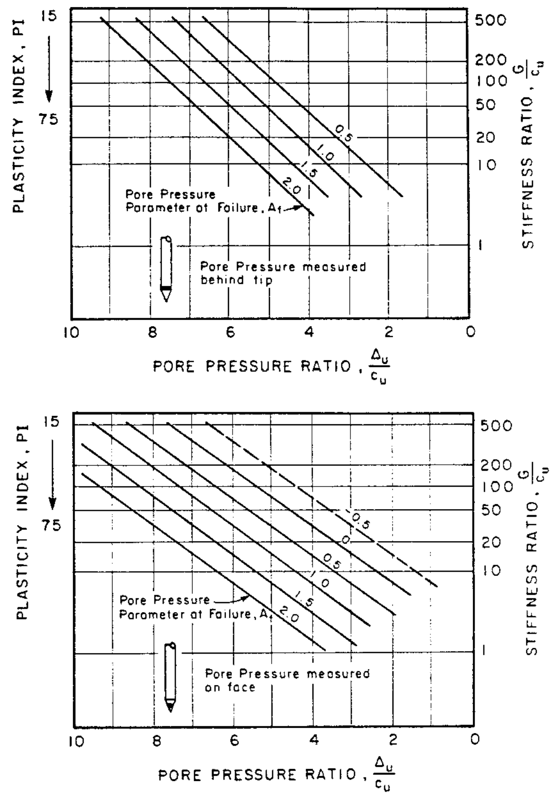

Fig. 1. Method to obtain c_u from excess pore pressure measured during CPT.

Saturated clays:	A_f
Very sensitive to quick	1.5-3.0
Normally consolidated	0.7-1.3
Lightly overconsolidated	0.3-0.7
Highly overconsolidated	-0.5-0

Details of how to estimate sensitivity and overconsolidation ratio from cone data is given by Robertson and Campanella (1983). A knowledge of the plasticity index (PI) would provide a guide to the stiffness ratio (Ladd et al., 1977) as shown in Fig. 1.

Fig. 2 shows the comparison between the measured c_u from field vane and the predicted c_u using Δu measured on the face and behind the tip for the normally consolidated clayey silt (PI = 15%) at the McDonald's Farm site. Using Fig. 1 applied to cone data from sites around the world, the authors have found that good predictions of c_u can be obtained. However, the pore pressure element location is important.

Additional data has been collected recently at several different sites with the pore pressure element located behind the tip and on the face of the tip. Fig. 3 illustrates the results of these measurements in a variety of different soil types. In normally consolidated insensitive clays and silts, where large positive pore pressures are generated during

R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

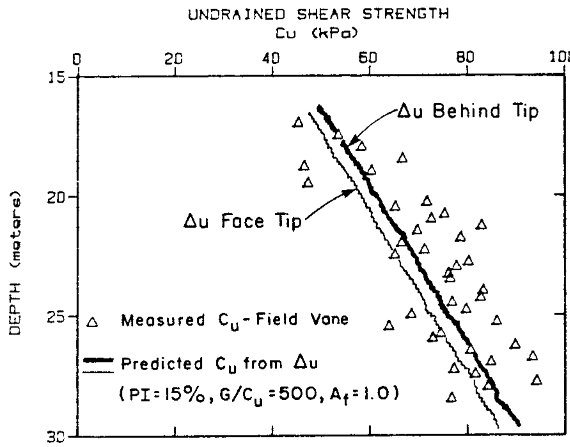

Fig. 2. Comparison of Measured c_u and Predicted c_u Using Δu Measured on Face and Behind Tip.

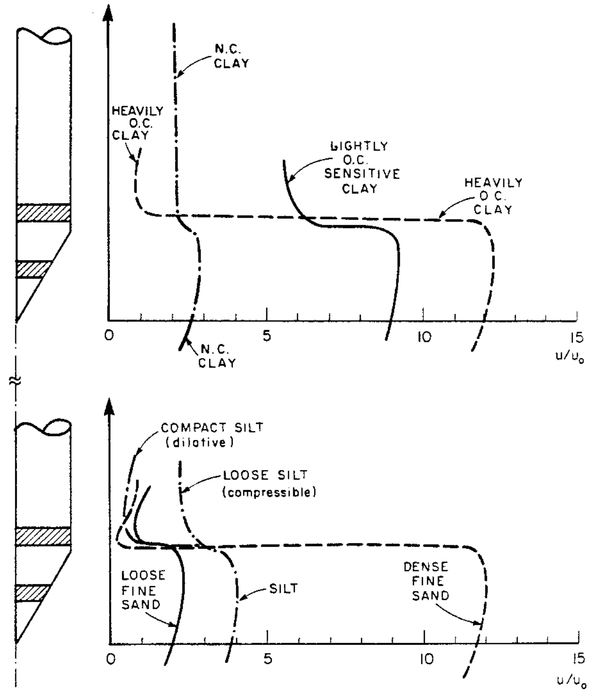

Fig. 3. Conceptual pore pressure distribution in saturated soil during CPT based on field measurements.

