FACTORS AFFECTING THE INTERPRETATION AND ANALYSIS OF FULL-DISPLACEMENT PRESSUREMETER TESTS IN SANDS

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A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

in

THE FACULTY OF GRADUATE STUDIES

Department of Civil Engineering

We accept this thesis as conforming

to the required standard

THE UNIVERSITY OF BRITISH COLUMBIA

January 1991

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ABSTRACT

The Full-Displacement Pressuremeter (FDPM) Test is one in which a pressuremeter is installed in the soil by pushing it behind a conical tip. Earlier work had indicated that the unload-reload modulus measured with the FDPM was very similar to that obtained from self-boring pressuremeter (SBPM) testing. It had also been suggested that if the pressuremeter was capable of sufficient expansion, the interpreted soil properties would be those of the soil beyond the zone of disturbance. This study examined the factors affecting the measurement, analysis and interpretation of soil properties from FDPM pressure-expansion curves in sands with emphasis on the unload-reload modulus.

The effects of equipment design and dimensions, installation method and of test procedure on the analysis and interpretation of lateral stress, shear strength and stiffness were studied during laboratory and field evaluation of two prototype FDPMs. The overwhelming importance of instrument dimensions and tolerances on the test results was clearly shown. Movements of a fraction of a millimetre can have a large effect on the measured lateral stress and stiffness. Test procedures were also shown to have a large effect on the data obtained. It was demonstrated that rate effects became important in pressuremeter tests involving expansion to large strains and a stress-strain strain rate concept was proposed to aid in the understanding of these effects.

Theories developed for the interpretation of shear strength of sands from SBPM tests were shown to be inapplicable to the interpretation of FDPM test results. The unload-reload modulus was shown to be an indicator of soil stiffness but the effects of stress level and degree of unloading have to be considered when attempting to derive a stiffness for design. A rational approach to the interpretation of modulus was presented and it was shown that unload-reload moduli from both SBPM and FDPM could be interpreted using the same approach. The need for standardising the equipment design, testing procedures and methods of analysis and interpretation was shown.
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I am grateful for the financial support provided to me during my four and a half years at UBC by:

- B.C. Science Council GREAT Grant from 1984 to 1987
- University of B.C. University Graduate Fellowship, Summer 1987.

In addition, I wish to acknowledge the support of Foundex Explorations (the industry partner for the GREAT Grant) for providing the drill rig for SBPM testing and Fugro for making their prototype FDPM available to me. The technical staff of the Department of Civil Engineering were of great assistance, particularly Art Brookes, Jim Greig, Ian Hers, Glenn Jolly and Harald Schremp. Dr. John Hughes gave generously of his time both in the field and in discussion of both theoretical and practical aspects of pressuremeter testing. Dr. Peter Robertson also helped in the field and was a source of ideas. Thanks are also due to Drs Byrne, Campanella, and Vaid for their critical review of the contents of the thesis. Lastly, I wish to thank the people at Hardy BBT Limited for their help during preparation of this thesis.
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<thead>
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<th>Description</th>
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<tbody>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
</tr>
<tr>
<td>CPTU</td>
<td>Piezo-Cone Penetration Test</td>
</tr>
<tr>
<td>DMT</td>
<td>Flat Plate Dilatometer</td>
</tr>
<tr>
<td>FDPM</td>
<td>Full-Displacement Pressuremeter</td>
</tr>
<tr>
<td>HPM</td>
<td>Hughes Self-Boring Pressuremeter</td>
</tr>
<tr>
<td>PIPM</td>
<td>Push-in Pressuremeter</td>
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<tr>
<td>SBPM</td>
<td>Self-Boring Pressuremeter</td>
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<tr>
<td>$E_{PM}$</td>
<td>Pressuremeter Modulus (Menard Rules)</td>
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<td>e</td>
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<tr>
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<tr>
<td>$G_{ur}$</td>
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<tr>
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<tr>
<td>$G_{max}$</td>
<td>Small Strain Shear Modulus</td>
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<tr>
<td>$p_a$</td>
<td>Atmospheric Pressure</td>
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<tr>
<td>$p_o$</td>
<td>In-situ horizontal total pressure</td>
</tr>
<tr>
<td>$p_{o}'$</td>
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<tr>
<td>$P_t$</td>
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<tr>
<td>$R_i$</td>
<td>Radius of Influence</td>
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<tr>
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<tr>
<td>r</td>
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<td>$\gamma$</td>
<td>Shear Strain</td>
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<tr>
<td>$\Delta \gamma$</td>
<td>Small change in shear strain</td>
</tr>
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<td>$\epsilon_c$</td>
<td>Cavity Strain = $\Delta r/r_o$</td>
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<tr>
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<tr>
<td>$\phi$</td>
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<tr>
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<td>Constant volume friction angle</td>
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<tr>
<td>$\phi_{CYL}$</td>
<td>Friction angle from cylindrical cavity expansion</td>
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<tr>
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<td>Friction angle from spherical cavity expansion</td>
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<tr>
<td>$\nu$</td>
<td>Dilation angle</td>
</tr>
<tr>
<td>$\tau$</td>
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1.0 INTRODUCTION

1.1 SITE INVESTIGATION

Traditionally, site investigation has consisted of drilling boreholes and sampling the soil in order to delineate the soil stratigraphy. Undisturbed sampling has been carried out in order to obtain samples which are returned to the laboratory and tested to obtain soil properties for use in engineering design.

The major advantages of this process are that soil samples can be visually classified and can be tested under carefully controlled boundary conditions. Laboratory tests can be designed to simulate the changes in stress or strain conditions to be imposed by the particular engineering work under consideration.

The major disadvantages of this type of site investigation are that:

- standard drilling and sampling techniques typically consist of sampling at discrete intervals. This could result in important stratigraphic features being missed. Continuous sampling is slow and thus becomes expensive and can only be justified on special projects;
- soil behaviour is predicted on the basis of a small number of single element tests;
- it is impossible to obtain a truly undisturbed sample;
- as the in situ stress conditions are unknown it is impossible to return the sample to its original stress state.

The problems of sample disturbance, especially in sands, and the need for rapid, economical assessments of soil consistency and strength led to the development of in-situ tests.
1.2 IN SITU TESTING

The most widely used in-situ test is the Standard Penetration Test (SPT) which has been in existence for over fifty years. This test consists of driving a split-spoon sampler a distance of 0.45 m beyond the base of a borehole using a standard hammer mass of 63.5 kg falling a standard height of 0.76 m. The number of blows required to drive the split-spoon the final 0.3 m is termed the Standard Penetration Resistance, N-value or Blow-Count.

The ease and economy of the SPT have led to its wide acceptance and many empirical correlations have been derived between N-value and soil properties or between N-value and soil behaviour, e.g. the Terzaghi and Peck (1967) relationship between N-value and allowable bearing pressure on sand. The SPT N-value remains the primary index for assessing the liquefaction susceptibility of sands.

In recent years, the SPT has come under great scrutiny with the result that the many potential sources of error are now much better understood. In addition, the advent of micro-processors has offered the possibility of constructing robust tools which can measure a number of different parameters simultaneously and repeatably, independent of the operator's skill. Consequently, there has been a great increase in the number of in situ testing tools available in recent years.

In situ tests can be divided into two main groups:

1) logging methods;
2) specific test methods.
Logging methods are primarily directed towards delineation of the stratigraphy but also allow estimation of strength and stiffness parameters by empirical correlations. Specific test methods are designed to measure a specific soil property such as strength, stiffness or permeability for use in engineering design and ideally should be carried out after critical strata have been identified using a logging tool.

In recent years, interest has been expressed in combining logging and specific test capability into one instrument, especially on offshore geotechnical investigations where mobilization costs are high. This thesis describes an assessment of one such tool, the Cone Pressuremeter (CPM). This instrument combines the most widely accepted logging tool, the piezometer friction cone (CPTU), with the pressuremeter, the specific test method which many researchers feel comes closest to allowing a direct determination of in situ soil properties. The pressuremeter unit of the CPM is pushed into the ground behind a conical tip of identical diameter. Because of the method of installation, the pressuremeter is termed a Full-Displacement Pressuremeter (FDPM).

The cone-pressuremeter seems to offer the following desirable features:

- it can be inserted in a repeatable manner, independent of the operator's skill;
- when the pressuremeter is mounted behind an electronic friction cone, the pressuremeter test location can be accurately selected;
- the requirement for two separate systems for pressuremeter and cone soundings is removed, resulting in significant cost reductions particularly in offshore site investigation;
- the modulus determined from an unload-reload cycle conducted during a pressuremeter test appears to be insensitive to the degree of disturbance around the probe and so can be determined from the FDPM test (Hughes and Robertson, 1985);
- it can be used as a model displacement pile to produce P-Y curves for design of laterally-loaded piles.
1.3 OBJECTIVES OF THIS STUDY

The objective of this research was to study the CPM in order to better understand the effects of equipment design, installation method and test procedure on the analysis and interpretation of the results obtained. Seismometers were incorporated in the instrument permitting measurement of downhole shear wave velocity from which the small strain shear modulus of the soil could be derived. The major focus of the investigation was the Full-Displacement Pressuremeter component of the instrument as the Seismic CPTU (Campanella and Robertson, 1984) is relatively well understood in comparison to the FDPM. In addition, methods of analysis and interpretation of FDPM tests were examined. The bulk of the study concerned the factors affecting the interpretation of FDPM tests.

1.4 SCOPE OF WORK

The work described herein formed part of a larger project at the University of British Columbia (UBC) to design, develop and produce a Seismic Cone Pressuremeter specifically with a view to its use for offshore site investigation. In May 1984 the project was funded by the Natural Science and Engineering Research Council of Canada (NSERC) for a three year period as part of the Cooperative Research and Development Programme. The proposed concept was described by Campanella and Robertson (1986). Design of the UBC Seismic Cone Pressuremeter was carried out by a team under the supervision of Dr. R.G. Campanella. The laboratory and field evaluation of the instrument and of several prototypes was carried out by the author as was the interpretation of the results.

Development of the UBC Seismic Cone Pressuremeter (SCPM) began in 1984 but the concept had been proposed by Robertson (1982). Concurrently with the work at UBC, Fugro B.V. of the Netherlands was also developing a Cone Pressuremeter in conjunction with Cambridge In-Situ of England. Recognizing
the common interests of the two organizations, Fugro sent a prototype of their pressuremeter element to UBC for evaluation of its mechanical and electronic design and of its performance in the field at a well-documented research site. This gave the UBC design team an opportunity to gain valuable insight into some of the potential design and deployment problems posed by such an instrument prior to finalizing the UBC design and allowed the writer to gain experience of full-displacement pressuremeter testing primarily in sands.

At the same time as the evaluation of the Fugro instrument was being conducted, the UBC instrument was under design and development. Seismic CPTU testing was already the subject of research at UBC (Rice, 1984; Laing, 1985). The task of the design team was to combine this known technology with a pressuremeter. A modular approach was adopted and the writer undertook laboratory and field evaluations of the components of the pressuremeter module and preliminary field evaluation of the combined seismic cone pressuremeter. The laboratory and field evaluations were undertaken to investigate the effects of equipment design, instrument tolerances, and of test procedures on the parameters measured or interpreted.

Traditionally, the results of pressuremeter tests obtained with a wide variety of instruments are interpreted using methods based on theories of cylindrical cavity expansion. The theories currently in use are briefly reviewed and the interpretation of unload-reload modulus in sands is examined in some detail. The application of these interpretation methods to the results of FDPM testing in sands is then examined.
2.0 THE CPT, PRESSUREMETER AND THE CONE PRESSUREMETER

2.1 STATIC CONE PENETRATION TESTING

The static cone penetrometer was originally developed in the Netherlands in 1930 by P. Barentsen (Broms and Flodin, 1988) and has been widely used in Europe since then. It is now becoming accepted in North American practice. The standard cone has a 60° conical tip with a plan area of 10 cm$^2$ and a friction sleeve immediately behind the tip which is 150 cm$^2$ in area. The instrument is pushed into the ground and the force on the tip of the cone and the force on the friction sleeve are measured. These measurements are used to calculate bearing or penetration stress, $q_c$, and stress on the friction sleeve, $f_r$. These parameters are used in the determination of the soil stratigraphy and properties.

The original cone penetrometer was a mechanical device for which a double rod system was required to allow the tip to be advanced independently of the sleeve followed by advancement of the tip and sleeve together. This simple test is relatively economical but is slow, provides poor resolution of stratigraphy in soft soils and is subject to inaccuracy due to soil particles impeding free movement of the moving parts.

The electronic friction cone incorporates independent load cells to allow simultaneous measurement of penetration resistance and friction (de Ruiter, 1971). A typical configuration is shown in Figure 2.1. The signals from the load cells are transmitted to the surface along a cable or acoustically through the rods. At the surface, the data can be output in analogue form on a strip chart recorder or can be digitized for processing by computer. Alternatively, the cone data can be stored in the cone and can be
Figure 2.1  Typical Friction Cone Designs (adapted from Schaap and Zuidberg, 1982).
down-loaded after withdrawal of the instrument. The electronic friction cone provides readings typically at 2.5 cm or 5.0 cm intervals and thus allows detailed delineation of stratigraphy. A typical CPT profile is shown in Figure 2.2.

Interpretation of CPT results for soil type is usually based upon correlation of $q_c$, $f$, and the friction ratio, $f/q_c \times 100$ (%), with values obtained in other well-defined soil deposits. It is still desirable to establish site-specific correlations by drilling a borehole adjacent to a CPT sounding but sufficient experience is now being gained to allow correlation using published data from soil deposits of similar geological origin in other parts of the world. A very detailed treatment of CPT testing and interpretation in sands and clays was given by Robertson and Campanella (1983 a,b).

Cones are usually required to be capable of operating in dense soils as well as soft soils. The high load cell capacity required for the dense soils can result in poor resolution of stratigraphic detail being obtained in soft soils. A significant advance in the stratigraphic logging capability of the CPT has been the introduction of pore pressure measurements. The improved instrument is termed the Piezocone or CPTU. Penetration of fine grained soils at the standard rate of 2 cm per second tends to be undrained or partially drained depending on the permeability of the soil and so excess pore pressures are induced by the penetration of the cone. A properly designed and fully saturated piezometer element in the CPTU will sense these pore pressures and can respond very quickly to pore pressure changes. It is thus possible to distinguish between drained and undrained or partially drained cone penetration. By stopping penetration and monitoring dissipation of pore pressure, an indication can be obtained of the consolidation characteristics of the soil. This allows better definition of soil type and layering.

The CPTU is the subject of intensive research. Early attempts to interpret engineering properties such as soil strength were based on penetration resistance and friction measurements. The introduction of pore
**Figure 2.2. Typical CPT Profile**

- **Friction Resistance** ($F_C$ (BAR))
- **Bearing Resistance** ($Q_T$ (BAR))
- **Friction Ratio** ($R_F = F_C/Q_T$ (%))

**Soil Profile**:
- **Soft CLAY & SILT**
  - Coarse SAND, loose to dense with layers of fine Sand
- **Fine SAND, some silt**
- **Soft, normally consolidated clayey SILT**
  - Sand = 10%
  - Silt = 70%
  - Clay = 20%
  - L.L. = 38%
  - P.I. = 15%
  - $w_o = 35%$
  - $k = 8 \times 10^{-7}$ cm/sec
  - $C_C = 0.3$

1 BAR = 100 kPa = 1 kgf/cm² = 1 ton/ft²
pressure measurement has led to attempts to infer undrained shear strength and stress history from the excess pore pressures generated, and to determine consolidation characteristics from the dissipation curves. A detailed discussion of the current status of the CPTU is given by Campanella and Robertson (1988).

In summary, therefore, the CPT and CPTU offer the following advantages:

- a high degree of stratigraphic detail is possible with the almost continuous data obtainable from the electronic CPTU;
- the test is repeatable and operator independent;
- the test is rapid and the electronic output is amenable to rapid processing and analysis by computer.

Disadvantages are:

- the equipment has a fairly high capital cost;
- the equipment requires very careful attention to calibration and maintenance;
- no soil sample is obtained to allow visual classification of the soil.

2.2 PRESSUREMETER TESTING

2.2.1 Introduction

The pressuremeter is a balloon-like device which is inflated against the wall of a borehole. The pressure-expansion relationship is recorded and can be used to infer the engineering behaviour of the soil.
The principal attraction of the pressuremeter test is that the test models the expansion of an infinitely long cylindrical cavity, a simple boundary-value problem in mechanics. This offers the possibility that fundamental strength and deformation properties of the soil may be derived. Soil properties commonly derived are in situ stress, stiffness and shear strength. A brief description of the development of the pressuremeter (PM) is given below.

2.2.2 Pre-Bored or Menard Pressuremeter (PBPM).

The first reference to such a device was by Kogler in 1933 but Menard (Baguelin et al., 1972) was responsible for the development of the instrument into a useful engineering test. Menard Pressuremeters were in use by consulting engineers in France in 1957 and are now widely accepted in that country as the primary site investigation tool for foundation design.

The basic principles of the Menard pressuremeter are illustrated in Figure 2.3. The instrument is inserted into a pre-drilled borehole. The probe consists of a measuring cell which is inflated against the sides of the borehole by pumping a fluid, usually water, into it. The change in cavity volume is evaluated by recording the volume pushed into the probe. Guard cells, which are independent of the main measuring cell, are included in an effort to ensure that the cavity expands approximately cylindrically. An idealized pressure-expansion curve is shown in Figure 2.4.

Attempts were made [Menard (1957), Gibson and Anderson (1961)] to interpret the results of PBPM tests using the theory of cylindrical cavity expansion to derive the strength and deformation properties of the soil. An idealized elastic-plastic soil model was assumed. The portion of the curve OA in Figure 2.4 was taken to represent reloading of the borehole wall, $P_s$ to be the in-situ horizontal stress, and AB to be representative of "elastic loading" of the soil. At $P_t$, yielding was assumed to occur after which the
Figure 2.3  Schematic of the Menard Type PBPM (adapted from Baguelin et al., 1978).
Figure 2.4  Idealized Pressure Expansion Curve from Menard Type Prebored Pressuremeter Test (after Robertson, 1986).
curve tended towards the Limit Pressure. The Limit Pressure can be shown theoretically to be the maximum pressure possible during expansion of a cylindrical cavity.

Research carried out mainly at the Laboratoire Centrale des Ponts et Chaussees (LCPC) in France found that the test results departed substantially from the theoretical predictions. In particular, the effects of disturbance due to unloading of the wall of the borehole prior to insertion of the probe were recognized and so design methods were developed which used the pressuremeter results empirically or semi-empirically for foundation design.

Conventional interpretation of the PBPM test makes use of the following parameters:

- \( P_e \), the pressure at the start of the linear section of the curve;
- \( P_L \), the pressure when the cavity has been expanded to twice its initial volume;
- \( E_m \), a deformation modulus derived from the quasi-linear portion of the curve, AB.

The importance of standardisation of the insertion and test procedures so that tests carried out in similar soil types could be directly compared was also recognized. This was an attempt to ensure that the degree of disturbance was reasonably consistent. The quality of foundation design using the empirical design method is generally high provided the standard Menard equipment and procedures are used.

In an effort to overcome the problems of disturbance, the Self-Boring Pressuremeter (SBPM) was developed independently by the French (Baguelin et al., 1972) and the British (Wroth and Hughes, 1973). A very detailed account of the development and use of the French pressuremeters (both PBPM and SBPM) and of the empirical correlations used for foundation design is given in Baguelin et al. (1978).
2.2.3 Self-Boring Pressuremeter (SBPM)

The Self-Boring probe consists of a measuring cell on the outside of a thick-walled tube, the inner core housing a rotating cutter which breaks up the soil. The cuttings are removed by water or drilling mud exiting up the inside of the drill rods. By carefully controlling the drilling, it has been found that it is possible to insert the probe into the ground with limited disturbance to the soil (Wroth, 1982). A schematic of the SBPM developed at Cambridge, England by Hughes (1973) and subsequently modified by him is presented in Figure 2.5(a). More recently, the rotating cutter has been replaced in some pressuremeters by a jetting-tool in which fine jets of pressurized drilling mud are used to break up the soil (Hughes, 1982). A typical jetting arrangement is shown in Figure 2.5(b).

After installation, the instrument is inflated by injecting water (French version) or gas (British) into the expansion unit. The degree of expansion is measured either by monitoring the volume of water injected and assuming that the membrane expands as a right cylinder (French) or by measuring the deflection at the centre of the membrane using spring-loaded strain arms (British). The probes can also be equipped to measure the pore water pressure using a pressure transducer mounted on the membrane. The basic characteristics of the French and English SBPMs are presented in Table 2.1.

The results of SBPM tests differed greatly from those of the PBPM tests. An idealized SBPM test curve is shown in Figure 2.6. The S-shape, characteristic of the PBPM test, is absent. This is because the unloading and reloading of the borehole walls in the PBPM test has been largely eliminated. When the unloading due to predrilling is prevented by self-boring, the pressure-expansion curve is concave-upward and the pressure at a given volume is higher. Consequently, the empirical design rules developed for the PBPM cannot be used for the SBPM. However, correlations have been developed between PBPM and
(a) SELF-BORING PRESSUREMETER
IN CONVENTIONAL FORM
FOR LAND OPERATION

(b) SELF-BORING PRESSUREMETER
MODIFIED FOR INSTALLATION
BY JETTING

Figure 2.5. Schematic of the Hughes SBPM.
(adapted from Hughes, 1984)
Table 2.1 Comparison of French and English Self Boring Pressuremeters (after Robertson, 1982).

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>French</th>
<th>English</th>
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<tbody>
<tr>
<td>Inflation fluid</td>
<td>Water</td>
<td>Nitrogen</td>
</tr>
<tr>
<td>Membrane measurement</td>
<td>Increase in volume</td>
<td>Radial displacement</td>
</tr>
<tr>
<td>Membrane displacement monitor</td>
<td>Flowmeter</td>
<td>Feeler arms and electronic recorder</td>
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<tr>
<td>Construction</td>
<td>Modular</td>
<td>Single unit</td>
</tr>
<tr>
<td>Cutter drive</td>
<td>Hydraulic mounted on probe</td>
<td>Hydraulic from surface</td>
</tr>
<tr>
<td>Type of test</td>
<td>Mainly strain controlled</td>
<td>Mainly stress controlled</td>
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<tr>
<td></td>
<td>Stress controlled easy</td>
<td>Recently developed more complicated strain controlled</td>
</tr>
<tr>
<td>Effect of leak</td>
<td>Termination of test</td>
<td>Test may be continued</td>
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<tr>
<td>Temperature effects</td>
<td>Requires special precautions</td>
<td>Electronic equipment may be sensitive</td>
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<tr>
<td>Pore pressure measurement</td>
<td>Hydraulic</td>
<td>Electronic</td>
</tr>
<tr>
<td>Cutter geometry</td>
<td>Not adjustable during insertion</td>
<td>Adjustable during insertion</td>
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</tbody>
</table>
Figure 2.6. Idealized Pressure Expansion Curve from SBPM Test (after Robertson, 1986)
SBPM results which allow the use of the well-established empirical PBPM-based design methods (Baguelin et al., 1978).

The development of the SBPM where insertion could be achieved with minimal disturbance led to renewed interest in theoretical interpretation of the test results using the theory of cylindrical cavity expansion. If the stress conditions at the walls of the cavity are unchanged by the insertion of the instrument, then the initial conditions for the analysis should be well-defined, enabling measurement of the in-situ lateral stress, strength and deformation parameters. Hughes, Wroth and Windle (1977), using X-ray techniques, showed that with careful attention to drilling procedures it was possible to limit disturbance in sands to 0.5% of the radius of the probe. However, for sands in situ, even after such careful attention, it is difficult to assess the level of disturbance except from the test results. In clays, some assessment of disturbance can be obtained by monitoring the pore pressure during and after insertion. Therefore, all SBPM test results will be subject to some unknown level of disturbance, a factor which led the French (see Baguelin et al., 1978) to treat SBPM test results empirically or semi-empirically.

Early attempts to derive soil modulus from SBPM tests used the initial part of the pressure-expansion curve. The shear modulus of the soil can be derived from the slope of a tangent or secant to the curve (Gibson and Anderson, 1961). Shear modulus derived in this manner has been found to be very sensitive to disturbance (Mair and Wood, 1987). Hughes (1982) and Fahey and Randolph (1984) noted that the value of shear modulus, $G_\text{ur}$, obtained from a small unload-reload loop performed during the loading phase of a test (Wroth, 1982) was insensitive to the manner of instrument installation. Jamiołkowski et al. (1985) found $G_\text{ur}$ from SBPM tests to be a reliable value of shear modulus for both clays and sands.
Research into the use of SBPM testing to obtain shear strength in sands (e.g. Fahey and Randolph, 1984) and clays (e.g. Ghionna et al., 1981; Mair and Wood, 1987) has also indicated the sensitivity of the results to disturbance.

In summarizing the state of the art of SBPM testing, Jamiolkowski et al. (1985) stated that the SBPM test requires careful attention to equipment, method of installation and test procedure. It appears that the person carrying out the SBPM test requires a very high degree of expertise and experience in order that the test is carried out satisfactorily. In addition, Jamiolkowski et al. (1985) stated that, even after much experience, it may still be difficult to select the appropriate insertion and test procedures without preliminary testing. SBPM testing is thus likely to be expensive and time consuming.

In an effort to devise a suitable instrument for carrying out the equivalent of SBPM tests in deep boreholes offshore, the Push-in Pressuremeter (PIPM) was developed (Reid et al. 1982). This instrument consists of a pressuremeter element on the outside of a steel cylinder which resembles a sample tube but which has the cutting edge bevelled inward in an attempt to minimise disturbance to the soil around the PM. Figure 2.7 illustrates the PIPM. The PIPM is pushed into the ground below the base of a borehole and the displaced soil enters the body of the instrument. One advantage of the PIPM is that a disturbed and elongated sample of the soil tested is obtained. A down-hole pressure developer is used to expand the membrane. Hughes and Robertson(1985) pointed out that as the instrument is a thick-walled tube with an area ratio of 40%, disturbance will inevitably occur during insertion leaving the initial stress state before the expansion test uncertain. Work carried out at the Norwegian Geotechnical Institute (Bandis and Lacasse, 1985) showed that insertion of the PIPM caused significant disturbance. The test also suffers from the disadvantage that the instrument must be withdrawn and the hole advanced before further testing can be carried out.
Figure 2.7  Pressuremeter Head of the Push-in Pressuremeter Device (after Reid et al. 1982).
The major problems in SBPM testing are thus:
- some soil disturbance occurs during insertion of the probe;
- a high degree of expertise is required to operate the instrument and interpret the results.

Given the above problems and the observation that unload-reload modulus is relatively insensitive to disturbance, Robertson (1982) investigated the possibility of inserting the PM by pushing it into the ground behind a solid conical tip. The instrument was termed the Full Displacement Pressuremeter (FDPM).

2.2.4 Full Displacement Pressuremeter (FDPM)

The idea of a pressuremeter pushed in behind a conical tip is not new. Baguelin and Jezequel (1983) described a "pressio-penetrometre" which was vibrated into the ground. The results were interpreted using correlations with PBPM design parameters. Briaud (1979) also described a form of FDPM used in pavement design which employed the slope of unload-reload cycles to infer the stiffness of airfield pavement subgrades. Again, interpretation was based on Menard rules.

Robertson (1982) confirmed the insensitivity of unload-reload modulus to disturbance by carrying out FDPM tests in which a self-boring pressuremeter was pushed into sand behind a solid conical tip. A schematic of the probe used is shown in Figure 2.8. Hughes and Robertson (1985) noted that the value of lift-off pressure in sands taken from the FDPM curves was of the same order of magnitude as those from SBPM tests and presented qualitative reasoning why this should be so. Analysis methods using cavity expansion theories were found to give unrealistic values of friction angle. It was suggested that if $G_w$ was the only parameter desired from pressuremeter work, it was unnecessary to go to the trouble and expense of self-boring. O'Neill (1985) described the results of an evaluation of the FDPM in both clay and sand. He found that the values of $G_w$ obtained were a consistent proportion of $G_{\text{max}}$ obtained by the seismic cone when stress and strain levels were taken into account but had no SBPM results to
Figure 2.8  Schematic of FDPM used by Robertson (1982).
which to compare his results. The values of undrained shear strength, $s_u$, obtained from the FDPM curves agreed favourably with those from vane shear tests.

Brown (1985) studied the FDPM with a view to deriving P-Y curves from the pressuremeter test curve. From test results in sand, where the cavity strain did not go beyond about 15%, he concluded that the disturbance due to installation did not significantly affect the maximum pressure reached in the test. He carried out a test in which the pressuremeter membrane was re-inflated at the same depth after a test had been carried out. The result is reproduced in Figure 2.9. He claimed that, had the pressuremeter been capable of sufficient expansion, the pressuremeter curve would have eventually rejoined the first curve indicating that initial disturbance has little effect on the large strain behaviour of a pressuremeter test. It is also evident from Figure 2.9 that the slopes of the unload-reload loops are very similar. In addition, the method of derivation of P-Y curves from the FDPM pressure-expansion curves for use in the prediction of the behaviour of laterally loaded driven piles was found to work well (Robertson et al., 1983, Robertson et al., 1986).

Given the above-noted attributes of the FDPM, particularly its ability to measure the shear modulus of sands, Robertson (1982) suggested that a potentially fruitful area of research could consist of the development of a cone-pressuremeter (CPM). This instrument would combine the features of the piezometer-friction cone penetration test (CPTU) and the FDPM. The CPTU would provide a continuous record of soil type and would allow an estimate of soil properties while the FDPM would give an indication of soil stiffness through measurement of the unload-reload modulus. This measured stiffness could be used to improve the empirical correlations used in interpretation of the CPTU. This thesis is a consequence of that early research.
Figure 2.9  Effect of Inflating Pressuremeter Twice at the Same Depth (after Brown, 1985).
3.0 RESEARCH SITES AND FIELD TESTING PROGRAMME

3.1 INTRODUCTION

The development of a new in situ testing instrument requires that testing be carried out at well-documented sites to enable a comparison between the results of the new instrument and those from other proven equipment. In the course of this investigation, field testing has been carried out at four research sites in the Vancouver Lower Mainland and at one site in Leidschendam, Holland. The sites are listed in Table 3.1 and the locations of the Lower Mainland sites are shown in Figure 3.1. The sites within the Lower Mainland are all UBC research sites at which data has previously been obtained with a variety of in situ testing equipment. Testing at the Lower 232nd Street site was in clay and will not be discussed herein. Test data for this site is presented and discussed in Hers (1989) and Campanella, Howie, Sully, Hers and Robertson (1990).

Testing with the Fugro FDPM (henceforth referred to as the FFDPM) was undertaken at the McDonald Farm Site and at Leidschendam. The former site was chosen as previous FDPM testing in sand (Robertson 1982, Hughes and Robertson 1985, Brown 1985) had been undertaken there. In addition, self-boring pressuremeter data from previous investigations were available (Robertson 1982, Hughes 1984). SCPM testing was also carried out at this site to allow comparisons of the two instruments. Access to this site was lost after the initial testing with the SCPM due to drainage works being carried out on Sea Island. SCPM testing was also carried out at the Laing Bridge Site and the Lulu Island Pile Research Site. Testing at the first site was predominantly in sand and at the other in soft cohesive soils.
Table 3.1  Full Displacement Pressuremeter Test Sites.

<table>
<thead>
<tr>
<th>TEST SITE</th>
<th>LOCATION</th>
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<tr>
<td>McDonald Farm</td>
<td>Sea Island, Richmond</td>
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<tr>
<td>Laing Bridge</td>
<td>Sea Island, Richmond</td>
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<tr>
<td>Lulu Island</td>
<td>Lulu Island, Richmond</td>
</tr>
<tr>
<td>Lower 232nd Street</td>
<td>Langley</td>
</tr>
<tr>
<td>Leidschendam</td>
<td>Netherlands</td>
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</table>
Figure 3.1  General Location of Research Sites.
Self-Boring Pressuremeter testing was carried out by the author at the Lulu Island Site using a PM supplied by Dr. John Hughes (henceforth called the HPM) and with the cooperation of Foundex Exploration. The test programme was limited by both the availability of the instrument and of a drill rig. The SBPM testing was undertaken in an effort to obtain comparisons between self-bored tests and FDPM tests and also to enable the author to obtain an appreciation for the procedures involved. The instrument was installed by jetting as was discussed in Section 2.2.3. The instruments used and procedures followed will be described in Chapter 4.

A brief description of each of the sites and an outline of the testing carried out at each one follow.

3.2 GEOLOGY OF LOWER MAINLAND

The Fraser River forms a major alluvial flood plain consisting of marine deltaic deposits. According to Blunden(1975), the delta began to develop about 11,000 years ago as a submarine delta. Prior to this time, the delta area had been the site of a number of earlier deltas and coastal plains and had also been subject to glaciation. The bedrock is of Tertiary age. Overlying the bedrock is a zone of glacial deposits interspersed with remnants of previous deltas formed between periods of glaciation. At the end of the most recent glaciation (about 13,500 years ago), the land had been depressed below sea level by the weight of ice. A submarine delta began to form but isostatic uplift resulted in the deep-water deposits being subjected to wave erosion.

Shallow-water marine delta deposits were then laid down. Further variations in land and sea level resulted in the formation of peat bogs and salt marshes. The annual flooding of the Fraser River laid down silt and clay levees and inundations from the Strait of Georgia deposited sand lenses and silt seams. An idealized geological cross-section through the delta is shown in Figure 3.2.
Figure 3.2 Geological Cross Section of the Fraser Delta (after Blunden, 1975).
To the east of the Fraser River Delta, in the Langley-Cloverdale area, lies a deposit of glacio-marine clay deposited during a period when the Fraser River became dammed by ice. Subsequent fresh water leaching has resulted in the formation of a sensitive clay.

3.3 McDONALD FARM, RICHMOND, B.C.

3.3.1 Site Description

This site is located on an abandoned farm on the north side of Sea Island which is located between the North and Middle Arms of the Fraser River on the north side of the main river delta. The site is approximately flat at an elevation of +1.6 metres. This has been a primary research site for the UBC In-situ Testing Group, has been extensively field tested with a variety of instruments and has been the subject of many research papers. A typical CPTU profile is shown in Figure 3.3.

The stratigraphy comprises 2 to 3 metres of soft, compressible silts and clays underlain by fine to coarse sand of variable density which becomes denser with depth. The sand layer includes thin layers of medium to fine sand and some lenses of silty sand and extends to about 13 metres depth. The sand is uniform with a $D_{50}$ of 0.1 to 0.6 mm and typically contains about 4% silt. A transition layer of fine sand with some silt exists between 13 and 15 metres depth. Below 15 metres is a deposit of soft, normally consolidated clayey silt of low plasticity.

Typical grain size distributions of the sand are presented in Figure 3.4 and a summary of properties of the underlying silt is presented on the CPT profile. A field vane shear strength profile in the silt is presented in Figure 3.5(a).
Figure 3.3  Typical CPTU Profile at McDonald Farm
Figure 3.4 Range of Grain size Distribution of Sand and Clayey Silt deposits, McDonald Farm. (adapted from Robertson, 1982)
Figure 3.5  Field Vane Undrained Shear Strength for
a) McDonald Farm
b) Lulu Island
(adapted from Hers, 1989).
3.3.2 Testing Programme

Figure 3.6 is a site plan giving the approximate locations of the soundings carried out in this study. Also shown are the locations of CPTU, SBPM and FDPM testing carried out by Robertson(1982), FDPM testing by Brown(1985) and SBPM testing by Hughes(1984) to which reference is made in this study. Table 3.2 lists the tests carried out in this study with both the Fugro FDPM (FFDPM) and the SCPM and the dates on which these tests were conducted.

3.4 LULU ISLAND PILE RESEARCH SITE (LIPRS)

3.4.1 Site Description

The Pile Research Site is located on the north side of the Annacis Channel which is a portion of the South Arm of the Fraser River. The site has been the subject of research into the application of in situ testing to pile design as detailed in Brown(1985), Robertson et al.(1983), Robertson et al.(1986) and Davies(1987). A typical CPTU profile is presented in Figure 3.7. Heterogeneous fill covers the site to a depth of from 2 to 4 metres except for a zone 5 metres by 12 metres in the vicinity of a group of test piles installed for the previous work. In this zone the fill was excavated and replaced with clean river sand to allow penetration of the sand by in situ testing equipment. This fill material was placed in three zones each of which was placed to a different density. Prior to the work reported herein, 0.6m to 1m of pit-run sand and gravel had been placed over the whole site including the clean sand zone.

Underlying the fill is a deposit of organic silty clay to silty clay which extends to about 15 metres depth. A summary of the soil properties of the upper organic silty clay is provided on the CPTU profile. A field vane shear strength profile is shown in Figure 3.5(b).
Figure 3.6 Site Plan of M'Donald Farm at Time of FFDPM Testing (adapted from Robertson, 1982).
Table 3.2  Pressuremeter Testing at McDonald Farm.

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Note: The table entries represent various test results or conditions at different depths and dates.
Figure 3.7  Typical CPTU Profile, Lulu Island Site (adapted from Davies, 1985).
Underlying the organic silty clay, a medium dense fine to medium sand deposited in a high energy environment (Davies, 1987) extends to between 25 and 30 metres depth. The $D_*$ of the sand is typically 0.13 to 0.17 mm. A soft, normally consolidated clayey silt of low plasticity containing thin sand layers underlies the sand and extends to a depth of more than 100 metres. The sand layers become thicker (up to 0.5 metres) below about 60 metres depth.

3.4.2 Test Programme

Figure 3.8 shows the locations of the in situ tests carried out in this study. Table 3.3 lists the pressuremeter tests carried out.

A total of 24 SBPM tests was attempted. Thirteen of the tests were in the organic silty clay and ten were in the underlying sand. The installation and testing procedures are described in Chapter 4. Two piston samples were obtained in each of the organic silt and the sand in hole HPM87-2. The samples were used for classification tests and for consolidation and triaxial testing (Zavoral, 1988).

FDPM testing was carried out on two occasions using the SCPM. On the first occasion, the instrument was operated with a solid steel cone ahead of the pressuremeter unit. Twelve tests were carried out at the depths shown in Table 3.3. On the second occasion, the instrument was operated with CPTU and seismic measurements. Seven tests were carried out before the membrane burst.
Figure 3.8 Locations of In Situ Tests Performed at Lulu Island Pile Research Site. (adapted from Davies, 1985)
Table 3.3  Pressuremeter Testing at Lulu Island Site.

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3.5 LAING BRIDGE SITE

3.5.1 Site Description

This site is situated close to the McConnachie Overpass near Vancouver International Airport, also on Sea Island. The site has been regraded during recent construction work and is flat and grass-covered. A typical CPTU profile is shown in Figure 3.9.

The stratigraphy comprises 1 to 2 metres of organic clays and silts over loose to medium dense sand which becomes dense with depth. The sand, which contains occasional silt layers, extends to about 20 metres depth. Below the sand lies soft normally consolidated clayey silt. Bertok (1987) describes the construction of the adjacent overpass and presents a summary of soil properties for the site. The index properties of the clayey silt are presented on Figure 3.9. Typical grain size curves for the sand are presented in Figure 3.10.

3.5.2 Testing Programme

FDPM testing was attempted twice at this site. The equipment malfunctioned on the first occasion. Acceptable results were obtained on the second occasion, on December 4, 1987. The locations of the test holes in relation to previous in situ testing (UBC course CE577, Fall 1987) are shown on Figure 3.11. The instrument was operated with the seismic CPTU module attached. FDPM tests were carried out at the depths listed in Table 3.4.
Figure 3.9  Typical CPTU Profile, Laing Bridge Site.
Figure 3.10  Grain Size Distribution, Laing Bridge Samples.
Figure 3.11 Location of Pressuremeter Testing at Laing Bridge Site.
TABLE 3.4

PM TESTING AT LAING BRIDGE

<table>
<thead>
<tr>
<th>DATE</th>
<th>4/12/87</th>
</tr>
</thead>
<tbody>
<tr>
<td>DEPTH (metres)</td>
<td>INSTRUMENT</td>
</tr>
<tr>
<td>3.0</td>
<td>SCPM</td>
</tr>
<tr>
<td>5.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>7.0</td>
<td>&quot;</td>
</tr>
<tr>
<td>9.0</td>
<td>&quot;</td>
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<tr>
<td>11.0</td>
<td>&quot;</td>
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<tr>
<td>13.0</td>
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</tbody>
</table>
3.6 LEIDSCHENDAM, HOLLAND

A CPT profile at the Fugro Research Site in Leischendam, Holland is presented in Figure 3.12. The upper 6 metres comprise uniform beach sands with a $D_{50}$ of about 0.17 mm and a silt content in the range 0-1%. Testing at this site was carried out by Fugro personnel using the Fugro FDPM.
Figure 3.12 Typical CPT Profile, Leidschendam, Holland (after Withers, Howie, Hughes and Robertson, 1989)
4.0 EQUIPMENT AND TEST PROCEDURES

4.1 INTRODUCTION

One rationale for the development of the FDPM was that, as it is extremely difficult to insert a SBPM without disturbing the soil, it seemed reasonable to use an instrument which could be inserted in a consistent manner with a consistent degree of disturbance. In order to gain an appreciation of the difficulties involved in SBPM testing and to provide additional reference data to which to compare the FDPM data, SBPM testing was carried out at the Lulu Island site and at another research site, the Tilbury site. The testing was done with the guidance of Dr J.M.O. Hughes and used his PM which will be referred to as the HPM. The HPM was modelled on the Cambridge SBPM. An earlier version of this instrument was used in the SBPM testing carried out at McDonald Farm by Robertson(1982) and Hughes(1984) and was the PM unit used in the first versions of the FDPM described by Robertson(1982), Brown(1985), O’Neill(1985) and Hughes and Robertson(1985). Details of the instrument are listed in Table 4.1. A brief summary of the results of the laboratory and field evaluation are presented in Section 4.2. The equipment and procedures used are described in more detail in Appendix A.1.

Much of the work conducted in this investigation involved the critical evaluation of the design and performance of two prototype full-displacement pressuremeters. The main features of the instruments are provided in Table 4.1. Sections 4.3 and 4.4 of this chapter describe the FDPM instruments and ancillary equipment and the main conclusions of the laboratory and preliminary field evaluations of these instruments. Some discussion of the relative merits of the instruments is then presented. Detailed descriptions of the laboratory and preliminary field evaluations are presented in Appendix A.2 and A.3.
Table 4.1 Details of Pressuremeters Used in Study

<table>
<thead>
<tr>
<th></th>
<th>FUGRO FDPM</th>
<th>SCPM</th>
<th>SBPM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (mm)</td>
<td>43.7</td>
<td>44</td>
<td>75</td>
</tr>
<tr>
<td>L/D Ratio</td>
<td>10.3</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>Inflation Method</td>
<td>Air From Surface Stress-Controlled</td>
<td>Oil, Down-hole PD Strain Controlled</td>
<td>Air from Surface Stress-Controlled</td>
</tr>
<tr>
<td>Strain Measurement</td>
<td>Strain Arms at 120°</td>
<td>Strain Arms at 120°</td>
<td>Strain Arms at 120°</td>
</tr>
<tr>
<td>Maximum Strain $\frac{\Delta R}{R_o}$</td>
<td>50%</td>
<td>27%</td>
<td>20%</td>
</tr>
<tr>
<td>Cone Tip to Centre of Membrane</td>
<td>0.92 m and 1.2 m</td>
<td>0.98 m</td>
<td>N/A</td>
</tr>
<tr>
<td>Maximum Pressure (MPa)</td>
<td>10</td>
<td>7</td>
<td>Unknown</td>
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<tr>
<td>Data Acquisition Method</td>
<td>Compaq</td>
<td>Custom Built</td>
<td>Custom Built</td>
</tr>
<tr>
<td>Other</td>
<td>Dummy Cone Tip</td>
<td>Piezometer Friction Cone With Accelerometers Below PM and Above Cone</td>
<td>Jetted In</td>
</tr>
<tr>
<td>Overall Length (m)</td>
<td>1.825</td>
<td>3.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Membrane Type</td>
<td>Urethane</td>
<td>Rubber</td>
<td>Urethane</td>
</tr>
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</table>
4.2 THE HUGHES PRESSUREMETER

4.2.1 Equipment

The HPM was 74mm in diameter and the expanding section had a length to diameter (L/D) ratio of 6. The urethane membrane was protected by a sheath or "Chinese Lantern" made up of overlapping stainless steel strips. The strips were welded or rivetted to a ring at each end, the upper one of which was free to move to allow the lantern to shorten as the membrane expanded. The membrane was inflated using nitrogen fed from the surface through a plastic tube which was taped to the rods as the HPM was drilled in. Tape was placed around the centre of the probe to prevent ingress of soil behind the lantern during drilling at shallow depth where the soil pressure was too low to prevent flexure of the strips. The tape was easily broken during the expansion.

Three strain arms, consisting of strain-gauged beryllium copper strips operating as cantilevers, were used to monitor the expansion of the membrane. The arms were located at mid-height of the probe and were spaced at 120 degrees around the circumference. Two effective stress cells were incorporated within the HPM. These cells could be attached to the membrane allowing the difference between the internal and external membrane pressures to be monitored. As the internal pressure is the total pressure required to inflate the membrane against soil and water and the external pressure sensed by the cells is the pore pressure, these cells enable the measurement of the effective stress against the membrane.

The analogue signals from the total and effective pressure transducers and the strain arms were transmitted to the surface through a cable which passed up the inside of the gas line. For the tests carried out in this investigation, the signals were amplified down hole and were transmitted to the surface using a multiplexed signal at a rate of about one set per second. The analogue signals were converted to digital
output using an eight bit A/D converter in the control box. The digital data were fed to a Sharp computer and were plotted on screen and stored in Random Access Memory (RAM). At the end of the test, the data were transferred to a floppy disk. The pressure was controlled using a regulator on the control box.

For the tests conducted by Robertson (1982) and Hughes (1984) at McDonald Farm, the pressure and strain arm outputs were recorded by reading from a digital voltmeter. The analogue outputs from the pressure and two strain arms were also recorded on XYY' strip chart recorders. This made continuous monitoring of the test possible without the need to plot the digital readings. However, the requirement to read all channels on the voltmeter resulted in a minimum time between pressure increments of approximately 30 seconds.

4.2.2 Laboratory Evaluation

As the main focus of this study was the FDPM, the SBPM testing was conducted primarily to allow comparison of the FDPM results to data obtained using a standard commercially-available SBPM installed and expanded using procedures fairly typical of industry practice. Consequently, the SBPM instrument was not subjected to rigorous evaluation. The laboratory evaluation consisted primarily of calibration of the strain arms and pressure transducers to allow conversion of the electrical signals to units of displacement and pressure respectively. Details of the calibration procedure are presented in Appendix A.1.

4.2.3 Installation

Installation procedures used in this study are described in this section. Details of the procedures used in the testing at the McDonald Farm site can be obtained in Robertson (1982) and Hughes (1984).
A track-mounted HT700 drilling rig and a Boyle 512 Positive Displacement Mud Pump operated by Foundex Explorations Ltd. were used to install the HPM. Previous research work (Clarke, 1981; Denby, 1978; Robertson, 1983; Jamiolkowski et al., 1985) has indicated that the major factors influencing the installation are:

- drilling fluid pressure control;
- rotary cutter or jetting tool geometry and position relative to edge of cutting shoe;
- insertion force;
- insertion rate;
- drill rod vibration (mainly relevant to rotary cutters).

In this study, drilling mud was pumped down the centre of the drill rods to the tip of the jetting tool. The mud broke up the soil in the cutting shoe and carried it upwards. The mud and cuttings escaped through the holes in the rod above the HPM unit. The mud then supported the borehole to prevent the sides caving in above the PM. The mud pressure was monitored and controlled by the driller. The mud pressure must not be so great that it washes out a cavity ahead of the cutting shoe but must be sufficient to carry the cuttings clear of the shoe.

The location of the tip of the jetting tool was adjusted according to the material being penetrated. If the soil was to be allowed to move sufficiently far into the cutting shoe as to form a plug, the shoe would then act as a closed-ended tube and would stress the soil ahead of the PM. Alternatively, if the jets were too close to the cutting edge, stress relief would occur. For soft soils, the tip could be placed well behind the cutting edge of the shoe. For very dense sands, it was necessary to have the jets just behind the cutting edge. The pushing pressure was also monitored. For satisfactory installation, the pushing force must be sufficient to overcome the friction on the lantern but must not result in the probe advancing faster.
than the cuttings can be removed. When all components of the drilling process are operating satisfactorily, it should be possible to install the HPM at a rate comparable to that of the CPT, i.e. 2cm/sec, (Hughes, 1986).

It is clear that the installation of the HPM with minimal disturbance requires considerable experience. During the testing programme carried out at Lulu Island, relevant information about the drilling process was recorded. Details are given in Appendix A.1. Much difficulty was encountered in the drilling process.

Comparison of the flow rates recorded during the PM testing at McDonald Farm conducted by Hughes(1984) (see Appendix A.1) to those recorded at Lulu Island showed that considerably higher flow rates and higher mud pressures were used in the more recent work. There is no apparent reason for this. The penetration rates were similar. Some of the increase in mud pressure may have been due to smaller diameter holes in the jetting tool than in the previous work.

A disadvantage of the jetting process is that when the mud or pushing pressures increase, it is not possible to determine the reason. If the mud comes up the outside of the probe rather than the inside, the only indicator may be a reduction in pushing force. However, this can also be an indicator of efficient jetting.

Table 4.2 lists the tests which, on the basis of an examination of the drilling parameters, it is judged that the installation was closest to being satisfactory. At the Tilbury site, no satisfactory SBPM tests were carried out due to problems with a leaking pneumatic line, a burst membrane and clogging of the cutting shoe.
Table 4.2

SBPM Tests For Which Drilling Parameters Indicate Satisfactory Installation.

<table>
<thead>
<tr>
<th>MATERIAL TESTED</th>
<th>HOLE NO.</th>
<th>DATE</th>
<th>TEST NO.</th>
<th>TEST DEPTH (metres)</th>
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<tr>
<td>Organic</td>
<td>SBPM87-1</td>
<td>11/2/87</td>
<td>2</td>
<td>4.80</td>
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<tr>
<td>Silty</td>
<td></td>
<td></td>
<td>3</td>
<td>6.35</td>
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<tr>
<td>Clay</td>
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<td></td>
<td>4</td>
<td>7.90</td>
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<tr>
<td>Organic</td>
<td>SBPM87-3</td>
<td>19/2/87</td>
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<td>6.35</td>
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<tr>
<td>Silty</td>
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<td>3</td>
<td>9.40</td>
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<tr>
<td>Clay</td>
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<td>4</td>
<td>10.90</td>
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<tr>
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<td>12.40</td>
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<tr>
<td>Organic</td>
<td>SBPM87-1</td>
<td>12/2/87</td>
<td>2</td>
<td>16.60</td>
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<tr>
<td>Sand</td>
<td>SBPM87-2</td>
<td>16/2/87</td>
<td>2</td>
<td>19.80</td>
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<td></td>
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<td></td>
<td>3</td>
<td>21.30</td>
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<td>2</td>
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<td>3</td>
<td>25.00</td>
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</table>
4.2.4 Testing Procedure

After the probe had been attached to the drill rods and prior to installation, the membrane was inflated in air to determine the correction to be applied to the pressure readings in subsequent tests. A typical pressure-expansion curve is shown in Figure 4.1. The probe was then jetted into the soil.

When the desired test location was reached, the thrust was removed from the drill rods. At Lulu Island, no effective pressure cells were used and so the pore pressures created around the membrane could not be measured. Inflation was commenced in most of the tests within approximately 5 minutes after completion of drilling.

The pressure was increased very slowly until all three arms had begun to move outwards, i.e. lift-off had occurred. Thereafter, the pressure was increased steadily until the maximum cavity strain was reached. (Cavity strain = \( \Delta r/r_o \) where \( \Delta r \) is the increase in radius from \( r_o \), the initial cavity radius.) In the tests in clay, some tests were carried out with no unload-reload cycles and others included an unload-reload cycle. The cycles were carried out quickly and were usually conducted with no pause. This was to limit potential for dissipation of pore pressure in order that an undrained modulus could be measured. In sands, the pressure tended to be increased in small increments. The main reason for this was that, due to the tests being rather deep ( > 15 metres depth) and the compressibility of the inflation system, there was some delay between setting of the pressure at the surface and response of the membrane. This resulted in a stepped pressure-expansion curve. At least one unload-reload cycle, conducted after a relaxation period to allow time-dependent strain to occur, was carried out in each test.
Figure 4.1 Membrane Correction Curve for the HPM (after Hers, 1989).
4.2.5 **Typical Results**

A problem associated with SBPM data is the lack of a consistent standard for assessment of the quality of the pressure-expansion curves. Robertson (1982) proposed the following criteria for acceptance of a test:

- no inflection point near the beginning of a test;
- all feeler arms move in a consistent manner;
- the pore pressure measured before expansion is close to the in situ equilibrium pressure.

As no pore pressure measurements were available, the only way of assessing the quality of the installation was to examine the expansion curves. Only the curves for which the observations during drilling indicated that installation had been satisfactory, i.e. those in Table 4.2, were considered.