shear, pore pressures measured on the face of the tip are usually about three times larger than the equilibrium pore pressure (u_o) and about 15% larger than pore pressures measured immediately behind the tip. As the overconsolidation ratio increases in clays and silts, the excess pore pressure on the face of the tip increases. This is because the area on the face of the tip is in a zone of maximum compression and shear, unlike the area immediately behind the tip which is in a zone of normal stress relief. Both areas have large shear stresses but the large normal stresses dominate the pore pressure response on the face, whereas the large shear stresses dominate the response behind the tip. Pore pressures are generated in saturated soils because of increases in both normal stresses as well as shear stresses. Increases in normal stresses will always generate positive pore pressures whereas, increases in shear stresses can generate increases or decreases in pore pressures depending on the stress history and sensitivity of the deposit. As the overconsolidation ratio increases pore pressures tend to decrease during shear, but the increased cone bearing causes a larger increase in normal stresses and thus larger pore pressures on the face of the tip. Also, as the sensitivity of a soil increases, the pore pressures increase during shear.

Pore pressures can also be generated in silty, fine sands during cone penetration at the standard rate of penetration (2 cm/sec). In these deposits, the pore pressures generated on the face of the tip can reach very large values when penetrating dense sand due to the extremely high increases in normal stresses on the face (q_c > 100 bar). Because of the dilatant nature of dense sands, the pore pressures generated immediately behind the tip can be less than the equilibrium value (u_o). Little or no excess pore pressures are generated in coarse sands because of their high permeability.

Little quantitative success has been achieved to date using the pore pressures measured at one location to correlate with stress history for all clays. This is because no unique relationship exists between pore pressure and OCR for all clays (see Fig. 1) since the stiffness ratio and sensitivity effect the generated pore pressures. However, a review of Fig. 3 illustrates the potential to correlate the <u>difference</u> between the generated pore pressures measured on the face and behind the tip with stress history for clays. This is an area of current research at UBC.

Because of the tendency for low or negative pore pressures measured behind the tip in insensitive, overconsolidated clays and silts, it is recommended to use the pore pressure measured on the face for determination of c_u in these soils.

It has also been observed that for thin pore pressure elements located immediately behind the tip very small pore pressures (less than u_o) have been recorded. These pore pressures have sometimes been smaller than those recorded with thicker elements located in the same position. It is believed that thin pore pressure elements can sometimes measure low pore pressures due to a shadow effect from a cone tip slightly larger in diameter. Thus, the O.D. of the cone tip should be identical or less than the O.D. of the porous element and friction sleeve by about 0.25 mm.

R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

FLAT PLATE DILATOMETER TEST (DMT)

A recent in-situ test method that is increasing in popularity is the Flat Plate Dilatometer Test (DMT). The DMT is essentially a logging test where estimates of soil parameters are based on empirical correlations. Recent research at UBC involving the DMT has centered on the development and use of a research flat plate dilatometer. The research dilatometer is identical in size, shape and operation as the Marchetti design except for the passive measurement of the following:

i) pore water pressure at the center of the moving diaphragm,
ii) deflection at the center of the diaphragm,
iii) total pressure activating the diaphragm,
iv) verticality of the dilatometer during penetration, and
v) the penetration force for the blade of the dilatometer.

Typical results from the research dilatometer for tests in a sand and soft clay deposit are shown in Fig. 4. The results show the total pressure and pore pressure versus deflection at the center of the diaphragm corrected for diaphragm stiffness.

The results in both the sand and clay deposits show a remarkable similarity in shape to the pressure expansion curves obtained from self-boring and push-in pressuremeter probes. For the tests in sand, there are no excess pore pressures generated during both the penetration and expansion phases of the test. The lift-off pressure, P_o, for the test in sand is higher than the expected in-situ total horizontal stress. Several unload-reload cycles were performed during the expansion phase of the test. The slope of the unload-reload cycle was considerably steeper than the slope of the straight expansion phase. The membrane returned to its closed position at a pressure equal to the in-situ equilibrium water pressure. Both these characteristics are seen in pressuremeter testing in sand.