Figure 4.2 shows the pressure-expansion curves for all three arms for a test at a depth of 12.5m in clay. Arms 2 and 3 lifted off at identical pressures but the Arm 1 lift-off pressure was considerably higher. Later in the test Arm 2 moved much further than 1 and 3. The cause of the anomalous behaviour of Arm 1 is uncertain. The soil around the probe may have been disturbed, the soil properties in the vicinity of each arm may differ or the results may indicate an equipment problem. The fact that the lift-off pressures are unrealistically high (for a $K_c$ of 0.5 a total horizontal stress of about 150 kPa would be expected compared to the lift-off pressures 250 kPa or greater observed in Figure 4.2) suggests that disturbance during installation resulted in an increase of total stress against the membrane which likely caused excess pore pressures. The degree of disturbance is unknown. Similar disturbance was observed in the other tests in clay in this study. Pore pressure measurements are clearly essential in SBPM tests in clay.
Figure 4.2  Hughes SBPM Test in Clay - "Satisfactory" Installation.
Figure 4.3 shows a comparison of two tests in sand at Lulu Island (at 21.3m and 24m) for which observations of the drilling procedures indicated that installation had been satisfactory. The test at 24m clearly shows inflexion during the early stages of the test for Arms 1 and 2 which, by the above criteria, would result in the test being "unacceptable". The test at 21.3m appears satisfactory although at large cavity strain, Arm 1 had moved a smaller amount than Arms 2 and 3. However, when the early part of the curve is examined at a larger scale (see Figure 4.4), Arm 3 could be interpreted as displaying inflexion.

It can be seen that determination of the acceptability of a test requires judgement based on experience and a thorough understanding of the installation and test procedures.

4.2.6 Summary and Conclusion on SBPM Testing

The above discussion attempts to indicate the complexity of SBPM testing and the effort required to obtain good quality results. It is important to observe details of the drilling process as an aid to assessment of the quality of the test results. In clays, the ability to measure pore pressure is essential. Even with this information, the assessment of the quality of the test data is difficult and requires judgement. No widely accepted criteria exist on which this judgement can be based.
Figure 4.3  Comparison of Two Hughes SBPM Tests in Sand - "Satisfactory" Installation, Lulu Island.
Figure 4.4 Lift-Off Behaviour, Hughes SBPM Test in Figure 4.3(a).
4.3 FUGRO FULL DISPLACEMENT PRESSUREMETER (FFDPM)

4.3.1 Equipment

4.3.1.1 The Instrument

The prototype 15 cm$^2$ FDDPM developed by Fugro was constructed by Cambridge In-Situ, Cambridge, England and is described by Withers et al. (1986). A schematic diagram of the probe and its deployment method is shown in Figure 4.5 (a) and details of the pressuremeter module are given in Figure 4.5(b). Although the pressuremeter was designed to fit behind a 15 cm$^2$ Fugro cone, a solid steel "dummy" cone was supplied for the prototype pressuremeter. The minimum distance from the centre of the membrane to the cone tip was 0.93 metres and an extension piece was available to increase this distance to 1.29 metres.

The probe had a 448mm long expanding portion consisting of a urethane membrane which was protected from soil during insertion by a "Chinese Lantern", an arrangement of butting stainless steel strips bonded to a thin rubber membrane. Specially-designed "contraction rings" were provided to allow longitudinal movement of the ends of the lantern during expansion while ensuring that the lantern was tightly held to the body during insertion or extraction of the probe. In the absence of these rings, the lantern might have buckled outwards leading to increased disturbance and possible damage. Care was taken to ensure that, when assembled, the outside diameter of the probe was the same as that of the cone tip ahead of it, i.e. 43.7mm. This resulted in an expanding section with a length to diameter (L/D) ratio of 10.3. The L/D ratio was selected by Fugro in an effort to ensure that the pressuremeter would expand as a right cylinder even at maximum expansion of 50% of the initial diameter. The Cambridge SBPM has an L/D of
Figure 4.5  Fugro Full Displacement Pressuremeter (adapted from Withers et al., 1986)
approximately 6 which is believed adequate to ensure cylindrical expansion for cavity strains up to approximately 10% (Jewell et al., 1980; Laier et al., 1975).

The urethane membrane was selected for the Fugro probe because of its resistance to abrasion, ability to expand to 150% of its original diameter, capacity to withstand a differential pressure of 5 MPa and the relatively small pressure required for inflation in air. O’Neill (1985) showed that, for a pressuremeter with a steel-reinforced membrane, high pressures required to inflate the pressuremeter severely limited the instrument’s usefulness in soft soils as the membrane correction was often as large as the soil resistance. In addition, the steel-reinforced membrane tended to display highly rate-dependent behaviour.

The strain arm system used on the FFDPM is shown schematically in Figure 4.6. It is the same system used in the Cambridge SBPM, consisting of a strain-gauged Beryllium Copper cantilever beam bearing on a pivoted arm. As the membrane expanded, the arm followed it out causing a change in output of the strain-gauges which were calibrated to give values of deflection. The strain arms were capable of sensing a deflection of up to 11mm, equivalent to a cavity strain of about 50%. The membrane was inflated by compressed nitrogen gas fed from the surface and the pressure was measured using a pressure transducer located on the amplifier board, mounted in the housing just above the pressuremeter unit. The signals were transmitted to the surface after amplification along a cable which came up the inside of the gas line. This line was threaded through the cone rods.

4.3.1.2 Test Configuration

A schematic of the test arrangement used for this study is shown in Figure 4.7. As the instrument had been designed to work with the Fugro Data Acquisition System (DAS) which was not available to the
Figure 4.6  Schematic Showing Principle of Strain Arm Operation Used on FFDPM.
Figure 4.7  Test Configuration for Fugro FDPM
author, some modifications to the connectors at the surface were required. Details of the DAS and test control system are presented in Appendix A.2.

A Cambridge In-situ Strain Control Unit was supplied with the FFDPM. This unit had been designed for use with the larger Cambridge In-situ SBPM and was found to perform poorly during unloading with the FFDPM. The unit controlled the rate of expansion by using the output of one strain arm in a feedback loop. The pressure was pulsed to achieve the required strain rate. However, the unit supplied had been designed to operate with the larger diameter Cambridge SBPM and so the pulses of air supplied were too large to permit close control of the rate of inflation.

4.3.2 Laboratory Evaluation

Before being field tested, the FFDPM required calibration and was subjected to a laboratory evaluation during the calibration process. Details of the procedures used are presented in Appendix A.2. The major finding of the laboratory evaluation was that the strain arms displayed hysteresis during simulated unload-reload cycles during calibration. This resulted in the calibration factor applicable to a particular unload-reload cycle varying with the magnitude of the loop and the arm deflection at the beginning of the loop. An approximate method of correction for this effect was obtained. The problem was believed to be caused by friction between the Beryllium Copper spring and the follower (see Figure 4.6).

4.3.3 Field Evaluation

Twenty-four pressuremeter tests were carried out at McDonald Farm using the Fugro FDPM in the sand, three in the overlying silts and clays and five in the underlying clayey silt. One test, was conducted in
a strain-controlled manner using the strain-control unit supplied with the probe. All other tests were stress-controlled.

Prior to pushing the probe into the ground, the membrane was inflated in air to check for satisfactory operation of the equipment and to determine the membrane correction. This inflation in air was repeated at the end of a sounding to ensure that no change had occurred in the membrane correction.

The probe was then pushed to the required test depth and the thrust was removed from the rods. For the stress-controlled tests, the pressure was gradually increased until movement of the arms i.e. lift-off, occurred. Thereafter, the test procedure varied depending on the specific focus of each test.

In most cases, the pressure was increased steadily until the strain level was reached at which an unload-reload loop was to be performed. In sands, the pressure was maintained constant to allow deformations to stabilize. In clays, the unloading commenced immediately to prevent pore pressure dissipation. The pressure was reduced a small amount and was then returned to its original value. The expansion was continued until the next holding level was reached. When the full expansion of the probe had been reached, the pressure was vented to atmosphere in a controlled manner. When the membrane was fully collapsed, the probe was pushed to the next test depth.

On the first field day, an attempt was made to carry out expansions to the full strain capacity of the probe, i.e. $\Delta r/r_o = 50\%$, but at 8.0m depth the membrane burst and the sounding had to be abandoned. It was later postulated that expansion to such high strains led to blistering of the membrane into the gap between the end of the membrane and the lantern clamp leading to pinching of the membrane as illustrated schematically in Figure 4.8. On subsequent field days, the expansion was limited to
Figure 4.8  Schematic Showing Possible Source of Membrane Damage, FFDPM.
approximately 40% cavity strain and it was recommended to Fugro that the distance between the point of fixity of the lantern and the membrane clamping rings be reduced.

The results of two tests carried out at the same depth, 11.2m, in adjoining holes are presented in Figure 4.9. One test was carried out in a strain-controlled manner. For this test, Strain Arm 3 was used for feedback to control the rate of expansion. The rate of strain was approximately 3% per minute. The cavity pressure at which expansion began was between 200 kPa and 300 kPa and the pressure at which the membrane closed after deflation was about 90 kPa. The other test was stress-controlled. In such tests, it was noted that when the pressure was maintained constant prior to unloading, deformation continued for some considerable time, especially at large cavity strains. Examination of the CPTU log shown in Figure 3.3 indicates that no significant pore pressures were generated during penetration at the standard rate of 2 cm/sec. Even if pore pressures were created during pressuremeter expansion, they should be small and would be expected to dissipate very rapidly. The holding phase of the stress-controlled test at a cavity pressure of about 1170 kPa was maintained for approximately 10 minutes and deformation was still taking place. It was thus concluded that deformation was due to creep of the sand. The deformation was allowed to continue at constant pressure until the rate had fallen to a low level (typically less than 0.1% per minute). An unload-reload (UR) loop was then carried out. In the stress-controlled test, the lift-off and closing pressures were similar to those in the strain-controlled test. The maximum pressure reached in the former test was about 1050 kPa and in the latter was about 1170 kPa.

A comparison of the two curves shows that in the strain-controlled test the beginning of each unload-reload loop was very rounded whereas, in the stress-controlled test, this point was well-defined. The creep deformation was a strong feature of all tests and will be dealt with in detail in a later section. The point to note is that the stress-controlled test allows the magnitude of this deformation to be measured
Figure 4.9 Comparison of Stress and Strain Controlled Tests, Depth = 11.2 m, McDonald Farm, FFDPM.
whereas in the strain-controlled test this behaviour is hidden. Also, in the latter test, the strain-control unit incorporated a "HOLD" switch which was designed to maintain the strain constant prior to an UR loop. Because of the creep deformation, the pressure dropped rapidly to achieve constant strain, i.e. stress relaxation occurred. The control on the unit was insufficiently fine to prevent the strain from continuing to increase, leading to the observed curvature.

Another drawback of the strain-controlled test is illustrated in Figure 4.10 which shows the strain-controlled test with the individual arms plotted rather than the average of the arms. It can be seen that arm 1 displayed much less movement than the other two. Hence, a unit relying on the output from one arm may not produce the desired average strain rate. Because of the above factors, all further tests were stress-controlled.

4.3.4 Summary of FFDPM Evaluation

The prototype of the Fugro FDPM was found to have several mechanical and electrical problems the most important of which was the hysteretic behaviour of the strain arms during unload-reload loops. There was also a problem of drift of strain arm output with time. Incompatibility between the instrument and the available Data Acquisition System (DAS) introduced additional problems of electrical noise which are less likely to occur when the probe is used with the DAS for which it was designed. Once inserted into the ground, the instrument functioned well enabling 13 tests to be conducted in approximately 7.5 hours of field time including set-up. One major new effect noted in tests with this instrument was the large amount of creep deformation observed during constant pressure phases of tests in sands. This posed problems in attempts to run strain-controlled tests and so all tests but one were stress-controlled. The analysis and interpretation of the test results are discussed in Chapter 6.
Figure 4.10  Pressure-Expansion Curve For Individual Strain Arms, Strain-Controlled FFDPM Test.
4.4 UBC SEISMIC CONE PRESSUREMETER (SCPM)

4.4.1 Equipment

Campanella and Robertson (1986) presented the concept of the UBC Seismic Cone Pressuremeter. The instrument was to combine the features of the Seismic Cone Penetrometer (Campanella and Robertson, 1984) with the Full Displacement Pressuremeter (Hughes and Robertson, 1985). Therefore, an additional feature of the SCPM over the Fugro instrument is the incorporation of accelerometers both in the cone and just below the pressuremeter unit to allow measurement of shear wave velocity, \( V_s \). From \( V_s \), one can calculate \( G_{max} \), the small strain elastic shear modulus of the soil.

4.4.1.1 Pressuremeter

A schematic of the SCPM is presented in Figure 4.11. The prototype used in this study included neither the development module nor the pressure developer motor controller and the cone signals were not multiplexed down-hole but were transmitted to the surface and digitized in the Data Acquisition System (DAS), also designed and built at UBC.

The pressuremeter expansion unit was 220 mm long and the membrane was protected by a lantern consisting of overlapping stainless steel strips. A new clamping mechanism was devised which allowed the stainless steel strips to move during expansion and during insertion and extraction. Unlike the Fugro probe, no second membrane was required to support the strips. This reduced the pressure required to inflate the probe in air but had the potential problem of allowing soil ingress behind the lantern.
Figure 4.11 Schematic of the UBC SCPM (adapted from Hers, 1989).
The UBC probe was designed to be 44 mm in diameter, resulting in a length to diameter ratio of 5. This decision was a compromise between the need to ensure expansion as a right cylinder and the requirement that the probe should not be too long for deployment in the UBC In Situ Testing Vehicle.

The pressuremeter membrane was made of natural rubber. Natural rubber possesses good elastic properties and requires lower pressures for inflation in air than the urethane used on the Fugro PM. The rubber has the disadvantage of being less rugged.

The membrane was expanded by pumping oil from the pressure developer (PD) into the pressuremeter (PM) unit. This method of inflation was chosen because one of the design criteria was to eliminate the need for a pneumatic hose to the surface. The oil was pumped from the inside of the probe to the underside of the membrane and flowed along channels milled in the surface of the probe to ensure even distribution of pressure. A light silicon oil was selected as this was found to be of sufficiently low viscosity to ensure rapid movement through narrow channels between the PD and PM but did not cause deterioration of the rubber membrane.

The three strain arms consisted of strain-gauged strips of Beryllium Copper clamped at one end to act as a cantilever. When the membrane was fully collapsed the arms were fully flexed. As the membrane inflated, the arms followed it out. The arms were capable of measuring deflections of up to 6 mm or a cavity strain of about 27%.
Pressure was measured in two locations:

a) at the base of the pressure developer;

b) on the body of the probe below the membrane.

In the original design, only the PD transducer existed. The second was added to investigate whether any errors were introduced by pressure gradients between the Pressure Developer and the PM. The effect was found to be minor.

4.4.1.2 Pressure Developer (PD)

Preliminary design of the PD began in early 1984. Two potential designs were considered:

a) a piston and reservoir system;

b) a pump.

The major constraint was that the diameter of the probe had to be limited to 44 mm. The concept of a pump was attractive due to the potentially high rates of inflation and high pressures attainable and also due to possible savings in probe length. The piston system was ultimately chosen because of its simplicity and relatively low cost and because the saving in length for a PM of 44 mm, L/D=5 and maximum expansion of 25% was insignificant.

The direction of motion of the piston could be reversed simply by reversing the motor. The motor was supplied by a variable voltage power supply at the surface and this supply was regulated down-hole to 12V. In preliminary laboratory evaluation of the PD configuration, pressures of 7 MPa were generated.
4.4.1.3 Seismic Cone

The cone was 44 mm in diameter (bearing area = 15.2 \( \text{cm}^2 \)), had a 224 sq. cm friction sleeve with equal end areas, pore pressure measurement locations either on the face or behind the tip and behind the friction sleeve, and an inclinometer. Above the cone, two piezo-electric benders were mounted perpendicular to each other in a specially-designed housing. The benders respond to seismic wave arrivals. A discussion of the seismic cone, seismic cone testing and factors affecting it may be found in Rice (1983), Laing (1985) and Campanella and Robertson (1984).

4.4.1.4 Data Transmission and Acquisition

In the prototype instrument used in this study, all signals were transmitted to the surface in analogue form. It was, therefore, necessary to carry the signals from all cone, accelerometer and pressuremeter transducers (14 channels) through an analogue multiplexer to the surface. The large number of wires passing up the outside of the PD along shallow grooves caused some maintenance problems. These will be eliminated in the final version which will incorporate a downhole A/D microprocessor system as shown in Figure 4.11. In addition to the cone and pressuremeter channels, a limit switch was incorporated in the PD to indicate when the piston had been fully retracted. All signals plus the motor power supply had to be transmitted along a 14-pin cable threaded through the centre of the cone rods.

In the preliminary field trials, the instrument was operated without the cone, as had been the case for the Fugro probe. Again, a Connection Box was constructed to allow signals to be taken to the Watanabe XYY' Chart Recorders and to the Data Acquisition System. The test set-up was similar to that for the Fugro probe but there was no longer a need for the pneumatic pressure system of valves. The computer
used in the system was specially designed and constructed at UBC. Details of the Data Acquisition System are presented in Appendix A.3.

4.4.1.5 Test Configuration

A schematic of the full test configuration used in this study is shown in Figure 4.12. When the SCPM project is completed, all data acquisition will be on a dedicated computer system with full graphic display for each mode. As for the tests with the Fugro probe, Watanabe XYY' recorders were used to monitor the tests carried out for this study.

4.4.2 Laboratory Evaluation

Prior to deployment of the system in the field, extensive testing was carried out in the laboratory in an effort to determine the characteristics of the instrument. The laboratory evaluation included calibration of the arms and pressure measurement system, development of a method for saturation of the hydraulic pressure system, and a study of pressure effects on the strain arms which included tests in a specially-designed pressure chamber. A detailed description of the work carried out is given in Appendix A.3. The laboratory evaluation gave a good understanding of the idiosyncrasies of the probe which proved useful in the interpretation of the initial field tests. In particular, a tendency for apparent arm movement during changes in either internal or external pressures was observed. This was traced to interaction between the arm head and the membrane and led to changes in arm head design as described in Appendix A.3.
Figure 4.12  Schematic Layout of the SCPM Test Control and Data Acquisition Systems (after Hers, 1989).
4.4.3 Field Evaluation

4.4.3.1 Initial Field Testing

The first day in the field with the UBC probe was January 14, 1987. At this stage, the pressuremeter unit was used behind a steel "dummy" cone identical in dimensions to the final fully-instrumented cone (i.e. 44 mm in diameter). The first sounding was at McDonald Farm, the same site at which the Fugro probe was tested.

A typical pressure-expansion curve is presented in Figure 4.13. The major difference observed between the SCPM curves and those obtained with the HPM and the FFDPM were that the SCPM curve was S-shaped, a feature of Menard or Pre-bored PM tests. The severity of the S-shape tended to reduce with depth. A partial explanation of the S-shape was found to be that the PM element was approximately 0.4 mm smaller in diameter than the cone ahead of it when the lantern strips were flattened against the probe. This resulted in additional unloading of the soil over that which usually occurs after passage of the cone tip. Attempts were made to increase the diameter by using two rubber membranes without success. The compressibility of the lantern led to further difficulties in that the degree of compression varied with the external effective stress and so varied from test to test. An approximate method of correction for this effect was devised (described in Appendix A.3). Figure 4.14 shows a pressure-expansion curve after it has been corrected for system flexibility. The S-shape is still apparent.

Also of note in the field testing was that due to the rigidity of the hydraulic inflation method, small movements of the piston in the PD resulted in rapid, large changes in pressure. This meant that at the beginning of inflation (i.e. close to lift-off), the data acquisition system had to record data at high frequency in order to define the curve. Also, if at any time expansion was stopped, there was a rapid
Figure 4.13 Uncorrected Pressure-Expansion Curve, Depth = 8 m, McDonald Farm, SCPM.
Figure 4.14  SCPM Pressure Expansion Curve Corrected for Instrument Flexibility.
drop in pressure. This was in contrast to the on-going expansion at constant pressure observed in the FFDPM tests. There was some indication that a leak may have existed in the hydraulic inflation system but this could not be confirmed. However, the rapid reduction in pressure was also consistent with stress-relaxation at constant strain, a phenomenon with the same origin as the creep noted at constant stress with the FFDPM. A discussion of this topic will be presented in Chapter 6.

The field evaluation also led to improvements in the behaviour of the strain arms at lift-off. As described in Appendix A.3, interaction between the membrane and the arm head resulted in apparent movement of the arms prior to lift-off. The strain arms were amended and the improved results at lift-off are shown in Figure 4.15. Detailed discussion of the procedures followed during the field evaluation is given in Appendix A.3.

4.4.4 Summary of SCPM Evaluation

The UBC SCPM combines the features of a seismic cone penetrometer and a full displacement PM. The laboratory and field evaluation led to the recognition of several areas of concern in PM testing. The problem of hysteresis during unload-reload loops identified in the Fugro probe was avoided by designing the strain arm as a simple cantilever. Attempts were made to improve the performance of the SCPM by improving the arm head design and by developing a method to correct the pressure-deflection curves for system compliance. Two membranes were used on the expanding section in an effort to reduce the effect of the slightly undersized PM body. Despite all the refinements, the pressure-deflection curves still displayed a S-shape which is more typical of the pre-bored PM. The S-shape was most apparent at shallow depth. It was noted that the shape of the initial part of the curve was very sensitive to the membrane correction as this correction curve is very steep in the initial stages. In weak soils, where the membrane correction was a significant part of the measured pressure, a slight error in the choice of
Figure 4.15 Effect of Different Strain Arm Head Designs on the Lift-Off-Stage of a Pressure-Expansion Curve (adapted from Hers, 1989).
reference values for the strain arm deflections could have a large effect on the corrected curve. In addition, at low values of effective stress, the compliance correction was large and changed rapidly with pressure. The potential for inaccuracy was again large, especially in view of the difficulty in determining the prevailing lateral effective stress. The effect of the probe being slightly undersized has still to be assessed.

4.5 INFLUENCE OF EQUIPMENT AND TEST PROCEDURES ON PRESSUREMETER TEST RESULTS

The major lesson learned from the evaluation of the instruments used in this study is that attention to detail is critical in the design, construction, calibration and installation of such equipment. The following general observations can be applicable to most pressuremeters.

4.5.1 Installation

The field work undertaken with the Hughes self-boring pressuremeter underscored the many factors which influence the quality of the results obtained. The importance of field experience, by both driller and field engineer, in soils similar to those being investigated is paramount. In addition, an accurate knowledge of the soils being penetrated is required so that the jets can be suitably located relative to the edge of the cutting shoe. Very little research into installation of SBPMs by jetting appears to have been undertaken. The jetting technique appears to have many advantages over the use of a cutting tool as is currently used on the Cambridge SBPM. Some of these advantages are:

- very rapid penetration is possible;
the requirements for a rotary drive mechanism, internal and external rods and the associated vibration are removed.

However, problems caused by lack of robustness of the jetting tool, clogging of the cutting shoe, blockage of the jets and an inability to determine whether mud was coming up the inside or outside of the PM were encountered in this study. The blockage of the jets by gravel from the mud tank is easily solved by filtering and should not happen more than once to an inexperienced crew. The flow path of the mud could be more accurately determined by introducing a double rod system and having the mud and cuttings flow to the surface but this would complicate the system considerably. Another potentially fruitful area of research would be in optimising the design of the cutting shoe. Some of the clogging was due to material becoming stuck in the throat of the shoe as shown in Figure 4.16. This was due to the low clearance between the jetting rod and the inner wall of the shoe. A thinner jetting rod would likely lead to greater fragility of the system. Much research remains to be done. Some work in this area has commenced at UBC.

Another important aspect of SBPM insertion is that it is impossible to determine the degree of disturbance caused to the soil by the drilling process. The incorporation of pore pressure measurement on the membrane would have been advantageous but, substantial expertise is required to ensure satisfactory performance of the pore pressure measurement system. In sands, no method exists to assess the degree of disturbance except by examination of the test curves. The wide range of concerns to be addressed in SBPM testing and the difficulty of achieving "undisturbed" installation is clear. Consequently, the concept of the FDPM with a consistent degree of disturbance is attractive provided applicable methods of interpretation can be devised.
Figure 4.16  Schematic of Cutting Shoe Showing Potential for Blockage, HPM.
4.5.2 Strain Measurement

When the measurement of deflections is an important part of the test, the smaller the diameter of the probe, the more critical becomes the measurement precision. In sands, where unload-reload moduli may be 50 MPa or greater measured over a typical pressure increment of 300 kPa, an error of 0.01 mm in the measurement of deflection will cause a change in calculated modulus of 15 to 20% for a 44 mm diameter probe. When the size of the sand grains against the membrane is considered (e.g. a D₅₀ of 0.1mm), it is clear that the requirement for such precision is an exacting one. If the head of the strain arm bears against the membrane over a very small area then it is possible that movement of the head from bearing against a particle to spanning a void space could introduce errors (See Figure 4.17). However, measurement of radial deflection at three discrete points around the circumference and averaging of the results will dampen such an effect. The introduction of a larger arm head would also reduce this effect. In addition, use of a larger diameter probe would reduce the effect of such measurement errors as the strain is calculated from the expression \( \Delta r/r_o \), where \( r_o \) is the radius of the instrument.

One alternative to the use of strain arms for displacement measurement is the measurement of volume injected into the membrane. This requires the assumption that the membrane expands as a right cylinder. Suyama et al. (1983) used X-rays to monitor the displacements of lead balls implanted in sand to show that a PM expanded very close to a right cylinder in sand. Brown (1985) confirmed this by comparing simultaneous measurements of volume and displacement in a series of field tests. However, he also showed that as the stiffness of the soil decreased, the measured deflection at mid-height of the membrane departed further from the theoretical relationship between deflection and volume change based on the assumption of a cylindrical cavity. This is illustrated in Figure 4.18. In fluid (e.g air), the membrane expanded like a balloon. Hence, the assumption of a right-cylindrical expansion is not always valid and becomes less plausible for expansion to large cavity strains.
Figure 4.17  Schematic of Strain Arm Head in Relation to Soil Particles.
Figure 4.18 Comparison of Volume Change and Strain Arm Measurement of Membrane Expansion (adapted from Brown 1985).
The departure from right-cylindrical expansion in air also has implications for membrane corrections. The correction is applied as a function of strain. However, if the membrane expands in the shape of a football in air then the pressure at a given strain will be less than it would be for the same strain if the membrane expanded as a cylinder. This is another consideration pointing to the need for the minimum membrane correction possible.

A major advantage of volume measurement in the determination of strains is that the results obtained would reflect the average behaviour of the soil over the length of the membrane which may be more representative of soil response to loading than results based on measurements at only three points on the membrane. An example of a potentially misleading situation in the latter case would be where the membrane straddles a softer layer between two stiffer layers resulting in an "hourglass" shape of the membrane. CPTU pushed ahead of the PM would be useful in attempting to identify such situations.

A major disadvantage of volume measurement, apart from the need to assume the nature of the expanded shape, has been the need for substantial corrections for compliance due to the compressibility of the large lengths of tubing required to feed fluid to the instrument from ground surface. The use of a down-hole pressure developer offers the potential for greater precision as the tubing lengths are small and it is easier to ensure saturation. In addition, as it is clear that there is no unique relationship between volume increase and radial displacement, the pressure-expansion curves obtained will depend on the method of measurement employed. This will affect the interpreted results. The use of volume measurement is an area worthy of research.
4.5.3 **Strain Arms**

In stiff soils, the displacements measured during unload-reload cycles are extremely small, especially for the new 44 mm diameter instruments. Hysteresis of the strain arms during unload-reload cycles must, therefore, be minimized by good design and must be investigated during calibration. A cantilever beam arm design appears to be preferable to a spring and lever arm configuration. A disadvantage of the cantilever design used in the SCPM was the tendency for the spring load to result in protrusion of the arm head beyond the initial diameter of the instrument.

For measurement of lift-off pressure, it is imperative that compliance of the arm be largely eliminated. The arm design must be such as to prevent apparent movements due to pressure effects or to arm-membrane interaction. The size of the arm head relative to that of the soil particles is also important.

4.5.4 **Membrane and Membrane Protection**

The membrane correction to be applied to PM test results can be a substantial part of the total recorded pressure especially in soft soils. It is, therefore, desirable to limit its magnitude. The selection of a membrane must be a choice between a low resistance to inflation and robustness in service. To provide a robust membrane, some manufacturers provide a thick, steel-reinforced membrane. Membranes of this type have highly non-linear, rate-dependent correction curves and the membrane correction depends on the inflation procedure adopted. Urethane membranes of the type used on the HPM and the FFDPM also display a rate-dependent response as is shown in Figure 4.19 which shows a comparison of stress-controlled and strain-controlled inflation.
Figure 4.19 Comparison of Stress and Strain Controlled Membrane Correction Curves - Urethane Membrane.
To prevent damage to membranes during installation, an alternative to a reinforced membrane is to provide a protective sheath or lantern consisting of butting or overlapping steel strips placed longitudinally against the membrane. The sheath designs used on the instruments described herein were:

- butting steel strips bonded to a second membrane;
- overlapping strips with a degree of overlap sufficient to prevent gaps between the strips at maximum deflection.

The first option has a tendency to result in a larger membrane correction than the second. However, the second option results in the entrapment of soil particles between the strips and the membrane, especially when the inflated shape departs from a cylinder.

A further consideration in the design of the protective sheath is its compressibility under load. The curved strips used on the SCPM were found to be compressible especially at low external effective stresses. This compression resulted in errors in the initial portion of the expansion curve and was important to the determination of unload-reload modulus.

4.6 CONCLUSION

The results of pressuremeter testing are very sensitive to a variety of factors, some equipment-related, some related to the method of installation, and some to testing procedure. Tolerances on instrument dimensions are very significant in stiff soils such as sands. As it is extremely difficult to assess the degree of disturbance caused by SBPM installation, the concept of the FDPM with a consistent level of disturbance appears to have merit. This assumes that the results can be interpreted in a meaningful way.
The UBC SCPM offers several advantages over the Fugro FDPM as a research tool and potentially in commercial applications once the equipment problems identified above have been overcome or can be reliably calibrated out. The major advantages are:

1. The seismic cone will allow comparison of the small strain modulus, \( G_{\text{max}} \), and the PM unload-reload modulus, \( G_{\text{ur}} \);
2. The down-hole pressure developer will allow comparison and assessment of the suitability of volume measurement as opposed to displacement measurement for determination of cavity strain;
3. The use of an electrically-powered down-hole fluid-filled pressure developer allows greater flexibility of test procedure as the electric motor in the PD can be computer-controlled. The use of a saturated system ensures rapid response.

The major disadvantages are:

1. The great complexity of the instrument;
2. The exacting requirements of system saturation in order to keep compliance to a minimum;
3. The difficulty of establishing zero pressure readings in a sealed system;
4. The serious consequences of even a slight leak in the fluid system, a problem which can be accepted in pneumatic systems.
5.0 THEORETICAL INTERPRETATION OF SOIL PROPERTIES FROM PRESSUREMETER TESTS

5.1 THEORETICAL FRAMEWORK

The pressuremeter test is commonly analyzed by assuming that expansion of the membrane models the expansion of an infinitely long cylindrical cavity. The soil is assumed to be an homogenous, isotropic linear elastic-plastic continuum. Deformation is assumed to be in the horizontal plane alone i.e. plane-strain deformation, and the initial stress condition is assumed to be isotropic in the horizontal plane. The idealized stress path for an element of soil adjacent to the membrane is shown in Figures 5.1 and 5.2 for undrained and drained tests respectively.

It is assumed that, prior to the start of inflation, the radial stress, \( \sigma_r \), equals the circumferential stress, \( \sigma_\theta \), i.e. the soil element is at Point A in \( \sigma_r, \sigma_\theta \) space. When the radial stress is increased, the soil deforms elastically and it can be shown that \( \Delta \sigma_r = -\Delta \sigma_\theta \) (Lame, 1852). When the effective stress path reaches the failure line at point B, the soil element begins to deform plastically. For clays, it is generally assumed that the cavity expansion is an undrained process and the soil fails according to the relationship:

\[
\sigma_r - \sigma_\theta = 2s_u
\]  

(5.1),

where \( s_u \) = undrained shear strength. Therefore, at initiation of failure, \( p_c = p_o + s_u \), where \( p_c \) is the cavity pressure and \( p_o \) is the initial total stress.

For an undrained process, there is no change in average stress in the horizontal plane and so there can be no change in effective stress at the cavity wall. The total stresses continue to increase and this is matched by an increase in the pore pressure. After initiation of plasticity, the plastic annulus expands as the pressure increases.
5.1a Effective and Total Stress Paths in Clay.

5.1b Stress Distribution around Cylindrical Cavity in Clay.

Figure 5.1 Idealized PM Stress Path in Clays (Undrained)
Figure 5.2 Idealized PM Stress Path and Stress Distribution for Drained Tests.
During this phase, the pressure is given by the expression:

\[ p_c = p_o + s_u [1 + \ln(G/s_u)] + s_u \ln(\Delta V/V) \]  \hspace{1cm} (5.2),

where \( V \) is the current volume of the cavity and \( \Delta V \) is the increase in volume from its initial value.

It can be shown that a limiting pressure will be attained beyond which strain will continue with no further increase in pressure at \( \Delta V/V = 1 \). This is termed the Limit Pressure, \( p_L \), given by the expression:

\[ p_L = p_o + s_u [1 + \ln(G/s_u)] \]  \hspace{1cm} (5.3)

Therefore, the response of the PM can be represented by the expression:

\[ p_c = p_L + s_u \ln(\Delta V/V) \]  \hspace{1cm} (5.4)

For sands, the Mohr-Coulomb criterion for failure is usually assumed, i.e.,

\[ \sigma_c' / \sigma_s' = (1 + \sin \phi) / (1 - \sin \phi) \]  \hspace{1cm} (5.5)

where \( \phi \) is the friction angle. After the onset of plasticity at \( p_c' = p_o'(1 + \sin \phi) \), the soil is assumed to fail at constant stress ratio, i.e. the stress path travels along the failure envelope from B to C in Figure 5.2. All soil elements in the zone being stressed follow stress path ABC but, at any particular cavity pressure, each element is at a different position on that path. For example, when the soil element next to the membrane is at point C, the element at point X will lie somewhere along the line AB.

In either sand or clay, if at any time during expansion of the cavity the pressure is reduced slightly, the stress path will move below the failure envelope and the soil will behave elastically provided the unloading is sufficiently small to avoid plastic strains. Wroth (1982) pointed out the need to avoid unloading far enough to cause failure with the circumferential stress as the major principal stress. Upon reloading, the soil will behave elastically until the failure envelope is again encountered. On the basis of the assumed soil model, the behaviour during an unload-reload loop can be expected to be elastic.
In both the drained and undrained cases, when the cavity has been fully expanded, an annulus of failed soil surrounds the cavity out to the radius of the plastic zone, \( R_p \), beyond which the soil remains elastic as is illustrated in Figures 5.1(b) and 5.2(b). When unloading is begun the soil behaves elastically until unloading is sufficient to cause plastic strains. For the soil model assumed herein, the soil will remain elastic until failure occurs with the circumferential stress, \( \sigma_t \), as the major principal stress, i.e. point D in Figure 5.2(a). Houlsby and Withers (1988) refer to this as reverse plasticity. As unloading continues, a zone of reverse plasticity spreads outwards. If the unloading could be continued, a limiting radial pressure in unloading would be reached in clays, (Houlsby and Withers, 1988). Practically, the pressuremeter deflation would result in the membrane separating from the soil. Wroth (1982) noted that in sands the membrane is pushed back in by the ambient water pressure.

The above is the basic theoretical framework upon which most methods of interpretation of PM results are based.

5.2 INTERPRETATION OF PRESSUREMETER RESULTS

Analyses based upon the above theoretical framework can allow interpretation of PM results to provide soil parameters such as in situ horizontal stress, shear strength and stiffness. The most common methods of interpretation available are discussed below.

5.2.1 In-situ Horizontal Stress

If the pressuremeter can be inserted into the ground with no disturbance to the surrounding soil, then the pressure at which expansion begins should equal the total horizontal stress in the soil. Only the SBPM
offers the potential for "perfect" installation. In this case, the total horizontal stress could be determined by observing the pressure at which first movement or "lift-off" of the strain arms occurs.

As indicated in Chapter 4, the interpretation of lift-off pressures is complicated by instrument compliance. The ideal instrument would be one in which the measurement system gave no response until the pressure in the membrane equalled the external pressure. In practice, however, some movement is usually detected before this pressure. Ghionna et al. (1983) point to the need to reduce the compliance of the strain arms.

In sands, in order for the PM to be inserted into the ground, sand grains would have to be displaced slightly. Even if this displacement was limited to a half of one grain diameter, disturbance would still be occurring. For a typical particle diameter of 0.2 mm and a SBPM diameter of 80 mm, the movement of half a grain diameter is equivalent to a circumferential or cavity strain of 0.25%. In a linear elastic soil with a shear modulus of 20 MPa, such a strain would result in a radial stress change of 100 kPa. Hughes and Wroth (1973), using X-ray techniques, showed that even in controlled conditions in the laboratory, self-boring in kaolin resulted in radial displacement of the soil adjacent to the probe of an amount equal to 0.5% of the cavity radius, i.e. a circumferential strain of 0.5% of the material at the cavity surface. Similar results were claimed for sands. The above suggests that the "lift-off" method of in-situ stress determination is unlikely to be successful in sands.

Bellotti et al. (1987) reported the results of tests with a Cambridge SBPM in sand in a calibration chamber. The PM was placed in the chamber in two ways:

1. sand was placed around the PM to simulate "ideal" installation;
2. the PM was drilled in to simulate field self-boring conditions.
The measured lift-off pressures were compared to the stresses applied to the chamber. Figure 5.3(a) shows the results obtained for the case of ideal installation after modifications had been made to the strain arms to reduce mechanical compliance. Even under these ideal conditions, agreement was not perfect. Measured values were generally within 10% to 15% of the pressures applied to the chamber wall. Figure 5.3(b) shows the results obtained from the PM which had been drilled in. The measured pressures are very different from those applied which indicates that drilling disturbance occurs even in controlled conditions. However, Bellotti et al. (1987) commented that the poor comparison between measured and applied stresses may have been due to the extreme sensitivity to disturbance of sand pluvially deposited in the laboratory and that natural sands may be less sensitive to disturbance. Support for this suggestion may be found in the work of Clarke and Wroth (1987) who presented data from a test in sand which they claimed showed that drilling techniques have evolved to a level at which lift-off pressures very close to the in-situ stress can be measured.

Another complication in the measurement of in-situ stress is that, in many cases, the arms lift off at different pressures. Jamiolkowski et al. (1985) suggested that this phenomenon may be due to:

- the non-circular shape of the PM hole and consequent non-symmetrical disturbance of the soil;
- mechanical compliance of the instrument;
- deviation of the instrument from the vertical;
- non-uniform shear stress at the probe-soil interface;
- anisotropy of in situ horizontal stress.

Dalton and Hawkins (1982) suggested that in their tests in stiff clay, the difference in lift-off pressures was an indication of stress anisotropy and Benoit and Clough (1986) proposed a method of calculating
Figure 5.3 Comparison of Measured and Applied Horizontal Stresses in Calibration Chamber Tests (adapted from Bellotti et al, 1987).
the principal horizontal stresses from the three lift-off pressures. In view of the other potential causes of variable lift-off pressures listed above, it would seem imprudent to assume the existence of stress anisotropy.

It is clear that the use of the SBPM to measure horizontal stress by observing a lift-off pressure is fraught with difficulty and uncertainty. In sands, no real alternative exists but, for clays, alternative methods have been devised in an attempt to overcome the effects of disturbance.

One alternative method proposed for clays by Wroth and Hughes (1974) is based on the fact that in normally consolidated soils an increase in pressure above the in situ stress should result in the generation of pore pressure in the soil adjacent to the membrane. Mair and Wood (1987) point out that as most natural clays are slightly overconsolidated and so would initially deform elastically without generation of pore pressure, this method is unlikely to be successful.

Alternatively, graphical methods may be used where iterative procedures are carried out until agreement is obtained between the data and the stresses predicted by a particular soil model, e.g. Marsland and Randolph (1977). Such methods suffer from the drawback that soil seldom fits the assumed model.

Notwithstanding the above criticisms, Jamiolkowski et al. (1985) noted that the SBPM has provided reasonable estimates of $\sigma_{ho}$ compared to the "best estimate" by other available methods in soft and medium stiff clays. For stiff to hard clays and sands, the instrument has been less successful as the characteristics of the instrument become more important. Jamiolkowski et al. (1985) suggested that even if the instrument was reliable, it was likely that preliminary testing would be required at a site in order to determine the most reliable insertion procedure. Pressuremeters other than the SBPM disturb the soil too greatly for reliable estimates of in-situ stress to be obtained from the lift-off pressure.
5.2.2 Shear Strength

Tests in sands are usually drained and so the soil cannot be considered to deform at constant volume. Consequently, methods of analysis developed to derive friction angle from PM test results require some assumption about the volume change behaviour of sand during shearing.

Gibson and Anderson (1961) presented a solution to the cavity expansion problem in sands but made the unrealistic assumption that plastic deformation was equivoluminal. Ladanyi (1963) and Vesic (1972) proposed methods of solution of the cavity expansion problem in sand which attempted to incorporate volume change. Both methods required laboratory test results as input. Methods requiring the measurement of volume change behaviour in the laboratory are subject to the usual limitations of laboratory testing which in situ testing attempts to avoid such as the effects of sample disturbance and the choice of a suitable stress path. This tends to limit the usefulness of the test.

The expansion of a cylindrical cavity can be shown to be equivalent to simple shear. Typical soil behaviour in simple shear tests on loose and dense sand is shown in Figure 5.4. When failure is reached at peak stress ratio the rate of volumetric expansion is approximately linear and is a maximum. For dense sand at low to moderate confining pressures, dilation occurs almost from the beginning of the test. In loose sands, the sample contracts initially but is generally dilating at its maximum rate when failure occurs at peak stress ratio. The peak stress ratio is attained at larger values of strain in loose sands than in dense.

In 1977, Hughes, Wroth and Windle proposed a method of analysis which incorporated the main features of sand behaviour outlined above. Their analysis combined cavity expansion theory with the
Figure 5.4 Typical Sand Behaviour in Direct Simple Shear Tests.
stress-dilatancy theory of Rowe(1971). The soil behaviour was idealised as shown in Figure 5.5. The soil was assumed to behave elastically until the peak stress ratio, \( \sigma_p/\sigma_s \), was reached.

Thereafter, the sand was assumed to be dilating at a constant rate. The constant volume friction angle, \( \phi_{cv} \), must be determined from lab tests but as it is a material property and is independent of density, it can be obtained from tests on reconstituted samples. Dilation angle and friction angle are obtained from the analysis.

Jewell et al. (1980) showed that the Hughes et al. (1977) method worked well in dense sand where the term "c" is negligible. However, Robertson (1982) pointed out that in medium dense and loose sand, the standard SBPM which expands to a maximum of about 10% cavity strain will not strain the soil sufficiently for it to attain the maximum dilation rate and so "c" is not negligible. Robertson and Hughes (1986) proposed charts based upon laboratory simple shear test data to allow correction of the friction angle obtained from conventional SBPM test results for this effect.

The Hughes et al. (1977) theory has generally been accepted as the most appropriate for the analysis of SBPM tests in sands. Carter et al. (1986) presented a closed-form solution for cavity expansion in an ideal cohesive-frictional soil based on a similar model of soil behaviour. They derived an expression which takes into account both elastic and plastic strains in the plastic zone and pointed out that the Hughes et al. (1977) solution does not account for the former. They also indicated that, for small strains (< 10%), the distinction is unimportant but, for larger strains, the elastic strains in the plastic zone can be significant. Numerical methods are required to obtain a solution for larger deformations.

Manassero (1987) presented an analysis closely related to that of Hughes et al. (1977) in which Rowe's stress dilatancy relationship is used to describe the volumetric strain behaviour of the soil at any point.
Figure 5.5  Idealized Soil Model for Hughes, Wroth and Windle (1977) Analysis.
during the test. A finite difference technique is used to derive the complete stress-strain curve and stress path for the soil element at the cavity wall. The volumetric strain-shear strain relationship is also obtained. In this method, all strains are assumed to be plastic, and so no input of shear modulus is required. Only $\phi_\infty$ must be known. The great advantage of the Manassero (1987) method is that contractive and expansive volume changes can be considered and so tests in both loose and dense sands can be analysed.

Using data obtained in chamber tests in Ticino sand, Manassero compared the results of his method to those using the Hughes et al. (1977) analysis and the Robertson and Hughes (1986) correction. Manassero's method produced results which were in qualitative agreement with the expected soil behaviour but the corrected Hughes et al. (1977) method appeared to be more reliable when compared to laboratory data.

As with the methods for undrained shear strength involving differentiation of the test curve, Mannassero's method is sensitive to inconsistencies in the data such as those introduced by electrical noise and soil disturbance. Some insight into the sensitivity of the method was obtained by attempting to use it to analyse SBPM results from testing at Lulu Island conducted for this study.

Figure 5.6(a) shows the measured data and two attempts to fit the curve with a hyperbola. The derived stress-strain curves are shown in Figure 5.6(b). In this case, the peak stress ratio is predicted to occur within the first 2% cavity strain, which is the part of the PM curve most susceptible to error due to soil disturbance. Figure 5.6 shows that a slight change in the fitted curve results in a large change in the predicted peak stress ratio.
Figure 5.6 Sensitivity of Manassero's Method for PM Test Interpretation.
All analysis methods assume that all soil elements stressed by expansion of the pressuremeter follow identical stress paths. As discussed earlier, some disturbance must occur during installation of the pressuremeter and so the initial stress and strain state will not be uniform throughout the soil mass being tested. In addition, friction angle is a function of stress level. As a large stress gradient exists around the cavity, the peak stress ratio attained by a soil element will vary with radius. It is unlikely, therefore, that all soil elements will follow exactly the same stress path.

Fahey and Randolph (1984) suggested that slight disturbance could be allowed for by an iterative procedure in which the measured strain was corrected until the best straight line possible was obtained on a logarithmic plot of effective pressure vs cavity strain. The correction procedure was based on the idea that a small strain change is necessary to return the stress state next to the membrane to that prior to insertion of the PM and that if no disturbance had occurred, the data would plot as a straight line. Robertson and Hughes (1986) commented that the log-log plot is only likely to be a straight line in dense sand where the soil behaviour will be closest to that assumed in the soil model.

Mair and Wood (1987) discussed the use of the Push-in-Pressuremeter (PIPM) for derivation of friction angle. They found that disturbance resulted in the derivation of very low values of friction angle.

Houlsby et al. (1986) presented an analysis of the unloading portion of the pressuremeter curve, again based on the soil model of Hughes et al. (1977). The method requires input of the soil shear modulus. The attraction of such a method is that the effects of disturbance could be overcome by first expanding the membrane until all soil in the disturbed zone had been stressed beyond the yield surface pertaining after installation. Unloading would then commence from a known stress state. The predicted friction angles were in some cases lower than the constant volume friction angle, \( \phi_v \), and so were unrealistic.
The authors commented that a large strain formulation would likely be required to allow realistic values of friction angle to be derived.

In summary, therefore, the analysis method best suited to the estimation of friction angle in sand is one in which the volume change behaviour of sands in simple shear is modelled. The method of Hughes et al. (1977) has been found to work reasonably well in dense sand but does not model loose sand behaviour very well. All interpretation methods derived since 1977 have been based on the Hughes et al. (1977) method. The method devised by Manassero is conceptually attractive and seems to predict both loose and dense sand behaviour well when used with very well-conditioned test results but requires a numerical solution. However, as any more than very slight disturbance has been found to result in unsatisfactory friction angles, it would appear that only the highest quality SBPM testing has the potential for derivation of friction angle from currently available theoretical considerations. Interpretation of friction angle from the unloading curve is also conceptually attractive as it might be expected that the effects of disturbance would be reduced or eliminated by expansion so that unloading begins from a known stress state. Experience to date has not supported this expectation.

5.2.3 Modulus

In the previous sections of this chapter, disturbance effects have been shown to greatly influence the interpretation of PM test results. One reason for the development of the FDPM was that experience had indicated that the interpretation of modulus from PM results is relatively insensitive to disturbance. This section examines the assessment of modulus in some detail in an attempt to understand why the modulus should be insensitive to disturbance.
If the soil around the pressuremeter is assumed to be behaving elastically, it can be shown that the slope of the curve of pressure vs cavity strain is twice the shear modulus, $G$, i.e.

$$G = 0.5 \frac{dp}{dc}.$$  \hspace{1cm} (5.6)

If the PM has been inserted without disturbance, initial deformation should be elastic and so the slope of the initial linear portion of the curve should allow an estimate of $G$ (see Figure 5.7).

However, Fahey (1980) showed that in sands, the peak stress ratio is reached next to the membrane at cavity strains of typically less than 0.1% and so the initial linear portion of the curve will be very small and difficult to define. The relevant modulus will thus be obtained from a tangent to the initial portion of the expansion curve. Any disturbance during installation will alter the value of $G$ obtained.

Menard and his co-workers in France (Baguelin et al., 1978) recognized the difficulty of defining an intact modulus from PBPM tests and so defined a PM modulus, $E_{pm}$, which is derived from the linear portion of the expansion curve as shown in Figure 5.7(b). This modulus is used empirically in design.

As it is difficult to define an elastic modulus from the initial part of the curve, it is now widely accepted that the modulus can be defined from the slope of a small unload-reload (UR) cycle or loop performed during the test (Wroth, 1982; Hughes, 1982; Jamiolkowski et al., 1985). Interpretation of unload-reload cycles within the framework of elastic-plastic theory assumes:

- all soil elements around the probe follow one unique stress-path during loading;
- when unloaded below the current yield surface, the soil behaves linear-elastically with a unique modulus, $G_w$.  

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Figure 5.7 Methods of Determining Modulus from PM Tests.
Conventionally, it is assumed that the behaviour measured at the cavity wall represents the soil behaviour at one point on a unique stress path and that the stiffness measured is identical to the stiffness at all points within the plane of loading. In reality, the soil stiffness varies with effective stress level and strain level. Due to the variation in stress and strain levels with radius, the stiffness of a particular soil element varies with its radial distance from the cavity wall. The following sections address the effect of such variations from the simple linear elastic-plastic soil model.

Palmer (1972) showed that for the unique case of homogeneous soil deforming at constant volume, with the assumption that all soil elements follow identical stress-strain curves, the behaviour at the wall was sufficient to define the stress-strain curve of the material. In sands, once plasticity is induced, it can be shown that for a linear elastic-frictional plastic soil model, the plane strain average stress, \( \frac{1}{2}(\sigma_r + \sigma_\theta) \), increases as expansion proceeds and varies with radius in the plastic zone in accordance with the expression:

\[
\sigma'_m = \frac{P'_e \left( \frac{a}{X} \right)^{2 \sin \phi}}{1 + \sin \phi} \quad (5.7)
\]

As soil stiffness varies with mean normal stress, then the soil stiffness varies with radius and each element must follow a different stress-strain curve. The assumptions of the Palmer analysis are, therefore, not valid for sands.

The stress-level dependence of moduli in sand obtained from pressuremeter tests has been recognized by previous investigators. It has been used to explain the increase in modulus observed when several unload-reload loops are carried out over the course of a PM test or to explain scatter in measured values. Robertson (1982) suggested that the modulus should be normalized to the in situ stress level using the
Janbu (1963) expression:

\[ G = k_0 p_e (\sigma_m^*/p_e)^n \]  \hspace{1cm} (5.8)

where \( k_0 \) is the modulus number

\( p_e \) is a reference pressure

\( \sigma_m^* \) is the mean normal stress

\( n \) is an exponent.

Clarke and Wroth (1985) showed that the scatter observed when \( G_w \) values were plotted versus depth could be greatly reduced by plotting \( G_w \) versus mean effective stress. However, it is not clear which is the relevant stress applicable to a particular modulus value.

### 5.2.3.1 Effect of Stress Level

The estimation of a suitable stress level with which to associate the measured modulus requires consideration of the stress state around the probe. The calculated distribution of plane strain mean normal stress, \( \sigma_m^* \), with radius, assuming cylindrical cavity expansion in a linear-elastic, frictional-plastic material with \( \phi = 40^\circ \), is shown in Figure 5.8 for various values of cavity pressure. The effect of volume change has been neglected. Parameters have been normalized as follows:

- the radius has been normalised by the cavity radius;
- the cavity pressure by the initial horizontal stress;
- and the mean normal stress by its initial value, \( \sigma_{bo}^* \).
Cylindrical Cavity Expansion

Mean Normal Stress Distribution, $\phi = 40$

\[ \sigma_m' = \frac{\sigma_r' + \sigma_\theta'}{2} \]

\[ p_o' = \text{in situ} \]

horizontal stress

Figure 5.8 Calculated Variation of Mean Stress with Radius for Varying Cavity Stress Levels.
It is clear that as the test proceeds, the stress level close to the membrane increases greatly and the zone of soil experiencing significant stress increase becomes larger. For the simplified soil model considered, the mean normal stress remains constant prior to the limiting stress ratio being reached and so the stress level increases only within the plastic zone.

The variation in stress level results in a variation in modulus from a high value close to the cavity to the in situ value at the elastic-plastic interface and beyond. The deformation measured at the cavity wall will thus reflect the average behaviour of the soil being deformed.