Experience at UBC has shown (Campanella and Robertson, 1983) that the moduli (E_d) derived from the standard DMT is close to the Young's moduli at approximately 25% of the failure load (E_{25}). Experience from pressuremeter testing (Hughes and Robertson 1984) has shown that the moduli from unload-reload cycles is close to the initial 'elastic' moduli, which is somewhat higher than the E_{25}.

The moduli obtained from the standard DMT appears to provide a reasonable moduli for design in sands for the following reasons:

i) stress level during the test is higher than in-situ stress, therefore the moduli is somewhat stiffer,

ii) strain level during 1 mm expansion from P_o to P_1 is large, therefore moduli is somewhat softer.

These two factors when combined appear to produce a reasonable Young's moduli for most design purposes in sand.

In general, the research DMT has shown that for tests in sands, the results are dominated by high effective stresses and little or no excess pore pressures are generated.

The research DMT in soft clay shows that very large pore pressures are generated and that the effective stresses during the penetration and expansion phases of the test are very small. The effective stress adjacent to the membrane appears to remain unchanged throughout the pressure expansion and unloading phase of the test. The increase in total stress applied to the membrane is equally matched by an increase in the pore pressure.

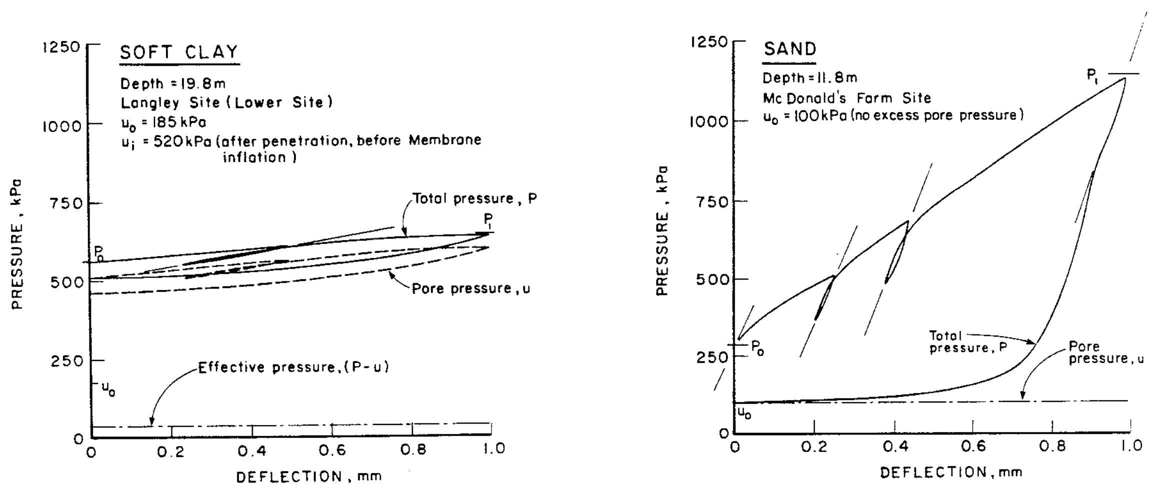

Fig. 4. Typical results from Research DMT in sand and soft clay deposits.

R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

Cavity expansion theories have shown that a limit pressure exists for undrained cavity expansion in soft clays. It appears that the penetration process during a DMT in soft clay is sufficient to induce pressures equivalent to some limit pressure. Because of the stress relief phenomena due to the location of the membrane relative to the blade tip, creep and pore pressure dissipation, the lift-off pressure, P_o, is less than the limit pressure. However, the expansion of 1 mm is sufficient to re-establish the limit pressure. The shape of the pressure expansion curve is therefore remarkably similar to the latter section of the pressure expansion curve from pressuremeter tests in soft clay.

The fact that both P_o and P_1 are dominated by the pore pressure response of the clay and are related to the limit pressure for some form of cavity expansion, it is not surprising that they can be related to parameters such as undrained shear strength, stiffness and stress history.

It is interesting to note that the total pressure recorded as the membrane returned to its closed position is the same as the initial pore pressure at the start of the test. This is due to the fact that the effective stress appears to remain unchanged throughout the test. Therefore, it may be possible to estimate the initial pore pressure around the membrane by recording the closing pressure during a standard DMT.