Bellotti et al. (1987) suggested using the Janbu (1963) expression and the average stress in the plastic zone given by:

\[
\bar{\sigma}_n = \frac{1}{\pi} \int_{0}^{R_p} \frac{\sigma'_m \, dr}{R_p} = \frac{p'_c \left( \frac{R_p}{a} \right)^N - 1}{(1 - \sin \phi) \left( \frac{R_p}{a} - 1 \right)}
\]

(5.9)

where \( p'_c \) is effective cavity pressure;

\[
\sigma'_m = 0.5 \left( \sigma'_r + \sigma'_s \right);
\]

\( R_p \) is radius of the plastic zone;

\( a \) is radius of the cavity;
\[ N = \frac{1 - \sin \phi}{1 + \sin \phi} \]

The average pressure can be expressed in the form:

\[ p'_{ave} = \chi p_c' \]  \hspace{1cm} (5.10)

\[ \chi = \frac{1}{1 - \sin \phi} \frac{\left[ (R_p/a)^{n-1} \right]}{\left[ (R_p/a-1) \right]} \]

One deficiency of this method is that by integrating along a radius, the expression fails to incorporate the larger volume of soil subject to increased stress as the test progresses. A better expression may be derived by considering thin annuli of soil around the cavity. The expression obtained is:

\[ p'_{ave} = \frac{P_{ave}}{(R_p/a)^{n-1} - 1} \frac{\left[ (R_p/a)^{n-1} - 1 \right]}{\left[ (R_p/a)^{n-1} - 1 \right]} \]  \hspace{1cm} (5.11)

The values obtained by both expressions are plotted against cavity pressure in Figure 5.9. Both expressions result in the calculated average stress level becoming a smaller proportion of the cavity stress as the plastic zone increases in size, i.e. as the cavity pressure increases. The value of \( p_{ave} \) suggested by Robertson (1982) [i.e. \( p'_{ave} = 0.5 (p_c - u) \)] is also shown. The Robertson assumption of variation with \( (p_c')^n \) suggests a more rapid increase in \( G_w \) over the course of a test than for variation with the average stress level in the plastic zone. However, there does not appear to be any theoretical justification for considering the stress level only in the plastic zone. For the purpose of the following discussion, the Bellotti et al. (1987) expression will be used.
Average Stress in Plastic Zone
Comparison of Bellotti and Howie Method

\[ \text{Pave} = 0.5 \text{ Pc} \] (Robertson, 1982)

Figure 5.9  Variation of Average Stress in Plastic Zone with Normalized Cavity Pressure.
If it is assumed that the modulus measured at the cavity wall is an average modulus which is a function of \( p_{\text{ave}}' \), i.e.

\[
G_{\text{ave}} = k_g p_a (p_{\text{ave}}'/p_a)^{0.5} \tag{5.12}
\]

and that the modulus at the in situ stress level is:

\[
G_o = k_g p_a (p_o'/p_a)^{0.5}, \tag{5.13}
\]

then it follows that:

\[
G_{\text{ave}}/G_o = (p_{\text{ave}}'/p_o')^{0.5} \tag{5.14}
\]

The exponent \( n \) has been taken to be 0.5 in the above equations as suggested by Byrne et al. (1987). Figure 5.10 shows the effect of varying the radius of the zone influencing the modulus measured at the cavity wall. It can be seen that for a small radius of influence (say \( R/a = 2 \)), the modulus should at least double during a PM test in which the cavity pressure increases ten times. Alternatively, if a large radius of influence is assumed, an increase in measured modulus of approximately 20% or 30% would be expected. Also shown on Figure 5.10 is the increase in modulus to be expected if the radius of influence coincides with the radius of the plastic zone. It is clear, therefore, that a rational basis must be developed to account for the effect of stress level on the measured modulus. First, some attention must also be given to the degree of unloading.
Figure 5.10  Effect of Assumed Radius of Influence on Average Modulus.
For an UR loop to remain elastic, the unloading must not be so large that plastic strains are induced (Wroth, 1982). Mair and Wood (1987) recommended that, as soil does not respond as an elastic-plastic material, the degree of unloading should be kept considerably below the limits suggested by Wroth if significant hysteresis is to be avoided. In addition, it has been noted that in sands, a rotation in principal stresses results in a considerable change in stiffness (Ladd et al., 1977). This would occur if unloading was such as to result in the circumferential stress, $\sigma_c'$, exceeding the radial stress, $\sigma_r'$. It is, therefore, clear that the degree of unloading is an important parameter in the determination of $G_{ur}$.

For a cylindrical cavity in linear elastic isotropic soil, it can be shown that the distribution of strain around the cavity varies with $1/r^2$ as shown in Figure 5.11. Thus the strain has reduced to 1% of that at the cavity wall within ten radii. Figure 5.12 shows the variation of shear modulus with strain level given by Seed and Idriss (1970). $G_{max}$ is the approximately constant modulus for strains less than $10^{-4}$%. It is commonly measured using the resonant column test. Negussey (1985) showed that the static initial tangent modulus in the triaxial test tended towards the modulus measured in the resonant column as precise strain measurement allowed more accurate definition of the tangent modulus. Therefore, for the strain distribution in Figure 5.11 and assuming a uniform stress level with radius, the Seed and Idriss curve indicates that the soil close to the cavity wall would exhibit softer behaviour than that more remote from the cavity with $G$ tending to $G_{max}$ at large radius. This is the opposite effect to that of stress level.

Again, the problem of assigning a relevant strain level to the measured modulus arises. Robertson (1982) suggested that an average strain of $0.45\Delta\gamma_c$ be taken to be the strain increment relevant to $G_{ur}$ where $\gamma_c$ is approximately twice the cavity strain increment. Bellotti et al. (1987) suggested $0.4\Delta\gamma_c$. In order to reduce variability between moduli due to strain level effects, Bellotti et al. tried to carry out loops with
Cylindrical Cavity Expansion

Elastic Strain Distribution Around PM

Figure 5.11 Variation in Strain Increment with Radius for Linear Elastic Soil.
Figure 5.12
Variation of Shear Modulus with Shear Strain for Sands

Shear Strain, \( \gamma \) - percent

Hrange of values for data

Shear Modulus at Shear Strain, \( \gamma \)
an approximately constant value of shear strain increment of 0.2%. More recently, Bellotti et al. (1989) have suggested that the relevant strain increment is the average strain in the plastic zone. There is no apparent theoretical justification for the above assumptions.

The deformation measured at the cavity wall is thus the result of soil deformation occurring in the zone around the PM at a range of stress and strain levels. The following section attempts to consider the combined effects.

5.2.3.3 Combined Effect of Stress and Strain Level

The combined effects of stress and strain level can now be assessed. A further factor not discussed above is the observation by Yu and Richart (1984) that the small strain modulus, $G_{\text{max}}$, reduces as the shear stress ratio increases as is shown in Figure 5.13. All the above effects can be accounted for in an approximate fashion as follows.

The stress level prior to unloading at any radius can be calculated using equation 5.13 in the plastic zone and can be taken to be $p_a$ in the elastic zone. The initial unloading modulus, $G_{\text{max}}$, at any radius $r$ is then calculated using the expression [Yu and Richart, 1984]:

$$G_{\text{max}} = A F(e) P_a \left( \frac{\sigma'}{P_a} \right)^{0.5} (1 - 0.3k_n^{1.5})$$

where

$$F(e) = \frac{(2.17 - e)^2}{1 + e}$$

and $A = 700$ for rounded sand,
Figure 5.13  Variation of $G_{\text{max}}$ with Stress Ratio (after Yu and Richart, 1984).
\[ F(e) = \frac{(2.97-e)^2}{1+e} \]

and \( A = 326 \) for angular sand, and

\[ k_n = \frac{(\sigma'_r - 1)}{(\sigma'_r - \sigma'_e)_{max} - 1} \]

Equation 5.15 was derived from resonant column testing. The shear induced changes in void ratio during loading can be computed but were found to have a small effect on the calculated values of \( G_{\text{max}} \) (Byrne, Salgado and Howie, 1990).

A finite element analysis was then carried out using a plane strain axisymmetric domain [Byrne and Salgado, 1990] to investigate the effects of degree of unloading on the apparent modulus at the cavity wall. The soil was assumed to behave in a non-linear elastic manner with the tangent modulus at any radius \( r \) decreasing according to the hyperbolic expression:

\[ G(r) = G_{\text{max}} \left( 1 - \frac{\Delta \tau}{\tau_L + \tau_f} \right) \quad \text{5.16} \]

where \( \Delta \tau, \tau_L, \) and \( \tau_f \) are as shown in Figure 5.14.

The unloading was carried out in a number of small steps with the cumulative displacement at the cavity wall being used at each stage to compute the cavity strain increment. The unload-reload modulus, \( G_u \), was calculated for a particular degree of unloading, \( \Delta p_c' \). The results are presented in Figure 5.15 for a range of stress levels. The figure shows the predicted variation of \( G_u/G_{\text{max}} \) with normalized cavity stress, \( p_c'/p_c' \), for various degrees of unloading. The analysis predicts that, at any value of \( \Delta p_c'/p_c' \), the modulus ratio, \( G_u/G_{\text{max}} \), increases gradually as the cavity stress increases. At a given stress level, a
Figure 5.14  Assumed Unload Stress-Strain Behaviour.
Figure 5.15  Variation of $G_{ur}/G_{max}$ with Cavity Stress Level for Variable Degree of Unloading (adapted from Byrne, Salgado and Howie, 1990).
greater degree of unloading will result in a lower modulus ratio. The dip in the curves is a consequence of the assumed soil model, as the stress ratio effect depicted in Figure 5.13 causes a softening of the modulus in the elastic zone which is not balanced by a stiffening due to increasing mean normal stress level.

Some investigators have attempted to reduce the effect of strain level by maintaining the strain increment at the cavity wall approximately constant (e.g. Bellotti et al., 1986). A constant strain increment implies that, as the soil stiffens, the magnitude of the unloading must increase. As $G_w$ is likely to increase as a function of the square root of the cavity pressure or less, it seems reasonable to deduce that for a constant strain increment, $\Delta p'_c/p_c$ will decrease over the course of a test. Figure 5.15 would suggest that the modulus should therefore increase at a faster rate than for a test conducted with the amount of unloading a constant proportion of the cavity pressure. Alternatively, by increasing the size of the strain increment as the test proceeds, it would be possible to obtain a relatively constant value of measured modulus. For example, consider Figure 5.16 which is an idealized PM curve with the pressure normalized to the in-situ horizontal stress. If the degree of unloading at $p'_c/p_c = 4$ is $0.2p'_c$, then, according to Figure 5.15, at a normalized cavity stress of 8, unloading by $0.46p'_c$ should give the identical modulus. This ignores any effect of reversal of principal stresses.

The above analysis suggests that the unload-reload modulus is not indeed a true modulus but is an apparent modulus reflecting the average behaviour of soil elements at a variety of stress and strain levels. In order for the parameter to be useful in engineering design, some relationship must be sought between it and other soil parameters. The following section examines some data from SBPM tests available in the literature in an attempt to discern such a relationship.
Figure 5.16 Variation of Measured Modulus with Test Procedure.
5.2.3.4 SBPM Results

Due to the natural variability of sand deposits, it is difficult to use field data to investigate stress and strain level effects. Bellotti et al. (1987) have published the results of a large number of PM tests carried out in the calibration chamber at ENEL-CRIS, Milan, Italy using a Cambridge SBPM. Two types of test were conducted:

- where the sand was rained into the chamber around an already installed PM ("ideal" installation);
- the probe was self-bored into the chamber after sample preparation.

A range of relative densities and overconsolidation ratios was tested and chamber size effects were considered negligible. Of the 48 tests, 41 were carried out with a constant stress on the chamber boundary. During each PM test, several unload-reload loops were performed, the shear strain increment being maintained at approximately 0.2%.

Figure 5.17 shows the measured $G_{ur}$ values plotted vs. cavity pressure at the start of unloading for tests in Ticino sand with Relative Density ($D_r$) between 60% and 80%. Values from each particular test have been connected together. There is a general trend for $G_{ur}$ to rise as the cavity stress increases. However, the increase with cavity stress level within any one test appears to be at a much lower rate than is the increase from test to test as shown by the trend line drawn through the first value in each test in Figure 5.17. This reflects the importance of initial stress state to soil stiffness.

In order to investigate the effects of stress and strain level within individual tests, it is necessary to reduce the effects of variable relative density and initial stress level. This can be done by dividing $G_{ur}$ by $G_{max}$.
Figure 5.17  Unload-Reload Modulus from SBPM Tests in Calibration Chamber. (D_r=60 - 80\% Ticino Sand).
obtained from resonant column tests on identically prepared specimens at a similar initial stress level and by dividing $p_c'$ by the initial plane strain average stress in the horizontal plane, $p_0'$. Figure 5.18 shows the data from Figure 5.17 plotted in this fashion. The values of $G_{ur}/G_{\text{max}}$ fall between 0.3 and 0.7. There is a tendency for a gradual increase in $G_{ur}/G_{\text{max}}$ with stress level. A similar trend is exhibited for tests in sands with $D_r$ between 40% and 50%, as shown in Figure 5.19. For this range of $D_r$, $G_{ur}/G_{\text{max}}$ falls between 0.2 and 0.7.

In Figures 5.18 and 5.19, a distinction has been made between those tests in which the PM was drilled into the sample ("self-bored" tests) and those in which the sample was placed around the PM ("ideal" tests). The values of $G_{ur}/G_{\text{max}}$ tend to be lower for "self-bored" tests than for "ideal" tests, this difference being more noticeable for the looser samples. Only one "self-bored" test gives $G_{ur}/G_{\text{max}}$ comparable to the "ideal" tests in Figure 5.19 and it is interesting to note that, for this test, the relaxation time between installation and testing was 96 hours as compared to between 1 and 3 hours for the other tests. The lower values of $G_{ur}/G_{\text{max}}$ for the self-bored tests may indicate a disturbance effect caused by instrument installation which is reduced if sufficient time is allowed to elapse before the test begins. The improvement with time may be an aging effect as previously observed for sand by Schmertmann (1970) and Mitchell and Solymar (1984). If the measured modulus was only influenced by the soil in the plastic zone as is assumed in Bellotti's method of accounting for stress and strain level, one would expect the shearing during expansion to eliminate any aging effects on $G_{ur}$.

When only the ideal tests are considered, $G_{ur}/G_{\text{max}}$ falls mainly between approximately 0.5 and 0.7 for both ranges of relative density. The data tend to suggest that even installation of the SBPM in a very carefully-controlled manner leads to disturbance of the soil sufficient to cause a reduction in the unload-reload modulus. As discussed in Section 5.2.1, recently deposited sand may be more sensitive to disturbance than natural sands.
Figure 5.18  $G_{ur}/G_{\text{max}}$ Versus Normalized Cavity Stress (Ticino Sand, $D_t = 60 - 80\%$).
Figure 5.19  $G_{ur}/G_{max}$ Versus Normalized Cavity Stress (Ticino Sand, $D_r = 40 - 50\%$).
For most of the tests, $\Delta p_c'/p_c'$ varies between 0.15 and 0.35. There is no discernible trend with magnitude of unloading although the two "ideal" tests which plotted lowest on Figure 5.19 were conducted with $\Delta p_c'/p_c'$ between 0.3 and 0.48. This lack of trend is not surprising given the small range of values and the precision required to measure the very small movements occurring. Figure 5.15 indicates that for this range of $\Delta p_c'/p_c'$, $G_{ur}/G_{\text{max}}$ would be expected to range between about 0.7 and 1.1. The predicted magnitudes of $G_{ur}/G_{\text{max}}$ are thus considerably higher than the values measured. This difference may be due to inadequacies of the soil model. Alternatively, the difference may be due to the effects of anisotropy on $G_{ur}$. However, the correct trend is predicted.

Figure 5.20 presents the results of SBPM tests carried out in Po River Sand by Bruzzi et al. (1986) using a Cambridge SBPM. Values of $G_{\text{max}}$ were obtained from shear wave velocity, $V_s$, measurements in crosshole tests. The sand is medium dense and is predominantly quartz. The values of $G_{ur}/G_{\text{max}}$ lie between 0.4 and 0.85 for $p'_c/p'_o$ between 2 and 9. Most values of $G_{ur}/G_{\text{max}}$ are between 0.5 and 0.7 with little increase due to increasing stress level being apparent. The values are in reasonable agreement with those observed in the "ideal" chamber tests. From this it can be deduced either that the disturbance due to PM installation observed in the chamber tests was unique to the chamber tests as suggested by Bellotti et al. (1987) or that the true in situ stiffness ratio was higher than 0.5 to 0.7. The latter explanation seems the more credible because of the much greater age of in situ Po River Sand than the chamber test samples. If this is the case, then the utility of the SBPM must be doubted for measurement of fundamental soil properties if it cannot be inserted without disturbance even under highly-controlled research conditions. It may also explain why it has been observed that $G_{ur}$ is insensitive to disturbance i.e. all PM installation causes disturbance. Nevertheless, $G_{ur}$ is still an indicator of the soil stiffness.
Figure 5.20 $G_{uw}/G_{max}$ Versus Normalized Cavity Stress (In Situ Values in Po River Sand).
In summary, therefore, the test data examined indicate that the modulus measured in the PM test is dependent upon the stage of the test at which the unload-reload cycle is performed and upon the procedure used to obtain it. It also appears that the modulus obtained is affected by disturbance during PM installation. In addition, the analysis presented indicates that the modulus obtained is an "apparent" modulus which reflects the composite behaviour of the soil elements around the cavity. As such, it is not a fundamental soil property but is an indicator of the stiffness of the soil under a given loading. Similarly, a plate bearing test can be used to infer an average modulus of the soil being tested. Given that \( G_{ur} \) is an indicator of soil stiffness, it is necessary to devise a means of relating it to a parameter which will be of use in engineering design. This is examined in the next section.

5.2.3.5 Relationship between \( G_{ur} \) and Soil Stiffness

Bellotti et al. (1989) presented an analysis of the above chamber test data in which they corrected the measured \( G_{ur} \) using the average stress in the plastic zone and plotted the corrected value, \( G_{ur}^{c} \), normalized to \( G_{max} \) versus half the calculated average shear strain in the plastic zone on the Seed et al.(1986) diagram. The Seed et al.(1986) envelope is very similar to that of Seed and Idriss(1970). The value of half the calculated average strain was chosen because Seed et al. used double amplitude shear strain in preparing their figure. The use of the average strain calculated over the increasing size of the plastic zone means that, for a constant cavity strain increment, the calculated average shear strain over which \( G_{ur} \) is being measured reduces as cavity stress and hence the radius of the plastic zone increases. The results are shown in Figure 5.21. Reasonable agreement is obtained between the calculated data and the relationship of Seed et al.(1986).

Figure 5.22 shows the values of \( G_{ur}^{c}/G_{max} \) for chamber test data after correction using the average stress in the plastic zone. This procedure can be seen to have resulted in almost constant moduli over the range
Figure 5.21 Comparison between Seed et al. (1986) curve and $G_{ur}/G_{max}$ vs. Strain Increment for Chamber Test Data (adapted from Bellotti et al., 1989).
Figure 5.22 $G_u/G_{\text{max}}$ Versus Normalized Cavity Stress Level for Tests with Ideal Installation from Bellotti et al. (1989).
of stress levels used in the tests. For the test procedure followed in these tests, the use of such an average stress appears to remove the effect of stress level from the data obtained. However, as shown in Figure 5.16, different test procedures can produce different moduli.

An alternative and much simpler approach to the interpretation of $G_{ur}$ values is illustrated in Figure 5.23. Figure 5.23 shows $G_{ur}/G_{max}$ for the "ideal" chamber tests in uncorrected form plotted against $0.15\Delta \gamma_c$. The values of $G_{ur}/G_{max}$ now fall between 0.4 and 0.8. Very reasonable agreement is obtained between the Seed et al. (1986) envelope and the plotted data.

It is clear, therefore, that good correlation can be obtained between $G_{ur}$ and the relationship of Seed et al. or Seed and Idriss in a number of ways. One reason for this is that the strain axis in the Seed and Idriss attenuation relationship is a logarithmic scale and so small changes in calculated strain level have a small effect. The Italian procedure is really a semi-empirical correlation which gives reasonable agreement between the pressuremeter modulus, $G_{ur}$, and the strain attenuation relationship of Seed and his co-workers. The calculation of average stresses and strains gives it an apparent theoretical basis but there is no clear rationale for the procedure. The use of the average stress in the plastic zone for correction for stress level does appear to have the effect of removing the tendency of the measured modulus to increase with cavity stress level. It is emphasized, however, that although this is the case for the particular test procedure followed in these tests, it may not hold for tests conducted in a different manner. Even without elaborate corrections, it has been shown that a reasonable correlation can be derived between $G_{ur}/G_{max}$ and a laboratory attenuation relationship for sands. Although this is the case, the relationship between the measured modulus and the shear modulus at the in situ stress level is unknown. The arguments presented above suggest that the insitu value is likely to be higher than $G_{ur}$ from PM tests.
Figure 5.23  \( G_{\omega}/G_{\text{max}} \) Versus Cavity Shear Strain Increment for Chamber Tests.
The pressuremeter modulus in sands is therefore another index property which can be related to soil properties by empirical or semi-empirical correlation. Given the scatter observed in the chamber tests and the effects of disturbance, there seems little justification for elaborate correction methods. However, the stress levels in the chamber tests seldom exceeded eight times the initial horizontal stress. Where higher stress levels are attained, it may prove necessary to carry out some correction for stress level. For this, the use of the average stress in the plastic zone appears potentially useful for cycles carried out over approximately constant cavity strain increments. It might be argued, however, that if laboratory attenuation relationships are considered adequate for design use then it becomes only necessary to measure G\text{max} at the in situ stress level and use the Janbu relationship and the Seed et al. (1986) envelope to derive moduli at other stress and strain levels. Further research is required.

In order to confirm or refute the above observations, it is desirable that the method of measurement of unload-reload modulus be standardised in order that variations in modulus due to test procedure may be minimized. Only then may correlations be developed between G\text{ur} and G\text{max} and other soil parameters or may empirical design methods be developed. It is believed that the unload-reload cycle should be carried out sufficiently late in the test to ensure that the bulk of the soil being stressed has not been disturbed by installation. In addition, it is suggested that the degree of unloading should be a constant proportion of the cavity stress which should not exceed 0.4p_c'. This will be easier to control in the field than a loop of, say, 0.2% shear strain. The degree of unloading should be sufficiently large to ensure that enough points are obtained to define the loop but that a large degree of hysteresis is avoided. The standardisation of the test method also requires that instrument errors be minimized by careful instrument design, calibration and maintenance.
5.2.3.6 Summary of Modulus

The unload-reload modulus appears to be the most promising method of determining soil stiffness from SBPM tests. In sands, due to the variation of soil properties with stress and strain level, the assumption that soil elements at all radii will follow identical stress paths is untenable.

An attempt has been made to investigate the combined effects of stress and strain levels. The analysis indicates that the modulus obtained will depend greatly on the procedures followed as the increase in stiffness with stress level can be modified by variations in the degree of unloading. The modulus, $G_u$, appears to be an indicator of the soil stiffness rather than a true measurement of shear modulus. It is suggested that $G_u$ should be measured in a standard manner and that correlations should be established between it and engineering parameters. Reasonable agreement was obtained between the Seed et al. (1986) modulus attenuation curves and $G_u/G_{max}$ plotted versus 0.15 times the equivalent increment of shear strain. There are indications that even in ideal conditions, installation of the SBPM results in disturbance and the measured moduli are lower than the undisturbed values.

It is important to follow a consistent test method in order to minimise variations in measured values. Only then will it be possible to begin to resolve such issues as the detection of modulus anisotropy, i.e. theoretically, the SBPM gives a stiffness in the horizontal plane, the seismic cone gives a stiffness in a vertical plane.

5.3 SUMMARY OF CHAPTER 5

A major attraction of the PM and particularly the SBPM is the existence of closed form solutions to the expansion of a cylindrical cavity. It is possible to incorporate most of the significant aspects of soil
behaviour into the analysis methods. The major hindrance to the successful use of the PM is the inevitable disturbance caused to the soil during the installation of the instrument. Disturbance is difficult to quantify and can have a large effect on the results of the analysis.

Only the SBPM offers the possibility of direct measurement of in situ horizontal stress. Reasonable results have been obtained in soft and medium stiff clays. Evidence of success in stiff soils and sands is scarce.

For shear strength determination, methods are available for determining the complete stress-strain curve in both sand and clay. Disturbance can lead to erroneously high values of shear strength. For sands, the new method proposed by Manassero (1987) is conceptually good but the Hughes, Wroth and Windle (1977) method with the Robertson and Hughes (1986) correction appears to be more consistent. However, such are the potential errors due to rate effects and disturbance that it may be unwise to rely on analytical methods of interpretation.

Methods of determining modulus from the early part of the expansion curve are very prone to error due to disturbance. Consequently, the unload-reload modulus, \( G_u \), is the favoured method of determining soil stiffness. It has been shown to be relatively unaffected by disturbance. The degree of unloading over which \( G_u \) is measured has been shown to be of importance. In addition, it has been shown that interpretation of modulus in sand is complicated by the stress and strain level dependence of stiffness and that \( G_u \) reflects the average behaviour of the soil but does not represent a fundamental parameter.

The following chapter will examine the applicability of the above interpretation methods to FDPM tests.
6.0 ANALYSIS AND INTERPRETATION OF FULL DISPLACEMENT PRESSUREMETER TESTS IN SANDS

6.1 INTRODUCTION

The interpretation of PM tests in sands is greatly influenced by soil disturbance during installation, the degree of which is extremely difficult to assess from test to test. In the previous chapter, it was indicated that, even under ideal conditions, it is difficult to avoid disturbance. It seems reasonable, therefore, that if the instrument can be inserted into the soil in a consistent manner such that disturbance is reproducible, it might then be possible to develop interpretation procedures which produce consistent soil properties from test to test. This and the fact that the unload-reload modulus appeared to be insensitive to disturbance led to the development of the FDPM (Hughes and Robertson, 1985). In addition, Withers et al. (1986) suggested that, if after full-displacement installation, the pressuremeter element could be expanded sufficiently to allow stressing of a large body of the soil unaffected by probe insertion, this previously-undisturbed soil would dominate the observed behaviour. The pressure-expansion curve could then be expected to duplicate SBPM behaviour at larger strains. The later portions of the expansion curves could then be analysed using conventional methods. This chapter examines the applicability of such reasoning.

6.2 EFFECTS OF INSTALLATION

The deformation pattern around the tip of an advancing cone is complex and is difficult to model analytically. Very high stresses occur on the face of the cone and, from back-analysis of friction sleeve measurements and from pore-pressure measurements in clays, it appears that substantial unloading occurs in the region of the shoulder of the cone. A possible stress history for an element in the path of an
advancing cone is presented below. It is based on the arguments presented by Hughes and Robertson (1985) and Withers, Howie, Hughes and Robertson (1989). The stress path is shown only for the horizontal plane. Significant changes in vertical stress will also occur.