Preliminary data collected with the research DMT indicates that the standard data (P_o, P_1) are not significantly effected by slight non-verticality of the blade, for penetration depths less than 15 m.

SEISMIC CONE

A new device that has been under development at UBC combines a piezometer, friction, bearing cone with a set of miniature seismometers built into the cone (Campanella & Robertson, 1984). The bearing, friction and pore pressure measurements are used to log the stratigraphy of a site during penetration and a downhole seismic technique is performed during pauses in the penetration to provide a profile of the in-situ shear wave velocity, V_s and hence the in-situ dynamic shear modulus, G_{max}.

A schematic layout of the downhole seismic cone penetrometer test is shown in Fig. 5. To obtain the measurement of dynamic shear modulus a triaxial package of small, rugged seismometers has been incorporated into the cone penetrometer. The seismometers are Geospace GSC-14-L3 with a standard natural frequency of 28 Hz. The horizontal seismometers are oriented in radial and transverse orientation to the signal source. A suitable seismic signal source preferentially generates large amplitude shear waves with little or no measured compressional wave components. The design and construction of the seismometer carrier provides a snug seating for the triaxial package. The method of advancing the cone penetrometer provides

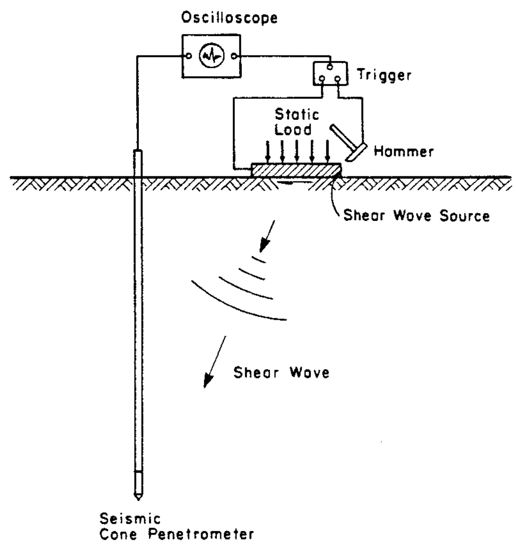

Fig. 5. Schematic layout of Downhole Seismic Cone Penetrometer test.

continuous firm mechanical contact between the seismometer carrier and surrounding soil. This allows excellent signal response. In addition, seismometer orientation can be controlled and accurate depth measurements obtained.

Fig. 6 shows the profile of dynamic shear modulus, G_{max} (measured every half meter), obtained using the seismic cone at the UBC research site at McDonalds Farm. The cone data can be interpreted to give a continuous prediction of soil type and shear strength and the seismic data interpreted to give a near continuous profile of soil modulus. Therefore, the reasonably rapid procedure using the seismic cone allows a fast determination of soil type, strength and stiffness in one sounding. Hole verticality is monitored throughout the sounding with a small slope sensor installed in the cone.

CONE PRESSUREMETER

Another new device under development at UBC combines a piezometer, friction, bearing cone with a pressuremeter element. The pressuremeter element of the probe is designed along similar lines to the pressuremeter element in self-boring probes. The cone penetration can be stopped at selected intervals and a pressure expansion test performed using the pressuremeter element. The pressuremeter test is referred to as a full-displacement pressuremeter test (FDPMT) since the cone produces a full-displacement installation technique. Test results have shown (Hughes and Robertson, 1984) that the shear moduli obtained from unload-reload cycles using the FDPMT in sands is the same as those obtained from self-boring pressuremeter tests (SBPMT) (Fig. 6). The moduli in sand

R.G. CAMPANELLA, P.K. ROBERTSON, D.G. GILLESPIE and J. GRIEG
Civil Engineering Dept., University of British Columbia, Vancouver, B.C., Canada

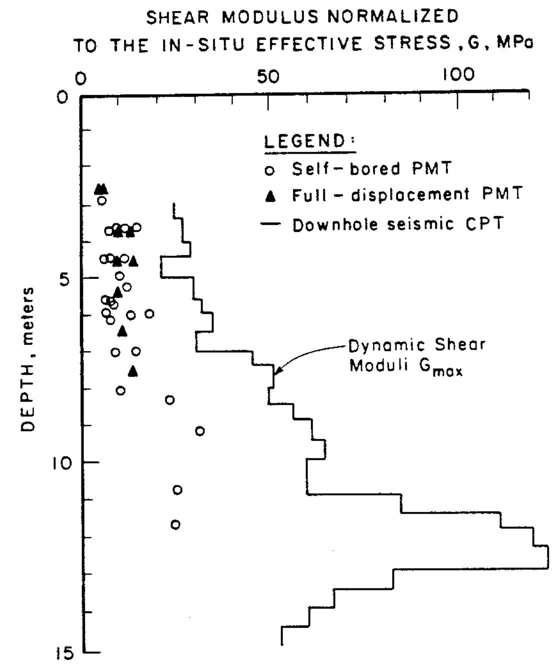

Fig. 6. Comparison of Shear Moduli from downhole seismic CPT, Self-bored and Full-displacement Pressuremeter tests in sand at McDonald's Farm.

from SBPMT and FDPMT are approximately equal and about one-third of the dynamic moduli derived from seismic data.

With the electric cone located in front of the pressuremeter, the soil can be logged and the appropriate location for pressuremeter testing selected. The pressuremeter data can be used to obtain moduli. It is also expected that the combined data from the cone and the pressuremeter test will enable an estimate of in-situ stress to be made in sands.

The full-displacement pressuremeter can also be looked upon as a modelling tool to provide a direct physical model of driven displacement piles. The resulting pressure displacement data has been successfully used to generate lateral load displacement (P/Y) curves (Robertson et al., 1983).

In principle, the cone pressuremeter is similar to the DMT in that it is a penetration tool with a lateral expansion. The standard DMT has the advantage that it is extremely simple and relatively inexpensive. However, the cone-pressuremeter incorporates the already well established experience of the cone plus the axisymmetric expansion of the pressuremeter, which is fundamentally easier to understand and analyse, even though much of the future interpretation may be semi-empirical.

CONCLUSIONS

Advances have been made in the use and understanding of existing logging techniques (CPT and DMT). New test methods that can both log the soil profile as well as measure specific soil properties are an area of current research.

ACKNOWLEDGMENTS

The assistance of the Natural Sciences and Engineering Research Council of Canada is much appreciated. The work of T. Rice, C. Tsang and N. Laing is also appreciated.

REFERENCES

Campanella, R.G. and Robertson, P.K., (1981), "Applied Cone Research", Symposium on Cone Penetration Testing and Experience, Geotechnical Engineering Division ASCE, Oct., pp. 343-362.

Campanella, R.G. and Robertson, P.K., (1982), "State-of-the-Art in In-situ Testing of Soils: Developments since 1978", Engineering Foundation Conference on Updating Subsurface Sampling of Soils and Rocks and Their In-situ Testing, Santa Barbara, California, January, 23 pgs.

Campanella, R.G. and Robertson, P.K., (1983), "Flat Plate Dilatometer Testing: Research and Development", Proceedings First International Conference on the Flat Plate Dilatometer, Edmonton.

Campanella, R.G. and Robertson, P.K., (1984), "A Seismic Cone Penetrometer to Measure Engineering Properties of Soil", Submitted to Society of Exploration Geophysics Meeting, Atlanta, Dec.

Campanella, R.G., Gillespie, D. and Robertson P.K. (1982), "Pore Pressures during Cone Penetration Testing", Proceedings of the 2nd European Symposium on Penetration Testing, Amsterdam, Vol. 2, pp. 507-512.

de Ruiter, J., (1982), "The Static Cone Penetration Test State-of-the-Art Report", Proceedings of the Second European Symposium on Penetration Testing Amsterdam, Vol. 2, pp. 389-405.

Hughes, J.M.O. and Robertson, P.K., (1984), "Full Displacement Pressuremeter Testing in Sand", Submitted for publication in Geotechnique.

Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F. and Poulos, H.G., (1977), "Stress-Deformation and Strength Characteristics", Proceedings, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 11, pp. 421-494.

Massarch, K.R., and Broms, B.B., (1981), "Pile Driving in Clay Slopes", Proceedings International Conference Soil Mechanics and Foundation Engineering, Stockholm.

Robertson, P.K. and Campanella, R.G. (1983), "Interpretation of Cone Penetration Tests - Part I and II", Canadian Geotechnical Journal, Vol. 20, No. 4, pp. 718-745.

Robertson, P.K., Hughes, J.M.O., Campanella, R.G. and Sy, A., (1983), "Design of Laterally Loaded Piles Using a Pushed-in Pressuremeter", Symposium on the Design of Lateral Loading of Piles and Pile Groups, ASTM STP 835, Missouri, June.