6.2.1 **Stress Paths During Cone Penetration**

As the cone approaches a soil element, the radial stress rises until the peak stress ratio, $\sigma'/\sigma'_s$, is reached (point B in Figure 6.1). At this point, the shear strength of the soil element has been fully mobilized. As penetration proceeds, the stress path climbs the failure line, pushing the yield surface out as it does so. As the soil element passes the shoulder of the cone (i.e. at point C), substantial reduction of the radial stress has been observed to occur. The unloading is likely sufficient to cause failure with the circumferential stress as the major principal stress. As unloading continues, the yield surface is pulled inward i.e. failure in extension erases the soil's memory of previous stress and strain history. Lift-off pressures from FDPM tests indicate that unloading ceases when the radial stress has dropped to a value of the same order of magnitude as the initial in situ stress. Points at a greater distance from the cone will undergo similar stress paths but the maximum stresses will be lower (see dotted line in Figure 6.1). Hughes and Robertson (1985) suggested that arching may occur during the unloading resulting in locked-in hoop stresses (i.e. high residual circumferential stresses) at some distance from the probe surface after installation. Withers, Howie, Hughes and Robertson (1989) suggested that the element of soil adjacent to the wall of the cavity would have been subjected to intense shearing and suggested the possible locus of stresses after penetration illustrated schematically in Figure 6.2(b). The highest stresses are postulated to occur midway through the plastic zone. Beyond the previously stressed zone, the soil is assumed to have been unaffected by the insertion process. The resulting stress distribution around the pressuremeter element prior to inflation will thus be extremely complex and very unlike that existing around a SBPM after installation.
Figure 6.1 Hypothetical Stress Paths for Soil Elements Around an Advancing Cone in Sands (after Hughes and Robertson, 1985).
Figure 6.2  Stages in FDPM Testing (adapted from Withers, Howie, Hughes and Robertson, 1989).
6.2.2 Stress Paths During Inflation and Deflation of FDPM

Under the assumed soil model, when inflation of the pressuremeter begins, the soil will behave elastically until the yield surface is encountered. If sufficient pressure is applied, the soil will reach failure and the soil element will climb the failure line, pushing the yield surface ahead of it (see Figure 6.2(c)). If the pressuremeter is capable of sufficient expansion, a pressure will be reached at which the soil in the zone affected by FDPM insertion will have been returned to stresses higher than those previously experienced. From this point on, the effect of pressuremeter expansion may be similar to that experienced by the soil around a SBPM.

After inflation is complete, the pressure is reduced causing the membrane to contract. The stress path will be similar to that hypothesized for unloading of a soil element as it passes the shoulder of cone. Initially, the deformation will be elastic as the stress path drops below the current yield surface. When the yield surface is again encountered, plastic straining will begin (see Figure 6.2(d)). If a simple elastic-plastic model is assumed, the soil will remain elastic until the peak stress ratio is reached with the circumferential stress as the major principal stress. From this point on, the soil next to the membrane will deform plastically.

As unloading proceeds, the new plastic zone will expand. It has been observed that a point is reached at which the stresses arch and the water pressure controls deflation (Wroth, 1982). Houlsby et al. (1986) showed that the radius of the plastic zone, \( R_p \), during unloading would be less than during expansion.

If at any point during the expansion (or contraction) the pressure is reduced (or increased), the stress path will move inside the current yield surface and the soil will behave elastically, provided unloading is small enough to avoid any plastic strains. Upon reloading, the soil will behave elastically until the yield surface
is again encountered. This is the same theoretical justification for the measurement of unload-reload shear modulus, $G_w$, in the SBPM test. If the unload-reload loop is carried out in the later stages of the expansion when the effect of installation has been largely erased, then the moduli obtained from both the FDPM and SBPM tests may be expected to be similar. However, if the cycle is carried out early in the expansion, it is possible that the stress and strain conditions in the zone affected by installation may result in $G_w$ values much different from those obtained from SBPM tests. In addition, the soil immediately next to the probe has been greatly disturbed and may have been densified or may have dilated to a lower density than prior to probe insertion. It is also possible that the stresses during insertion may have caused some particle crushing. Whether the FDPM values of $G_w$ (even when measured late in the test) agree with those from the SBPM will depend on whether soil in the near-field or far-field dominates behaviour.

6.2.3 Approximate Analysis of Extent of Disturbance

Vesic (1977) suggested that the bearing capacity of piles in sand could be estimated by analyzing the failure mechanism shown in Figure 6.3. He stated that the pressure on surface BD could be calculated using the theory of spherical cavity expansion. Mitchell and Keaveny (1986) found good agreement between measured friction angles and those predicted using Vesic's (1977) approach for chamber tests on normally consolidated sands. Greeuw et al. (1988) found that Vesic's theory predicted cone penetration resistances close to those measured in chamber tests when the friction angle in the plastic zone was assumed to be $\phi_p$. It appears, therefore, that Vesic's (1977) cavity expansion theory describes the behaviour of the soil around a penetrating cone reasonably well.

An estimate of the extent of the plastic zone during cone penetration can thus be obtained using spherical cavity expansion theory. Figure 6.4 shows the calculated radius of the plastic zone normalized by the cavity radius, $a$, against penetration resistance, $q_c$, normalized by the initial vertical effective stress for
I Compressed wedge
II Radial Shear Zone
III Plastic Zone

Figure 6.3 Assumed Failure Pattern Under Pile Point (adapted from Vesic, 1977).
Figure 6.4  Radius of Plastic Zone Around Cone Tip Predicted by Spherical Cavity Expansion Theory.
normally consolidated quartz sand, assuming \( K_o = 1 - \sin \phi \). The soil has been assumed to be linear elastic-frictional plastic and volume change effects have been ignored. Treadwell (1975), in a study of disturbance around a cone penetrometer, found that the zone of disturbance ranged from 6 to 20 radii, so the values in Figure 6.4 appear reasonable. Due to the effect of stress level on friction angle, the soil friction angle will vary with radius but this has been ignored in the present analysis.

The stress path for a soil element in the vicinity of the cone during penetration can be idealized as shown in Figure 6.5. The soil at the elastic-plastic boundary can be assumed to fail at point B as the cone tip penetrates (based on spherical cavity expansion) and will then undergo plane strain unloading as the cone shoulder passes the soil element i.e. cylindrical cavity expansion. For the assumed soil model, the mean normal stress remains constant during elastic behaviour. It can be shown that for spherical cavity expansion \( \Delta \sigma_r' = -\Delta \sigma_r/2 \). In the cylindrical theory of cavity expansion, it can be shown that \( \Delta \sigma_r' = -\Delta \sigma_r \) during elastic behaviour and so the unloading path for the soil element at the elastic-plastic boundary can be represented by BG in Figure 6.5. The magnitude of the unloading is unknown but it is unlikely that failure will occur at the elastic-plastic boundary on the extension side (Houlsby et al. (1986)). In Figure 6.5, it has been assumed that the radial stress at the elastic-plastic interface has returned to the initial horizontal stress.

In order to stress the soil element at \( r = R_p \) beyond the stress state experienced during insertion of the FDPM, it will be necessary to return the soil stress state to point B. Figure 6.6 shows the calculated cavity pressures in cylindrical cavity expansion to return the stresses at a given \( (R_p/a)_{spherical} \) to the equivalent of point B in Figure 6.5. For the above analyses, no allowance has been made for the difference in friction angle with stress path i.e. \( \phi_m \) vs \( \phi_u \). It has been assumed that \( K_o = 1 - \sin \phi \). For a previously-disturbed zone of ten cavity radii, a cavity pressure of about 14 times the in situ horizontal
Figure 6.5  Idealized Stress Paths Experienced by Soil Element During Insertion and Inflation of FDPM.
Figure 6.6 Cavity Pressures Required to Erase Effects of Cone Penetration Predicted by Idealized Model.
stress is required to return the stress at the edge of the plastic zone during cone penetration to the maximum stress experienced during penetration.

When the stress at \( r = (R_p)_\text{spherical} \) has returned to point B, the size of the plastic zone around the pressuremeter will be much larger than \( r = (R_p)_\text{spherical} \). This is due to the different degrees of symmetry between spherical and cylindrical cavity expansion. In the former, failure occurs at \( \sigma'_r = \sigma'_r[3/(1+2N)] \), and, in the latter, at \( \sigma'_r = \sigma'_r[2/(1+N)] \) where \( N = (1-\sin\phi)/(1+\sin\phi) \). Figure 6.7 shows the extent of the plastic zone calculated using cylindrical cavity expansion theory against \( p_{av}/p_o \) for a range of friction angles.

To illustrate the above calculation, the test at 5 metres at McDonald Farm discussed by Hughes and Robertson (1985) and shown in Figure 6.8 will be reviewed. From the CPT profile, \( q_c/\sigma'_r \) is approximately 85 and the calculated zone of disturbance from Figure 6.4 is about 5 radii. To return the soil at this radius to the stress state experienced during cone insertion, Figure 6.6 indicates the necessary cavity stress to be about 7 times the in situ horizontal effective stress plus the water pressure i.e. about 500kPa. This pressure is reached after only about 3% strain in the FDPM test. At this stress, the plastic zone around the FDPM would extend to about 7.5 radii.

Although the above analysis is simplistic, it does suggest that it is possible to carry out a pressuremeter test after the probe has been inserted in a full-displacement manner in which the bulk of the soil being stressed has been relatively unaffected by probe insertion. Further support for this concept is shown in Figure 6.9, the result of a test using the FFDPM at a depth of 7.2 m at McDonald Farm. The membrane was inflated to a cavity strain of about 4%, was unloaded and then reloaded. The reloading initially showed a much flatter slope than the first loading but seemed to eventually assume a slope which was very similar to a continuation of the first loading. If this is the case, then analysis methods derived for
Figure 6.7  Extent of Plastic Zone During Inflation.
Figure 6.8  FDPM Test at 5.5 m., McDonald Farm (after Hughes and Robertson, 1985).
Figure 6.9  Re-expansion of FDPM in Sand.
the SBPM may be applicable provided the instrument can be expanded to a sufficiently large cavity strain.

6.3 COMPARISON OF PRESSUREMETER EXPANSION CURVES

During previous studies at UBC, a number of tests were carried out at a depth of about 5 metres. Figure 6.10(a) shows the pressure-expansion curves for two SBPM tests carried out by Robertson (1982) using an earlier version of the HPM which was drilled into the ground rather than jetted. The two curves are of similar shape but the maximum pressure attained is greater for the 5.3m depth test than for the 4.6m test. The measured total horizontal stresses ($p_0$), closing pressures ($p_{cl}$) and pressures at 2.5%, 10%, 15% and 20% strain ($p_5$, $p_{10}$, $p_{15}$ and $p_{20}$) are listed in Table 6.1. The closing pressures at 4.6m and 5.3m are 40 and 45 kPa respectively, indicating a ground water level of approximately 0.6m to 0.8m below ground level. This is reasonable for this site. Also listed in Table 6.1 are test data from Hughes (1984) obtained from the HPM installed by jetting.

Figure 6.10(b) shows FDPM tests carried out by Robertson (1982) at approximately the same depths as the SBPM tests. Table 6.1 gives the lift-off pressures, etc. The curves are shaped slightly differently tending to slope more steeply and to reach higher pressures than the SBPM tests. Neither the SBPM nor the FDPM tests were expanded to greater than about 10% cavity strain.

During this study, five tests were conducted at 5.2 metres depth with the Fugro FDPM (FFDPM) at locations which were all within a plan area approximately 5.0 m. by 2.0 m. All five FFDPM pressure-expansion curves are shown on Figure 6.10(c). All tests were pressure-controlled, a technique similar to that used in the previous studies. Also noted on the plot are the times to reach 20% strain. The lift-off pressures vary from 70 to 120 kPa and the pressures at 20% cavity strain vary from 475 kPa to 765 kPa. The lowest lift-off pressure was measured in the test on Sept. 30th in which the initial...
Figure 6.10  Comparison of SBPM and Three FDPM Instruments, McDonald Farm.
Figure 6.10(cont) Comparison of SBPM and 3 FDPM Instruments, McDonald Farm.
Table 6.1  Comparison of Tests at Approximately 5.0 m Depth - McDonald Farm.

<table>
<thead>
<tr>
<th>SOUNDING (Instrument)</th>
<th>DEPTH (metres)</th>
<th>INSERTION METHOD</th>
<th>p₀ (kPa)</th>
<th>p₂.₅ (kPa)</th>
<th>p₁₀ (kPa)</th>
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<td>HPM-1 (HPM)</td>
<td>5.3</td>
<td>Bored</td>
<td>255</td>
<td>408</td>
<td>550</td>
<td>750</td>
<td>550</td>
<td>45</td>
<td>0.63</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>Jetted</td>
<td>110</td>
<td>350</td>
<td>750</td>
<td>750</td>
<td>750</td>
<td>75</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>5.0</td>
<td>Jetted</td>
<td>60</td>
<td>470</td>
<td>825</td>
<td>825</td>
<td>825</td>
<td>825</td>
<td>0.40</td>
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<tr>
<td>FDPM-1 (HPM)</td>
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<td>Pushed</td>
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<td>490</td>
<td>490</td>
<td>490</td>
<td>55</td>
<td>0.58</td>
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<td>FDPM-2 (HPM)</td>
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<td>Pushed</td>
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<td>75</td>
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<td>716</td>
<td>765</td>
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<td>549</td>
<td>576</td>
<td>57</td>
<td>0.54</td>
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<td>257</td>
<td>422</td>
<td>445</td>
<td>493</td>
<td>50</td>
<td>0.54</td>
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<td>5.2</td>
<td>Pushed</td>
<td>75</td>
<td>230</td>
<td>417</td>
<td>454</td>
<td>475</td>
<td>50</td>
<td>0.55</td>
</tr>
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<td>Pushed</td>
<td>73</td>
<td>140</td>
<td>516</td>
<td>600</td>
<td>695</td>
<td>65</td>
<td>0.85</td>
</tr>
<tr>
<td>SCPM-3</td>
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<td>Pushed</td>
<td>40</td>
<td>150</td>
<td>425</td>
<td>490</td>
<td>500</td>
<td>50</td>
<td>0.75</td>
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<tr>
<td>SCPM-4</td>
<td>5.1</td>
<td>Pushed</td>
<td>50</td>
<td>75</td>
<td>244</td>
<td>431</td>
<td>569</td>
<td>50</td>
<td>0.87</td>
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<tr>
<td>SCPM-5</td>
<td>5.1</td>
<td>Pushed</td>
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<td>70</td>
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<td>420</td>
<td>490</td>
<td>50</td>
<td>0.86</td>
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</tbody>
</table>
portion of the inflation was performed very slowly. The closing pressures are very consistent indicating a groundwater elevation of about 1.5m below ground level. The unload-reload loops appear to be of similar slope. Only the test from Nov. 2nd is substantially different in that the pressures attained are much higher. It is interesting to note that this test was the one with the fastest rate of expansion.

The characteristic shape of the curves mentioned in Chapter 4 is visible in all tests, namely that the curves appear to level off to a limiting pressure. Conceptually, this should not occur in sand at these low pressure and strain levels. The apparent limit pressure is due to creep strains occurring at constant pressure. When a new stress increment is applied the rate of strain increases and then decays with time at constant stress.

Figure 6.10(d) shows the expansion curves obtained with the UBC SCPM at a depth of about 5.0m. The curves differ in shape in that they tend to be S-shaped and, in general, do not reach an apparent limit pressure. As was explained in Chapter 4, the S-shaped curves are due, in part, to the instrument being undersized and because of the compressibility of the lantern at low stresses.

The absence of a limit pressure is because the rate of expansion is controlled by the rate at which the pressure developer is run i.e. the tests are strain-controlled. When inflation is stopped, the strain remains constant rather than the stress. The pressure was observed to drop rapidly at first and then more slowly during pauses prior to unload-reload cycles. This is another manifestation of the tendency for sands to creep.

The test curve from Feb. 2nd does not display the S-shape. It is believed that this was caused by sand intruding behind the lantern strips increasing the diameter of the probe and reducing the amount of
strip-flattening possible during inflation. This suggests that tolerance in the probe diameter has an important influence on the shape of the pressure-expansion curve.

The closing pressures are slightly less consistent with the SCPM. This reflects the difficulty of applying the membrane correction during the last stages of deflation when the membrane is sucked in very rapidly. This occurs because of the rigidity of the inflation system. Any slight deficit of oil in the system over the volume of the reservoir will result in the fluid being put into tension as the piston reaches the end of its travel.

A convenient method of comparing the shapes of the curves is to use the parameter, $\beta$, given by the expression:

$$\beta = \frac{(p_{10} - p_{2.5})}{(p_{10} - p_o)}$$

as shown on Figure 6.11. This parameter was suggested by Baguelin (1982) as an aid to classification of soils from SBPM results. The $\beta$ values are listed in Table 6.1 for all tests in which inflation to 10% strain occurred. Tests with the SBPM, FDPM and FFDPM gave reasonably consistent values but the SCPM tests gave values substantially higher because of the lower values of $p_o$ and $p_{2.5}$.

Within each set of tests the maximum pressure reached in each test was quite variable. This large variation is not surprising when the variation of cone penetration resistance at this depth is considered, although less variability might be expected between pressuremeter tests due to the larger volume of soil being tested. Figure 6.12 is a plot of four CPT profiles (Robertson, 1982) obtained close to the location of the FDPM tests. The cone bearing varies from 30 bars to 80 bars at 5m depth. This variability is present throughout the sand at the McDonald Farm site.
Figure 6.11  Definition of Parameter, $\beta$.

\[ \beta = \frac{P_{10} - P_{2.5}}{P_{10} - P_0} \]
Figure 6.12 Comparison of 4 CPT Profiles, McDonald Farm.
In summary, therefore, the pressure-deflection curves for stress-controlled self-bored, jetted and full displacement PM tests were similar in shape. At the larger strains possible with the FFDPM, creep strains became important and resulted in an apparent limit pressure. The strain-controlled SCPM results displayed an S-shaped curve due to problems of instrument tolerance and lantern compressibility and did not tend towards an apparent limit pressure. Creep effects resulted in a drop in pressure during holding phases at constant strain, i.e. strain relaxation. One test with an SCPM enlarged by soil trapped between the membrane and sheath did not display the S-shape. There were no obvious differences between full-displacement and bored or jetted test results.

The above observations suggest that the shape of a FDPM test curve is greatly affected by instrument geometry, tolerances, compliance and test procedure.

6.4. INTERPRETATION

In Section 6.2, it was postulated that, provided a FDPM was capable of sufficient expansion, a condition could be reached in which most of the soil being stressed had been largely unaffected by installation of the instrument. Some typical results will be examined using conventional SBPM methods of interpretation to assess their suitability in interpretation of FDPM results.

6.4.1 Total Horizontal Stress

6.4.1.1 Typical Results

Figure 6.13 summarizes the lift-off pressures measured in this study and compares them to those presented by Hughes and Robertson(1985) in support of their observation that lift-off pressures measured
Figure 6.13  Comparison of Measured Horizontal Total Stress from Self-Bored and Three Full-Displacement PM Instruments (adapted from Hughes and Robertson, 1985).
with the SBPM and FDPM are very similar. The averages of the lift-off pressures for the individual arms are presented. Also shown are contours of total horizontal stress using equivalent $K_o$ values so that it is easier to appreciate the very large degree of scatter in the results. In general, the FFDPM results exceeded those from the other instruments.

### 6.4.1.2 Effects of Equipment Geometry

One potential advantage of the FDPM is that it can be inserted into the ground in a consistent manner which would tend to produce a consistent amount of disturbance in the vicinity of each arm. Figure 6.14(a) shows the lift-off behaviour of the three arms on the FFDPM at a depth of 6.3m. The arms lifted-off at very different pressures. In Figure 6.14(b), the behaviour at closing of the membrane during deflation is shown. The arms closed at identical pressures. Some of the difference in lift-off behaviour may have been due to sticking of the arms, but the closing behaviour indicates that the arms were performing properly once they began moving. Another potential source of the different lift-off pressures was probably differing soil behaviour in the vicinity of each arm. The difference in soil behaviour may have been due to lateral soil variability or to slight differences in the surface features of the probe such as those caused by tolerances or by sand grains lodged under the strips of the sheath causing variable amounts of disturbance. Tests with the PM element at two different distances behind the cone tip showed no discernible difference in test results.

In general, the FFDPM values of lift-off pressure appear to be higher than those obtained with the other instruments. The difficulty of determining the lift-off pressure in tests at shallow depths with the SCPM due to compliance and the membrane correction has already been discussed but most of the values tend to fall in the same range as those obtained by Hughes and Robertson (1985).
Figure 6.14  Comparison of Lift-Off and Closing Behaviour, FFDPM.
6.4.1.3 Comparison to Flat Plate Dilatometer (DMT)

Another instrument which is inserted in a full-displacement manner is the Flat Plate Dilatometer or DMT. The DMT, developed by Marchetti (1980), is a flat plate 95mm wide, 14mm thick and 220mm in length. A flexible stainless steel diaphragm is located on one face of the blade. The membrane is inflated at 20 cm depth intervals using gas pressure. The readings obtained as the membrane begins to move (the A-reading) and at 1 mm deflection (the B-reading) are recorded. After correction for membrane seating and resistance effects, the corrected values of A and B, $P_A$ and $P_B$ respectively, are used in a variety of empirical correlations to derive soil stratigraphy and engineering properties. More detailed discussions of the DMT are contained in Marchetti (1980), Campanella and Robertson (1983), Jamiolkowski et al. (1985) and Schmertmann (1986).

In Figure 6.15, the values of lift-off pressure obtained in FFDPM and SCPM tests are compared to $P_0$ values from a Dilatometer (DMT) sounding. The data in Figure 6.15 show a trend for the DMT to give higher pressures than the FFDPM which, in turn, gives higher lift-off pressures than the SCPM. As there is no sharp change of shape on the DMT blade comparable to the shoulder of the cone, there is probably less unloading around the DMT blade during penetration than there is around a cone. The unloading which occurs behind the conical tip of the FDPM has been previously documented by Hughes and Robertson (1985). Since the SCPM was of smaller diameter than the cone, it is to be expected that the degree of unloading would have been even greater around the SCPM and that the measured lift-off pressures would have been lower. On the occasions when sand intruded behind the lantern strips increasing the diameter and rigidity of the SCPM probe, very high lift-off pressures were recorded as shown in Figure 6.16. The shape of the curve was also much more similar to those obtained with the other pressuremeters. The result shown in Figure 6.16 again illustrates the importance of tolerances and design details on the lift off pressure and shape of the FDPM expansion curve.
Figure 6.15  Effect of Probe Geometry on Lift-Off Pressure, McDonald Farm.
Figure 6.16  Effect of Sand Behind Lantern, SCPM.
6.4.1.4 Effect of Instrument Rigidity

In order to investigate the effects of instrument tolerances on SCPM test results in sands, an attempt was made to push the SCPM with the membrane slightly expanded. This was to see whether the increased diameter would lead to higher lift-off pressures. One result is shown in Figure 6.17.

After a SCPM test at a depth of 4.0 m at the McDonald Farm site, the probe was expanded by approximately 0.5mm or about 2.2% cavity strain. The instrument was then pushed 0.35 m to a greater depth. During the push, the arms moved out [AB in Figure 6.17] as the soil initially squeezed the oil in the lower part of the membrane upward causing a bulge at the arm location. The soil around the membrane before the push is likely to have been softer than that below due to the previous PM expansion test at that elevation. The arms moved in again as pushing proceeded [BC] before moving out again [CD] and taking up a constant deflection when pushing ceased. The final deflection was somewhat smaller than the initial one and an increase in pressure from 60 to 70 kPa occurred. This suggests either that the initial diameter of the cavity before pushing was not cylindrical i.e. larger at the arm location, or that, after the push, the cavity was not a right cylinder having a depression at mid-height.

In the French SBPM(PAF), a thin film of water is maintained between the body of the instrument and the membrane (Baguelin et al. 1978). The pressure is monitored during penetration and any increase is taken to indicate that the drilling procedure must be modified to prevent disturbance. Figure 6.17 indicates that, even for full-displacement installation in sands, substantial disturbance can take place with only a slight increase in pressure. A fairly rigid probe where the membrane bears directly on the instrument body and where there is no possibility of the lantern strips flattening out during expansion of the membrane would significantly reduce the possible inward and outward movement of the soil during penetration and would likely lead to higher lateral pressures. In a comparison of the PAF and the British
Figure 6.17  Pressure vs. Strain when Pushing with PM Slightly Expanded, SCPM.
SBPM (Camkometer) in Po River Sand, Bruzzi et al. (1986) found that the lateral pressures obtained with the PAF were much smaller than those obtained with the more rigid Camkometer. The Camkometer data tended to be slightly above the best estimate of the in situ stresses and the PAF lift-off was very close to the water pressure.

6.4.1.5 Implications for Measurement of In-situ Stress

The apparent sensitivity of the measured lateral stress in sands to soil strain during penetration and to equipment tolerances and flexibility suggests that great care must be exercised in the interpretation of the results from the various instruments presently used to determine in situ stress. Recent work on lateral stress (Schmertmann (1985), Marchetti (1985), Robertson (1986), Huntsman (1985), Jefferies et al. (1987)) has suggested that disturbance will occur and that the measured response of the soil to the insertion of the instrument can be used to infer the initial stress state, usually from empirical correlations. Schmertmann (1985) pointed out that in very few cases in nature are we likely to encounter soil, especially sands, laid down in such a manner as to result in a uniform lateral stress and that we should expect a large variation in the measured stress.

Robertson (1982) tried to relate the increase in lateral stress around the cone friction sleeve to the peak dilation angle of the sand. A similar concept relating the stresses measured on a lateral stress cell behind a cone to the State Parameter was suggested by Jefferies et al. (1987). Robertson (1986) showed that the value of $K_\sigma$ predicted from friction sleeve measurements is very sensitive to small changes in friction values. Figure 6.18 shows the chart suggested by Robertson (1986) to relate CPT friction to $K_\sigma$ which was derived from calibration chamber tests. For a particular value of $q_c/\sigma_\varphi$, the predicted value of $K_\sigma$ varies greatly with slight changes in friction, $f_c$. 

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Figure 6.18  Relationship Between Sleeve Friction and $K_o$ for Calibration Chamber Tests in Ticino and Hokksund Sands (adapted from Robertson, 1986).
Figure 6.19 shows the friction profiles measured with two different cones at McDonald Farm. The friction values from cone #1 are smaller than for cone #2 whereas the penetration resistance values from cone #1 are greater than for cone #2. In the light of the above discussion with respect to the sensitivity of the measured stress to probe diameter and the suggestion that stresses are changing extremely rapidly around the friction sleeve, it is likely that the variations shown in Figure 6.19 could be due to slight differences in the geometry of the cones and in the thickness of the friction sleeve. The current ASTM Standard D 3441 for cone testing stipulates that the friction sleeve should be the same diameter as the cone tip +0.5mm to -0.0mm. Such a tolerance represents a possible variation in diameter of 1.4% equivalent cavity strain which in stiff soils would lead to large stress changes. Lunne et al. (1986), in an evaluation of several cones, also suggested that the observed variability of CPT friction values was due to differences between the diameters of the cone and sleeve. This suggests that any relationship between lateral stress and sleeve friction or lateral stress cell measurements may be dependent on the individual cone used and could change as wear of the tip and sleeve occurs. Much tighter tolerances on the dimensions of the friction sleeve may be required. In addition to the problems of friction sleeve dimensions, the area immediately behind the cone is a region of rapid stress change. It would be preferable, therefore, for any lateral stress sensing element to be positioned some distance behind the cone tip.

The measurement system will also have an effect on the values obtained. All force or strain measurements require some movement of the measuring device in order to register a reading. This effect can be reduced by using a very rigid system. However, a system measuring hoop stress, for example, must contract slightly and will likely give a lower value of lateral stress than an instrument that is expanded against the soil. This is a similar concept to that of active earth pressure compared to passive pressure.
Figure 6.19 Comparison of Sleeve Friction Measurements, Two Cones, McDonald Farm.
6.4.1.6 Summary of Discussion on In-Situ Stress Measurement in Sands

The measurement of lateral stress is very sensitive to the method of measurement. Any instrument pushed into the soil will cause disturbance which will induce stress changes. Any attempt to infer in-situ stress will require quantification of the disturbance effect.

The zone immediately behind the cone tip is a region of very rapid stress change. Consequently, friction sleeve measurements are unreliable parameters upon which to base an assessment of in-situ stress. In addition, the measurement of lateral stress has been shown to be very sensitive to the tolerances in the dimensions of the measuring instrument. For instruments of comparable diameter to the cone penetrometer, the manufacturing tolerances stipulated in the current ASTM Standard for CPT testing are insufficiently stringent to result in repeatable values of lateral stress.

It is believed, therefore, that the only way in which in-situ stress can be estimated in sands is by measuring lateral stress with a number of different instruments. From an established hierarchy of methods, it may then be possible to interpolate the undisturbed stress. The variability of stresses measured by three very similar instruments in this study indicates the extreme difficulty of measuring in-situ stress in sands. The lateral stress cell being developed for incorporation in the SCPM will enable valuable research to be carried out and will allow direct comparison with PM lift-off pressures.
6.4.2 Friction Angle

6.4.2.1 Interpretation of Expansion Curves

The Hughes et al. (1977) method of analysis for the SBPM test shows that a plot of log effective pressure against log cavity strain should be a straight line with slope, $s$, where $s$ is a function of $\phi_v$ and the maximum dilation angle, $\nu_{\text{max}}$. Figures 6.20(a), (b), (c) and (d) show double logarithmic plots of pressure against strain for SBPM, FDPM, FFPM and SCPM tests, respectively, at about 5 metres depth at the McDonald Farm site. The slopes obtained are listed in Table 6.2.

As discussed earlier, Withers et al. (1986) postulated that the inability of Hughes and Robertson (1985) to derive reasonable values of friction angle from FDPM tests using conventional methods was due to the expansion of their instrument being limited to about 10%. They suggested that if expansion could continue to larger strains, the expansion curve would eventually coincide with the SBPM curve. Consequently, the log-log plot might be expected to tend towards a straight line at higher strains.

Figure 6.20(c) shows FFDPM test results in which expansion continued to greater than 30% strain. The plots show a continuous curvature and no straight line can be observed in the later stages of the tests. A straight line can be drawn through points from the first 10% of the test as was the case for the FDPM. The slopes obtained are higher than those from the SBPM. It is possible that the large creep strains observed in the stress-controlled tests obscure the frictional behaviour of the soil in the later stages of the test and so no straight line can be identified i.e. creep is the dominant behaviour.

It was hoped that the strain-controlled SCPM tests would reduce the effect of the creep strains by maintaining the deformation at a constant rate. Figure 6.20(d) shows that the curves still continue to bend
Figure 6.20 Hughes, Wroth and Windle Analysis of SBPM and FDPM Tests, McDonald Farm (adapted from Withers, Howie, Hughes and Robertson, 1989).
Figure 6.20 (cont.) Hughes, Wroth and Windle Analysis of SBPM and FDPM Tests, McDonald Farm.
Table 6.2  Slopes and Derived Friction Angles-Hughes, Wroth and Windle (1977) Analysis, McDonald Farm.

<table>
<thead>
<tr>
<th>SOUNDING (Instrument)</th>
<th>DEPTH (metres)</th>
<th>INSERTION METHOD</th>
<th>SLOPE</th>
<th>( \phi_{CYL} )</th>
<th>( \nu_{CYL} )</th>
<th>( \phi_{SPH} )</th>
<th>( \nu_{SPH} )</th>
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<td>SBPMT-1 (HPM)</td>
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<td>Bored</td>
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<td>34.3</td>
<td>-0.9</td>
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<td></td>
<td>5.3</td>
<td>Bored</td>
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<td>26.9</td>
<td>-9.5</td>
<td>15.0</td>
<td>-21.3</td>
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<tr>
<td>FDPMT-1 (HPM)</td>
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<td>12.6</td>
<td>26.1</td>
<td>-10.2</td>
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<tr>
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<td>4.6</td>
<td>Pushed</td>
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<td>39.1</td>
<td>5.1</td>
<td>22.6</td>
<td>-14.0</td>
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<td>FDPMT-2 (HPM)</td>
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<td>32.3</td>
<td>-3.3</td>
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<td>23.4</td>
<td>-13.2</td>
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<td>FFDPM-5</td>
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<td>N/A</td>
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<td>31.9</td>
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<td>15.1</td>
</tr>
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<td>N/A</td>
<td>55.7</td>
<td>28.7</td>
</tr>
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<td>Pushed</td>
<td>No linear portion</td>
<td>N/A</td>
<td>N/A</td>
<td>19.6</td>
<td>-17.1</td>
</tr>
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</table>
to the right when plotted in this fashion. The best straight lines continue to be given by the early portions of the curves where the effect of disturbance is likely to be greatest. The slopes obtained from the early part of the curves are much steeper than from the other pressuremeters.

The friction angles obtained from the slopes obtained above are also listed in Table 6.2. The values obtained using cylindrical cavity expansion theory from the SBPM tests are too low compared to the values of 36 to 40 degrees given by Hughes and Robertson (1985) for this deposit. The derived angle obtained from the loading curves appears sensitive to the assumption of $\phi_c$, and to slight variations in slope. When the Robertson and Hughes (1986) correction for loose sands is applied, however, more reasonable values are obtained for the SBPM tests.

The cylindrical model of expansion gives unacceptably high values of friction angle for the other instruments, especially for the SCPM. The Hughes, Wroth and Windle (1977) analysis can be extended to spherical cavity expansion as is shown in Withers, Howie, Hughes and Robertson (1989). When the spherical model is used, the FDPM and FFDPM give unacceptably low values of friction angle but the values obtained from the SCPM are still too high.

Another consideration must be addressed in the case of the SCPM and that is the effect of the compressibility of the protective steel strips on the expansion curves. Figure 6.21 shows the curve for the test on June 25, 1987 both corrected and uncorrected for the compressibility effect and the difference is considerable. The data obtained on this date are those for which the best estimate of compressibility effects is available. The data for tests at depths of 3.6, 5.1, 6.6, 8.1, and 9.7 metres have been corrected and have been analysed using the spherical expansion model. The friction angles obtained are listed in Table 6.3. The calculated friction angles are all unreasonably high.
UBC SEISMIC CONE PRESSUREMETER

Site: McDonald Farm  Depth: 5.1 m  Date: 25/6/87

Figure 6.21  Effect of Lantern Compressibility on Hughes et al. (1977) Analysis of Friction Angle, SCPM.
Table 6.3

Values of Friction Angle After Compliance Correction, SCPM 25/6/87, McDonald Farm.

<table>
<thead>
<tr>
<th>DEPTH (metres)</th>
<th>SLOPE</th>
<th>$\phi_{CYL}$</th>
<th>$\nu_{CYL}$</th>
<th>$\phi_{SPH}$</th>
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For the Full Displacement instruments, the values of friction angle are very variable with the SCPM data giving much higher slopes on the logarithmic plot. The reason for this is unclear. The FFDPM and FDPM give results of the same order despite the difference in L/D ratios and so L/D ratio is unlikely to be the cause. Rate effects are unlikely to be the reason as little creep was observed below a cavity strain of 10% over which most of the slopes were taken. It is most likely that the SCPM results are higher because of a disturbance effect due to the unloading around the expanding section of the pressuremeter.

6.4.2.2 Friction Angle from Unloading Curves

It is possible to use the analysis of Houlsby et al. (1986) to analyse the unloading portion of the pressuremeter curves. Figures 6.22(a), (b), (c), and (d) show typical plots of strain ratio against excess pressure ratio for tests at around 5 metres depth. All test data eventually form a straight line. The slopes obtained are listed in Table 6.4. The flat portion at the end of the curves in the case of the SCPM data is due to the difficulty of correcting for the membrane resistance when the membrane is sucked in very rapidly at the end of a test as discussed previously.

The values obtained from the unloading theory are too low, being in most cases less than a reasonable estimate of $\phi_{ev}$. Houlsby et al. (1986) also calculated very low values of friction angle when analysing SBPM data and found that the calculated value of friction angle is insensitive to the chosen value of $\phi_{ev}$. One problem they pointed out was that whereas there are many points on the loading curve, there are far fewer on the unloading curve due to the fact that the soil tends to arch around the cavity fairly early during the deflation phase. It seems unlikely, therefore, that the unloading portion will provide sufficient sensitivity for successful derivation of friction angle. Houlsby et al. (1986) suggested that a large strain theory would likely be necessary to allow analysis of the unloading portion.
Figure 6.22 Analysis of Unloading Curves, McDonald Farm (adapted from Withers, Howie, Hughes and Robertson, 1989).
Figure 6.22 (cont.) Analysis of Unloading Curves, McDonald Farm.
Table 6.4  Shear Strength Parameters from Unloading Curves.

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6.4.2.3 Effect of L/D Ratio

The issue of L/D ratio is likely to be important in the determination of friction angle once a suitable theory has been devised. A supposed advantage of the larger expansion capabilities of the FFDPM and the SCPM over the FDPM used by Hughes and Robertson (1985) is that the soil beyond the zone of disturbance will dominate the pressuremeter response. If the analysis used in section 6.2 is again considered, an appreciation of the possible significance of L/D ratio can be obtained.

For the previously-considered tests at about 5 metres depth, the in situ horizontal effective stress is likely to be about 25 kPa. For the FFDPM tests, the pressure at 20% cavity strain is about 450 kPa if the November 2 test is neglected. This represents a $p_c'/p_o'$ value of about 16 which, from Figure 6.7 (using cylindrical cavity expansion theory), indicates a plastic zone of about 18 times the current cavity radius or about 22 times the probe radius. The FFDPM has an expanding section only 20 radii long and so, as is illustrated in Figure 6.23, it is unlikely that cylindrical theory will be applicable. Spherical cavity expansion theory is likely to be more applicable. For shorter expanding sections, the use of a cylindrical model is even less tenable.

6.4.2.4 Summary

PM tests at McDonald Farm where the PM was pushed into the ground behind a solid cone tip gave friction angles which were unreasonably high when analyzed using the method of Hughes, Wroth and Windle (1977) derived from cylindrical cavity expansion theory. Extension of the analysis method to use spherical expansion theory resulted in unreasonably low values of friction angle for all instruments except the SCPM for which overpredictions were still obtained. Analysis of the unloading portions produced unrealistically low values of friction angle.
Figure 6.23  Schematic of Cavity and Resulting Plastic Zone at Cavity Strain = 20%.
It is concluded, therefore, that current theories do not allow a determination of friction angle from full-displacement pressuremeter test results. Further research is necessary to derive a large strain theory of cavity expansion. It is likely that while cylindrical theory may apply to the initial 10% of expansion, expanding sections with L/D ratios considerably greater than 10 will be required to enable the full expansion curve to 20% cavity strain and greater to be modelled as the expansion of a cylindrical cavity. When it is considered that the effects of tolerances in instrument dimensions discussed in Chapter 4 indicated the beneficial effects of an increase in probe diameter, the optimal size of probe to allow the use of interpretation methods based on the theory of cylindrical cavity expansion is likely to be prohibitively large from an operational viewpoint. Empirical methods of friction angle determination would seem to offer greater potential. The importance of strain rate effects on the current methods of analysis is still unclear.

6.4.3 Unload-Reload Modulus, $G_{ur}$

6.4.3.1 Test Results

Unload-reload loops were carried out in all tests. Table 6.5 shows the modulus data. Conventionally,

$$G_{ur} = \Delta p_c' (2\Delta \varepsilon_c).$$

For the results in the Table, the modulus was calculated using the more correct expression:

$$G_{ur} = \frac{\Delta p_c' (1 + \varepsilon_c)}{2\Delta \varepsilon_c} \quad (6.1)$$

The term, $(1 + \varepsilon_c)$, is usually ignored for strains less than 10% but, for instruments capable of a greater expansion, it becomes significant. The equivalent shear strain increments calculated using the expression:

$$\Delta \gamma_c = \Delta p_c' / G_{ur} \quad (6.2)$$
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<th>Gur</th>
<th>Gmax</th>
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Unload-Reload Modulus Data, McDonald Farm - THIS STUDY

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are also listed. All values of $G_{ur}$ in the sand at McDonald Farm are shown plotted against depth in Figure 6.24.

The FFDPM values shown have been corrected approximately for the hysteresis effect discussed in section 4.3. The correction factor was based on unload-reload cycles of 0.2% cavity strain or 0.4% shear strain. A range of strain increments occurred in the field testing and so the suitability of the correction for other sizes of unload-reload loop is unknown. Therefore, although some allowance has been made for inaccuracies due to hysteresis in the FFDPM data, it is likely that errors still exist. This will be the cause of some scatter in the data. The SCPM values have not been corrected for lantern compressibility as the correction cannot be defined with sufficient accuracy for unload-reload loops. In some cases, correction for lantern compressibility could result in an increase in modulus of as much as 100%. Consequently, the values of modulus obtained must be treated with some caution. Nevertheless, from Figure 6.24 it can be seen that the values obtained from both instruments are of a similar order of magnitude.

Figure 6.25 presents $G_{ur}$ values obtained by Hughes (1984) using the SBPM at McDonald Farm. The range of values obtained during the present study (Figure 6.24) is superimposed to illustrate that the SBPM moduli are of the same order of magnitude as those determined using the FFDPM and the SCPM and exhibit similar scatter. This supports the conclusion of Hughes and Robertson (1985) that the values of $G_{ur}$ appear to be insensitive to the disturbance caused by pushing the pressuremeter into the ground. Hughes and Robertson (1985) attributed the large scatter to soil variability.

Figure 6.26 shows the $G_{max}$ values obtained with the seismic cone compared to the profile predicted by the empirical equation:

$$G_{max} = 8 q_c$$

(6.3)
Figure 6.24 $G_u$ Measured in This Study vs. Depth, McDonald Farm.
Figure 6.25  $G_u$ From SBPM - McDonald Farm, Sept. - Oct. 1983.
Figure 6.26  Comparison of $G_u$, $G_{\text{max}}$ and $G_{\text{max}} = 8q_c$, McDonald Farm.
proposed by Robertson (1986). The relationship gives the correct general trend. The seismic cone gives a modulus averaged over a depth interval of commonly 1.0 metre whereas the profile developed from \( q_c \) indicates the likely variation of \( G_{\text{max}} \) over the full range of depth. Also on Figure 6.26 are the values of \( G_u \) measured with the FFDPM. The large variation in values of \( G_u \) is more readily understood when considered in relation to the variability in sand density as indicated by cone penetration resistance. The variability of the sand deposit was also illustrated in Figure 6.11.

To date, the ability of the FDPM to give comparable values of \( G_u \) to those obtained with the SBPM is the main attraction of the FDPM. However, the scatter exhibited by the data may mask any differences. In addition, the importance of stress and strain level in the interpretation of unload-reload moduli was discussed in Chapter 5 with reference to SBPM testing. In order to truly compare the SBPM and FDPM data, the effects of stress and strain level in FDPM testing must also be considered.

6.4.3.2 Importance of Stress and Strain Level

Because of the large range of expansion possible with the FFDPM and SCPM, many of the unload-reload cycles were carried out at higher stress levels than in the SBPM tests. It might be expected, therefore, that values obtained in FDPM tests should exceed the SBPM values.

It has been shown in Chapter 5 that normalizing \( G_u \) with respect to \( G_{\text{max}} \) and \( P_c \) to the in situ horizontal stress, \( p_0' \), could assist in the understanding of the measurements. Figures 6.27(a) and (b) show \( G_u/G_{\text{max}} \) plotted vs. \( p_0'/\sigma_v' \) for the FFDPM and SCPM results respectively. \( G_{\text{max}} \) was determined by seismic cone and \( \sigma_v' \) has been used rather than \( p_0' \) because of the uncertainty in estimating \( p_0' \). The data indicate a slight tendency for \( G_u/G_{\text{max}} \) from the FFDPM to increase with stress level but a clearer trend exists for
Figure 6.27  \( G_u/G_{\text{max}} \) versus \( p_c'/a' \), McDonald Farm.
the SCPM. The more consistent results obtained with the SCPM can be attributed to two main factors:

1. \( G_w \) and \( G_{\text{max}} \) were measured in the same sounding whereas \( G_{\text{max}} \) values used for the FFDPM results were obtained from a nearby seismic cone sounding;
2. the superior performance of the SCPM strain arms.

When the data are plotted in the form shown in Figure 6.27, it appears that the values obtained with the FFDPM are higher than those from the SCPM at a given stress level. However, from Table 6.5, it can be seen that the degree of unloading as reflected by \( \Delta \gamma \), was generally greater in the SCPM tests than in the FDPM tests. Hence, it appears that the FDPM unload-reload moduli are very similar to those measured using the SBPM. However, as the initial stress and strain conditions around the FDPM are very different from those around the SBPM, procedures for taking into account the effects of stress and strain level must be reconsidered.

### 6.4.3.2.1 Effect of Stress Level

In Chapter 5, the question of correcting \( G_w \) to a constant stress level was discussed. Bellotti et al. (1987) showed that for cycles of approximately equal strain increment size in chamber tests, the unload-reload modulus could be reduced to an almost constant value by correcting \( G_w \) using the average stress in the plastic zone in Janbu's formula, i.e.

\[
\frac{G_w}{G_{w,\text{avg}}} = \left[ \frac{p'_{\text{avg}}}{p'_0} \right]^{0.5}.
\]

The average stress was calculated assuming the soil to be linear elastic-frictional plastic. In the SBPMT, the average stress is always increasing as the plastic zone is pushed outwards. However, as was discussed in section 6.2.1, the stress conditions around the FDPM are likely to be very complicated. The possible
existence of a zone of locked-in stresses was also discussed. It is unclear how the modulus should vary as the membrane is expanded.

One possibility is that the zone of locked-in-stress may result in stiffer unload-reload moduli being measured during the early part of a FDPM test. At larger cavity strains, once the stresses have increased sufficiently to exceed those in the zone of locked-in stress, the moduli should reflect the greater influence of the sand beyond.

A test using the FFDPM was carried out in the sand at the McDonald Farm site in which seven unload-reload cycles were performed. The pressure-expansion curve is shown in Figure 6.28. Two similar tests were run using the same probe at the Leidschendam site. The modulus values are plotted against cavity strain in Figures 6.29(a) and (b). In the test at McDonald Farm, the moduli increased with cavity strain until a strain of about 6% after which they remained constant or decreased slightly. At Leidschendam, a peak modulus was reached very early in the test after which the modulus decreased until it appeared to remain constant beyond about 10% strain.

Similar tests were performed in the sand at McDonald Farm with the SCPM at 8.0 and 9.0 m depth. The results of three tests in which multiple cycles were conducted are shown plotted against cavity strain in Figure 6.30. All three show an increase in modulus with increasing strain and, hence, with stress level. A large increase in modulus with stress level is still observed. No high stiffnesses were recorded in the early portions of these tests.

The difference in behaviour between the two sites may be because the soil at Leidschendam is much denser than that at McDonald Farm. The $q_c/\sigma_v'$ value at the test depth at Leidschendam is about 265
Figure 6.28  FFDPM Test with Multiple-Unload Reload Cycles, Depth = 7.2 m, McDonald Farm.
Figure 6.29  \( G_w \) versus Initial Cavity Strain for Multi-Cycle Tests, FFDPM.
Figure 6.30  Variation of $G_m$ With Initial Cavity Strain, SCPM.
whereas, at McDonald Farm, it ranges from 40 to 93. However, if the data are examined in the light of the above discussion of stress and strain level, an alternative explanation can be advanced.

Figure 6.31(a) shows $G_{ur}$ plotted vs shear strain increment ($\Delta \gamma_s$) for the Leidschendam test at 4.9m. Shear strain increment was calculated using Equation 6.2. The modulus reduces with increasing strain increment. After correction for stress level using the method of Bellotti et al. (1987), the trend is still apparent. The lowest curve in Figure 6.31(a) shows $G_{ur}$ corrected using the simpler but more severe correction proposed by Robertson (1982), i.e.

$$G_{ur}' = G_{ur}/(0.5p'/\sigma_{ur}')^{0.5} \quad (6.4).$$

When $G_{ur}$ is corrected for stress level, the trend for $G_{ur}$ to reduce with increasing strain increment is much reduced. Figure 6.31(b) shows a similar plot for the McDonald Farm test at 7.2m. After application of the Robertson (1982) correction, there is little change in $G_{ur}$ with strain increment size. However, the range of shear strain increments is considerably smaller in the McDonald Farm case than in the Leidschendam test.

The degree of unloading can also be expressed using $\Delta p_{uc}/p_{uc}'$. Figure 6.32 shows the data plotted against this parameter. A similar trend for the Robertson (1982) correction to considerably reduce the effect of degree of unloading can be seen.

Bellotti et al. (1987) attempted to remove the effect of the degree of unloading by maintaining the shear strain increments at or about 0.2% strain. This was achieved by using a Strain Control Unit to control inflation. In the SCPM tests it was not feasible to control the strain increment because of the very small movements involved but the author attempted to keep the amount of unloading ($\Delta p_{uc}/p_{uc}'$) at about 0.3 to
Figure 6.31  Variation of $G_u$ with Strain Increment Size, FFDPM.
Figure 6.32 Variation of $G_w$ with Degree of Unloading, $\Delta p_c'/p_c'$. 
0.5. Some difficulty was encountered in achieving this due to system compliance. Nevertheless, the cycles were carried out and interpreted in a consistent manner. The SCPM tests in Figure 6.27(b), in which a fairly constant degree of unloading was maintained, show a steadily increasing modulus as the test progresses.

Figure 6.33 shows values of \( \frac{G_u}{G_{\max}} \) values obtained with the SCPM at McDonald Farm corrected for stress level using both the Bellotti et al. (1987) and Robertson (1982) methods versus normalized effective stress. The latter method results in a relatively constant modulus ratio of 0.3 to 0.4. Examination of the Seed and Idriss (1970) attenuation curve indicates that such a range of modulus ratio should correspond to shear strains between 0.15\% and 0.04\%. It appears, therefore, that if the Robertson (1982) correction is applied, a fairly constant value of modulus should be obtained even for a relatively wide range in degree of unloading. Figure 6.34 shows test data obtained at the Laing Bridge site. After the Robertson (1982) correction is applied, a reasonably consistent value of modulus ratio of between 0.3 and 0.5 is obtained. The scatter in measured values is considerably reduced by the correction.

The above results suggest that for FDPM tests in sands, unload-reload cycles carried out with \( \Delta p_c/p'_c \) of about 0.4 and corrected using the Robertson (1982) method will result in measured moduli which are about 30\% to 50\% of the small strain modulus, \( G_{\max} \).

6.4.3.2.2 Equivalent Strain Level

Figure 6.35 shows the values of modulus ratio obtained with the SCPM plotted on the Seed and Idriss (1970) attenuation curve using \( \Delta \gamma_\text{eq} = 0.15 \Delta \gamma_c \). The points for McDonald Farm conform reasonably well to the published envelope. For Laing Bridge, the values fall slightly above the envelope. While the above approach may not suit all sands, it appears from Figure 6.35 that a good starting point

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Figure 6.33 \( \frac{G_u}{G_{\text{max}}} \text{ Versus } p_c'/\sigma', \) McDonald Farm Site, SCPM.
Figure 6.34  \( G_{w}/G_{\text{max}} \) Versus \( p_{c}'/\sigma_{v}' \), Laing Bridge Site, SCPM.
Figure 6.35 \( \frac{G_m}{G_{\text{max}}} \) Versus 0.15\( \Delta \gamma_c \) Compared to Seed and Idriss (1970) Envelope.
for interpretation and comparison of moduli obtained using the SBPM and FDPM from various depths and different sites is to correct for stress level using equation 6.4 and to consider the corrected moduli in conjunction with the strain level over which each was measured. For the McDonald Farm and Laing Bridge sites using

\[ \Delta \gamma_{eq} = 0.15 \Delta \gamma_c \]

gave approximate agreement between the corrected moduli and the relationship of Seed and Idriss (1970).

6.4.3.3 Effect of Number of Cycles

Bellotti et al. (1987) also suggested that \( G_{ur} \) could vary with number of cycles depending on the relative density of the sand. A test was carried out with the SCPM in which a number of cycles was performed over the same stress increment. The results are shown in Figure 6.36. A summary of the measured moduli is given in Table 6.6. Thirteen cycles were conducted over an unloading stress increment of 350 kPa. Within the limitations of accuracy for deriving the modulus, the thirteenth cycle gave essentially the same modulus as the first indicating little effect on modulus of a small number of cycles. The effect of increasing the degree of unloading was also evaluated as shown on Figure 6.36. Ten cycles were conducted where the unloading stress increment was increased to about 550 kPa. The average modulus measured was 58 MPa compared to about 68 MPa for the 350 kPa unloading. Again, no significant change in modulus was observed over ten cycles.
Figure 6.36 Effect of Multiple Cycles on $G_m$, SCPM, McDonald Farm.
Table 6.6  SCPM Test at 9.7 m, McDonald Farm - Unload-Reload Moduli.

<table>
<thead>
<tr>
<th>CYCLE NO.</th>
<th>DEGREE OF UNLOADING (kPa)</th>
<th>Gur (MPa)</th>
<th>AVERAGE (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>350</td>
<td>67.3</td>
<td></td>
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<td>64.2</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>350</td>
<td>62.6</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>350</td>
<td>63.4</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>350 AVE=0.5</td>
<td>79.5</td>
<td>68</td>
</tr>
<tr>
<td>8</td>
<td>350</td>
<td>71.6</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>350</td>
<td>67.7</td>
<td></td>
</tr>
<tr>
<td>10</td>
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<td></td>
</tr>
<tr>
<td>12</td>
<td>350</td>
<td>68.6</td>
<td></td>
</tr>
<tr>
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<td>350</td>
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</tr>
<tr>
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<td></td>
</tr>
<tr>
<td>16</td>
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<tr>
<td>17</td>
<td>550</td>
<td>58.2</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>550 AVE=0.95</td>
<td>58.9</td>
<td>58</td>
</tr>
<tr>
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<td>550</td>
<td>57.5</td>
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<tr>
<td>23</td>
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</table>
Comparison of the data obtained in this study with previous FDPM and SBPM data indicates that the unload-reload moduli measured in sand are of similar order of magnitude and display similar scatter. Some of the scatter is due to variations in test procedure, some to inadequacies of the instruments and some to the natural variability of the soil. Consequently, careful calibration is required to eliminate errors due to instrument compliance and great care must be taken to ensure that moduli are measured using a consistent procedure over a consistent degree of unloading. A sufficient amount of unloading should be used to obtain the best possible definition of the unload-reload cycle consistent with the instrument and data acquisition system in use. Computer control of the test will greatly enhance the ability to obtain consistent moduli.

In order for comparisons to be made between tests using various instruments and procedures at different sites and test depths, consideration must be given to the effects of stress and strain level. In order to simplify such comparisons, it would be beneficial if unload-reload moduli were measured and interpreted using a consistent procedure.

For FDPM tests, unload-reload cycles measured over a degree of unloading, $\Delta p_{v}/p_{c}$, of approximately 0.4 would obtain the balance between unloading sufficiently to allow definition of the loop and not unloading far enough to cause stress reversal. To take account of the stage of the test and the test depth at which the unload-reload cycle was performed, it has been shown that a convenient and consistent method for correcting $G_{w}$ for stress level is that proposed by Robertson (1982), i.e.

$$G_{w}^{*} = G_{w}/(0.5p_{v}/\sigma_{v})^{0.5}$$  \hspace{1cm} (6.9).
After following the above procedures, no clear differences were observed between values of $G^*$ obtained from SBPM results compared to FDPM results. This confirms that, for the soils tested, the FDPM test gives unload-reload moduli in sands which are indistinguishable from those obtained from SBPM tests.

Arguments were presented in Chapter 5 which indicated that $G^*$ reflects the average stiffness of the soil around the FDPM. The soil elements are at a range of stress and strain levels. Consequently, $G^*$ is not a fundamental soil property. In order to relate $G^*$ to a more fundamental parameter such as the small strain shear modulus, $G_{ss}$, use can be made of the strain attenuation relationship suggested by Seed and Idriss (1970) or the set of curves presented in Figure 5.24. For the data obtained in this study, the modulus obtained was approximately 30 to 50% of the small strain shear modulus obtained from the downhole shear wave velocity. When the unload-reload moduli (corrected for stress level) were plotted against a strain level of $0.15 \Delta \gamma_c$, where $\Delta \gamma_c$ is the shear strain increment over which the cycle was conducted, reasonable agreement was obtained with the Seed and Idriss (1970) curve. This suggests that the FDPM could be used in a semi-empirical way to determine the variation of soil modulus with strain level as $G^*$ conforms to the correct general behaviour of soil modulus.

6.4.4 Rate Effects

It has been shown that both the stress-controlled and strain-controlled FDPM tests in sand display rate effects. In the case of the stress-controlled FFDPM test, the soil deforms at constant cavity pressure at a rate which reduces over time. In the strain-controlled SCPM tests, when the strain is held constant, the cavity pressure drops at a rate which reduces over time.

In Figure 6.37, the results of a test in sand at the Leidschendam site containing several phases of constant cavity pressure are presented. The inset plot shows the time-dependent strains for each stage. The
Figure 6.37  FFDPM Pressure-Expansion Curve With Multiple Creep Phases, Leidschendam Site.
amount of time-dependent deformation increased as the stress level increased. In the sixth holding phase, deformation was still occurring after 40 minutes. The pore pressure response on adjacent CPTU profiles at both the McDonald Farm and the Leidschendam sites indicated that the soil drained rapidly since no excess pore pressures were observed during penetration. The time-dependent deformations observed in the PM tests are, therefore, due to creep and not consolidation.

6.4.4.1 Creep in Sands

In the literature, references to creep of sands are rare. Deformation in sands is usually considered to occur immediately. Nonveiler (1963) presented a case history in which substantial time-dependent settlements of a grain silo bearing on rock fill overlying loose to medium dense sands were observed. The sand consisted of 66% quartz and 33% calcite. Nonveiler (1963) carried out long-term oedometer tests on disturbed samples of the sand and found that the compression was approximately linear with the logarithm of time. Schmertmann (1970) included a term to account for creep in his semi-empirical method for calculating settlement in sands which was based on similar observations of time-dependent settlements.

The creep behaviour of clays has been studied by various investigators and was shown by Mitchell et al. (1968) to conform to the general features of Rate Process Theory. Murayama et al. (1984) showed that the creep behaviour of Toyoura Sand in constant mean normal stress triaxial creep tests also fitted a rheological model based on Rate Process Theory. Zhikhovich (1985) carried out step-loading tests in a shear box and showed that the creep rate could be described by exponential laws. Zhikhovich (1985) also found that creep behaviour was greater in medium than in fine sand. No reason for this was given. However, it is likely that the increased intergranular stress due to the smaller number of grain contacts per unit volume of a medium sand compared to a fine sand increased the tendency for deformation at the contact points. Jackson et al. (1980) studied loading rate effects on the compressibility of sands in
uniaxial compression. They found that for loading times greater than 1 millisecond, rate effects had a relatively minor effect on stress-strain response. In a study of settlements in sand, Burland and Burbidge (1986) discussed case histories of time-dependent settlements in sand and found creep settlement to be approximately linear with the logarithm of time. It is clear, therefore, that sand displays time-dependent or creep behaviour.

6.4.4.2 Laboratory Study at UBC

An investigation of the time dependent behaviour of sand has been conducted in the laboratory at UBC (Mejia et al., 1988). Figure 6.38 shows the results of one of Mejia’s tests on a sample of Tailings Sand at 70% Relative Density ($D_r$) and a confining pressure of 200 kPa. The stress ratio was increased in increments, each lasting 20 minutes. At low stress ratios, the amount of creep was very small but, as the stress ratio increased, the amount of creep increased. Approximately 2% shear strain occurred in 20 minutes at a stress ratio of 5.

Mejia also showed that under one-dimensional compression, time-dependent behaviour was observed. Angular tailings sand was observed to possess a greater propensity for creep than rounded Ottawa sand. The difference was attributed by Mejia et al. (1988) to the angularity and to some extent to mineralogy. Under triaxial loading, both volumetric and shear creep strains were observed to increase with increasing principal stress ratio, angularity and confining stress.

Figure 6.39 shows a comparison of the behaviour of rounded Ottawa sand and angular tailings sand. The angular sand exhibited more pronounced creep than the rounded sand. The volumetric strain-shear strain relationships are also shown in Figure 6.39. At a low stress ratio, volumetric creep strains were contractive but at high ratios were dilative. At intermediate stress ratios, the creep strains were initially
Figure 6.38  Drained Triaxial Creep Test on Sand (Mejia, 1989).
Figure 6.39 Comparison of Creep Behaviour of Ottawa Sand and Tailings Sand (after Mejia et al. 1988).
dilative but became contractive as creep continued. Also noticeable in the upper part of Figure 6.39 is that, when shearing was restarted, the sample behaved very stiffly indicating that strain hardening had occurred during the creep phase. It appears, therefore, that the soil particles formed a more stable configuration during the creep phase although the mechanism is as yet unclear.

In summary, therefore, creep behaviour of sand has been observed in triaxial tests in the laboratory and appears to be of greater magnitude in angular sands than in rounded sands. Strain hardening appears to occur during the creep phase but the mechanism of creep is not yet clear. The observations from the laboratory study will now be used in consideration of the creep behaviour observed during FDPM field testing in sands.

6.4.4.3 Creep in FDPM Tests

When extending the above behaviour observed in the laboratory to the pressuremeter test results, the different boundary conditions must be considered. The laboratory test is a single element test where the stress and strain conditions are readily measured. In the PM test, the stresses and strains are known only at the wall of the cavity. The stress ratio varies from a peak at the wall of the cavity to the in situ value at some distance from the cavity. During a holding phase, therefore, the creep behaviour of the soil will vary with radius.

A typical FFDPM test with holding phases is shown in Figure 6.40. One of the most obvious differences between the triaxial test and the PM test is the amount of creep observed. The creep in the PM test is much greater than that observed in the triaxial test in Figure 6.38. A possible explanation for this can be found by considering the size of the plastic zone around the cavity. To illustrate this, assume that the soil conforms to the model of a linear elastic-frictional plastic material. In this model, it is assumed that
85/11/02 DEPTH=5.2 m. McDONALD FARM

Corrected for Membrane

Figure 6.40 Typical FFDPM Test With Creep Phases, McDonald Farm.
the soil remains elastic until failure occurs at the peak stress ratio. Assume that creep occurs only in the plastic zone. For soil with $\phi=40^\circ$, $p_o=50$ kPa and a cavity stress, $p_c=400$ kPa, the calculated radius of the plastic zone ($R_p$) is approximately $7.5a$, where $a$ is the radius of the cavity. Assume that the shear strain due to creep at $r=R_p$ is 1.0% which is equivalent to a circumferential strain of about 0.5% (i.e. $u/R_p=0.005$ or $u=0.0375a$). For the volume of soil in the plastic zone to remain constant, the size of the cavity would have to increase by 25%. When it is considered that volumetric strain is also occurring as was shown in Figure 6.39, then it is possible to explain the apparently high creep strains observed in the PM test. An additional contractive volumetric strain of 0.01% in the plastic zone would result in a cavity strain of 0.28%.

As previously mentioned, the full mechanism of the creep still has to be determined. Fines content may be important but creep has been observed in laboratory tests on clean Ottawa Sand, a predominantly quartz sand. The sand at Leidschendam has less than 1% fines and that at McDonald Farm less than 5%. However, Zhikhovich (1985) suggested that medium sands may exhibit more creep deformation than fine sands. The friability of the grains may also be important. These observations raise the possibility that the creep behaviour observed in large strain pressuremeter tests may be useful for identifying friable sands such as carbonate sands or for determination of fines content or gradation in a particular deposit.

The following sections present a proposed framework for understanding the effects of creep on expansion curves obtained from FDPM tests in sands.

6.4.4.4 Stress-Strain-Strain Rate Concept

Vaid and Campanella (1977) established a link between the stress-strain and strain rate behaviours during undrained loading of a marine clay. They showed that it was possible to predict the results of a constant stress creep test from the results of a constant strain rate test and vice versa. Convincing evidence of the
link was provided in the test illustrated in Figure 6.41. Three identical samples of clay were tested in undrained shear. Samples I and II were subjected to constant-rate-of-strain triaxial tests at two different rates (curves I and II). In the third test, the sample was initially sheared at the same strain rate as for sample I. At an axial strain level of about 0.8%, the rate of strain was suddenly increased to that of sample II. The stress-strain behaviour "curve-hopped" from I to II. Leroueil et al (1984) showed similar stress-strain-strain rate interdependence in one-dimensional consolidation testing.

The drained creep test behaviour of sand presented in Figures 6.38 and 6.39 can be interpreted from a strain rate perspective. At the start of a 20 minute creep phase, the strain rate is high but decays with time. As the stress ratio increases, the amount of creep occurring in 20 minutes is greater. To achieve identical ultimate strain rates in each creep phase, the duration of the creep phases must increase as the stress ratio increases. The creep test results can be considered to conform to the concept of a family of constant-rate-of-strain stress-strain curves as shown schematically in Figure 6.42. At the end of a creep phase, the addition of the next load increment causes the strain rate to increase and so the response is very stiff as the curve climbs to the relevant strain-rate contour. The longer the creep phase, the larger is the stress increment required to return to the original contour.

In order to investigate whether PM tests in sand could also be considered within the above proposed stress-strain-strain rate concept, a test was run in which the cavity pressure was increased at a steady but slow rate. Results are presented in Figure 6.43. After about 9% strain, the rate of strain was reduced and the PM curve appeared to follow a lower different curve (see Figure 6.43). When the pressure was again increased rapidly, the strain rate climbed to more than 20%/min and a higher curve was followed. This suggests that drained PM tests in sand are rate dependent so that the expansion curve obtained at one rate of strain will be different from one at a different rate. However, tests with an instrument capable of closer control of strain rates than was possible in this study are required to confirm this.
Figure 6.41  Influence of Step Change in Constant Rate of Strain on Undrained Stress-Strain Response (after Vaid and Companella, 1977).
Figure 6.42  Schematic of Strain Rate Contours in Triaxial Test.
Figure 6.43 Effect of Strain Rate on FFDPM Test, McDonald Farm.
Figure 6.44  Comparison of Creep and Relaxation in PM Tests in Sand.
Based on the preceding argument, it is postulated that a family of pressure-expansion curves exists and that the expansion curve obtained will depend on the rate of strain employed in the test. This concept can be used to explain the different behaviour exhibited by stress and strain controlled tests during holding phases as shown schematically in Figure 6.44. If two drained PM tests are performed in an identical sand at the same strain rate, they should follow identical pressure expansion curves. If at point A the stress is held constant, the strain will increase from $\epsilon_a$ to $\epsilon_b$ as the strain rate decays. Alternatively, if the strain is held constant, the pressure will decay along the line AC due to stress relaxation. Upon reloading, the strain-controlled test will quickly return to the original strain-rate curve. The stress-controlled test will require a large stress increment to join the original curve. This possibly explains the much steeper PM curves obtained with the SCPM. In these tests, expansion took place at approximately 8%/min and no constant stress holding phases were conducted. The implementation of the PD motor controller will allow further study of the validity of this concept as it will then be possible to carry out constant stress and constant strain holding phases in the same test. The preliminary results indicate that the stress-strain-strain rate concept is valid.

6.4.4.5 Rate Effects on Unload-Reload Cycles

In North America and Britain, there is currently a trend towards carrying out strain-controlled PM tests. Figure 6.45 presents the results of a strain-controlled SBPM test in Po River Sand from Bellotti et al. (1989). No stress holding phase occurred before commencing the unload-reload cycle. The rounded nature of the initial portion of the cycle due to creep is clear. At higher strains, this effect is likely to be of greater magnitude as is illustrated by the initial portion of the final unloading.

The potential effect of this creep behaviour on the interpretation of unload-reload modulus is illustrated in Figure 6.46 using the proposed stress-strain-strain rate concept. Unloading in a strain-controlled
Figure 6.45 Strain-Controlled SBPM Test, Po River Sand (after Bellotti et al. 1989).
Figure 6.46  Schematic Showing Effect of Creep on $G_w$.  

**CAVITY STRAIN**
Figure 6.47  Effect of Test Procedure on Cumulative Strain During Cyclic Tests.
manner during a relatively fast PM test without allowing complete stress relaxation will result in a more rounded unload-reload loop. The degree of rounding will depend on the rate of unloading. In an extreme case, if the degree of unloading was very small, an unrealistically large modulus could be inferred. However, if the stress is held constant until creep strains have essentially stopped, a better-defined unload-reload loop is obtained.

It is also possible that the cumulative strains observed during drained cyclic pressuremeter tests in sands could be affected by test procedures, as illustrated in Figure 6.47(a). Unloading below the current strain rate contour without allowing creep strains to stabilize will result in a rounded unloading curve. As the stress is increased again, the rate of creep will again increase and further strain will accumulate. The strain accumulation will depend on the rate of cycling. It will also depend on the stage of the test at which the cycling is conducted (i.e. the creep observed will depend on the volume of soil at a high stress ratio) with the strain potential becoming greater at larger cavity strains. In a strain-controlled test, the pressure drops off while the strain is held constant [see Figure 6.47(b)]. If an unload-reload loop is carried out in which the pressure is returned to the level at which the strain was first held constant, quite large strains are to be expected as the curve will have moved on to a contour of much higher strain rate. However, if the cavity pressure is held constant prior to the cycling, very small cumulative strains are likely to occur. The above concept may explain the large strains measured in the cycled test with the SCPM described in Section 6.4.3.3 and shown in Figure 6.36.

It was suggested by Hughes et al. (1980) that the cumulative strain measured during such multiple cycling in SBPM tests could be used as an indicator of liquefaction resistance. Robertson (1982) developed this idea to propose the relationship between cumulative strain in 10 cycles in a cyclic SBPM test and the cyclic stress resistance of sand shown in Figure 6.48. Robertson (1982) obtained his data from stress-controlled tests where the cumulative cavity strain over 10 cycles was measured after a 5 minute
Figure 6.48 Robertson's (1982) Proposed Relationship Between Cyclic Stress Ratio and Cumulative Strain at 10 Cycles.
holding phase to allow conditions to stabilize. Based on the above argument, it is likely that a different test procedure would have resulted in a different relationship between cumulative strain and liquefaction resistance.

The above discussion shows that, as in all other aspects of pressuremeter testing, great care must be taken in developing a test procedure that takes into account all the interrelated phenomena which influence the test results. It would appear that drained strain-controlled pressuremeter tests in clean sands, particularly where the PM is being expanded to cavity strains larger than 10% and in which $G_m$ is to be measured, should either be carried out at a low rate of expansion to reduce the potential effects of creep or should incorporate constant pressure holding phases to allow the dissipation of creep strains prior to the unload-reload cycle. Cyclic tests should also be conducted after creep strains have essentially ceased.

6.4.4.6 Rate Effects and Cone Penetration Testing

Tests conducted with the SCPM illustrate the behaviour of the radial stress around a cavity when the strain is held constant after rapid expansion. The pressure drops off at a rate decreasing with time. During cone penetration, the soil undergoes very large strains as the cavity is increased from zero radius to the cone radius in a very short time. The rate of strain for soil elements close to the cone is thus very high. Once the cone is inserted the cavity radius remains constant. The preceding strain rate argument would indicate that the cavity stress could reduce rapidly around the cone due to stress relaxation. It is possible that some of the unloading observed to occur as the cone tip passes a soil element is due to this phenomenon. This is in addition to any unloading due to a slight strain release as the soil passes the shoulder of the cone.
The introduction of another possible effect around the friction sleeve further emphasizes the need for caution in quantitative interpretation of friction measurements.

6.4.4.6 Summary of Rate Effects

Drained pressuremeter expansion tests in sand can be sensitive to the rate of expansion. A stress holding phase results in creep strains, the rate of which reduces with time. A strain holding phase results in a gradual decrease of pressure due to stress relaxation, the rate of which decreases with time. Such behaviour will affect the interpretation of the test but further study is required to determine by how much. Creep effects tend to be much smaller at cavity strains less than about 10% which explains why this phenomenon has not attracted much attention in the past and why present methods of interpretation of PM tests can neglect such effects.

The creep mechanism is unclear. The importance of fines content, soil mineralogy etc. has still to be determined. The possibility exists that FDPM testing could aid in the identification of soils comprising friable particles. However, a standardized test procedure would be required.

Unload-reload moduli may be in error if insufficient attention is paid to time-dependent deformations in the test procedure as discussed above. Again, standardization of the test procedure is essential to ensure comparability of moduli. The use of slow expansion rates or the incorporation of constant stress holding phases into the tests are two potential methods of minimizing rate effects. One possible test procedure is presented in the next section.
6.5 SUGGESTED TEST PROCEDURE FOR FDPM TESTS IN SAND

Chapter 4 investigated the importance of equipment characteristics on the quality of the results obtained. This chapter has identified the extreme sensitivity of the test results to equipment geometry and test procedures. It is therefore clear that in order for soil parameters from one PM to be comparable to those obtained with another, it is essential that both instruments and procedures be standardized. One possible procedure which incorporates the ability of the test to identify \( G' \) and creep characteristics is suggested below and is illustrated in Figure 6.49:

a) Penetration should stop when the centre of the membrane is at the desired test elevation and the thrust on the pushing rods should be removed;

b) Inflation should initially be pressure controlled until all three arms have begun to move. Thereafter, inflation should be strain controlled at a reasonably fast rate [say 10%/min];

c) At a cavity strain of about 5%, the pressure should be held constant and the creep rate should be allowed to drop to about 0.1%/min. Based on existing experience, this should generally take less than 5 minutes for reasonably clean quartz sand;

d) An unload-reload loop should be carried out. This can perhaps most easily be pressure controlled since the strain increments are small. The change in cavity pressure should be about 0.4\( p_c \). This will generally ensure that sufficient data is obtained to define the loop and that the radial stress will remain the major principal stress;

e) the membrane should then be expanded to 10% cavity strain and another holding phase should be conducted. Decay of the creep rate may take up to 10 minutes at this stage;

f) Another unload-reload loop should be performed;

g) The remainder of the test will depend on the limitations of the instrument in use. A further creep phase would be possible with the FFDPM but the SCPM would, in general, quickly reach the limit of its expansion;

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Figure 6.49 Proposed Test Procedure for FDPM Tests in Sands.
h) the membrane should then be allowed to deflate in order that the closing pressure may be determined.

Tests carried out in such a consistent manner will permit a better understanding of the variability of the soil by eliminating differences due to procedure.

6.6 SUMMARY OF CHAPTER 6 AND CONCLUSIONS

Comparison of SBPM, FFDPM and SCPM tests in the sand at McDonald Farm has shown the importance of instrument design and compliance, and of test procedure on the results obtained. In particular, the test results have indicated that the measured lift-off stress is extremely sensitive to instrument characteristics and tolerances.

The importance of eliminating or minimizing all sources of measurement imprecision due to instrument characteristics such as hysteresis in the strain measurement system, compressibility of the pressure system or dimensional tolerances cannot be over-emphasized. Precise calibration is imperative. This is especially true for the determination of unload-reload modulus in which the measured movements are typically as small as 0.04 mm for probes of 44 mm diameter. The importance of such factors increases as the diameter of the instrument reduces. In addition, repeatable lift-off pressures will not be measured unless there is stringent control over instrument dimensions. Given the apparent sensitivity of lateral stress to small strains, it is unlikely to be possible to measure identical lateral stresses in sands with different instruments.

Once instrument-related factors have been controlled, the effects of test procedure on the measurements obtained must be recognized. A previously unrecognized feature of drained PM testing in sands has been
identified in this investigation, namely that of rate effects. For the sand at McDonald Farm, creep behaviour became important to the pressure-expansion curve once cavity strains of greater than about 10% were attained. The creep was more readily apparent in stress-controlled tests than in strain-controlled tests where the stress relaxation at constant strain could have been interpreted as being due to compliance of the inflation system. It has been shown that the creep behaviour of sands appears to conform to a proposed stress-strain-strain rate relationship and that when FDPM expansion curves are examined from such a point of view, many details of the test curves and differences between tests can be more readily understood. The importance of a standardized test procedure is clear and, until the rate effects can be quantified more accurately, it is recommended that tests be conducted at either a very slow strain rate such that rate effects are unimportant or that constant cavity pressure holding phases are introduced prior to the performance of unload-reload cycles. The creep behaviour observed in the FDPM tests may provide a method of determining in situ the presence of sands containing friable grains, e.g. calcareous sands.

It has been shown that it is not possible to derive the friction angle of sand from full-displacement PM pressure-expansion curves obtained in this investigation analysed using interpretation methods which are currently available. It appears that, in order to use the portions of the curves obtained at cavity strains greater than 10%, it will be necessary to develop a large-strain theory of cavity expansion. At these larger strains, it is unlikely that the PM expansion can be assumed to model the expansion of a cylindrical cavity. It is also unlikely that the expansion can be modelled as the expansion of a spherical cavity. Indeed, for the instruments used in this investigation, it appears likely that the initial expansion will model that of a cylindrical cavity but that, as the test progresses, the importance of end effects will increase and the advance of the plastic zone may begin to resemble the expansion of a spherical cavity. It is unlikely, therefore, that a suitable theory will be found without some empiricism being introduced, possibly in the
form of correction factors, to adjust for departures from the idealised behaviour. Such correction factors will likely vary with instrument characteristics such as L/D ratio.

Within the levels of accuracy possible in this study, the determination of unload-reload modulus in sands appears to have been relatively unaffected by the disturbance due to instrument insertion. The moduli obtained appeared to vary with stress level and degree of unloading in a manner very similar to the moduli from SBPM tests. For values of $G_{ur}$ measured over a stress increment of approximately 0.4\(p_{c}'\), where $p_{c}'$ is the effective pressure at the start of the cycle, a reasonably consistent value of pressuremeter modulus can be obtained after correcting for stress level using the expression proposed by Robertson(1982), Equation 6.4. For the data presented, $G_{ur}$ was approximately equivalent to 30 to 50\% of the small strain shear modulus, $G_{max}$, obtained from measurement of shear wave velocity. For the sands tested, reasonable agreement was obtained between the Seed and Idriss (1970) strain attenuation relationship for shear modulus when $G_{ur}$ was plotted using a shear strain level of 0.15$\Delta\gamma_c$, where $\Delta\gamma_c$ is the strain increment over which the cycle was conducted.

The importance of test procedure to the results of FDPM test results cannot be over-emphasized. In order to allow comparison of test curves and soil parameters derived from them, it is proposed that FDPM tests in sands should follow a standard procedure. One possible test procedure designed to take account of all the factors outlined above has been suggested.
7.0 SUMMARY AND CONCLUSION

7.1 INTRODUCTION

The growth of interest in in-situ testing has resulted in the development of many new instruments. The instruments fall into two general categories:

(a) logging tools;

(b) specific test methods.

In recent years, instruments have been developed which endeavour to combine the logging and specific test capabilities. One such instrument is the cone pressuremeter which combines the stratigraphic logging capability of the CPTU with the Full-Displacement Pressuremeter. The FDPM is used to obtain a pressure-expansion curve for the soil after the instrument has been pushed into the ground. The work presented in this thesis examines the factors affecting the analysis and interpretation of FDPM tests.

It has been determined that the stress conditions around the FDPM and the shape of the subsequent expansion curve are very dependent upon the geometry and rigidity of the equipment with the effects being most apparent in stiff soils. The shape of the test curve is also greatly influenced by the method of inflation and the rate of expansion. The effects of the test procedure adopted are dependent upon the properties of the soil being tested and consequently the analysis and interpretation of soil parameters are dependent upon the test procedure. The previously unrecognized importance of rate effects in pressuremeter testing in sands has been shown.

The results of pressuremeter testing are commonly interpreted using methods based on the theory of cylindrical cavity expansion. A hypothesis applied to the FDPM is that if sufficiently large expansion can be achieved, the pressure-expansion curve will tend towards that obtained from a pressuremeter
installed with minimal disturbance, allowing interpretation of the results using existing theories. The validity of this concept has been examined and has been found wanting in the case of sands. A framework has been advanced upon which the interpretation of soil stiffness may be based which accounts for the effects of stress and strain level in a consistent manner thus allowing comparison of moduli from test to test and soil to soil. The major conclusions are outlined below. Although these have been developed mainly from FDPM testing, many also apply to SBPM tests.

7.2 EQUIPMENT

Details of equipment design and tolerances on manufacture have a large impact on the results obtained, particularly where very small displacements are important to the interpretation of soil parameters, e.g. lift-off pressure and unload-reload modulus. As cavity strains are calculated with reference to the instrument radius, the significance of such effects increases as the instrument decreases in size. The major conclusions concerning the equipment from this investigation are listed below.

(a) Instrument Geometry.

- The pressuremeter section should have a diameter identical to or slightly greater than the conical tip to ensure consistency of lift-off behaviour.
- No specific effect of location of the PM behind the tip was noted. Based upon other published work, it is likely that the PM should be located more than ten diameters behind the tip, i.e. remote from the severe unloading at the cone shoulder.
- The L/D ratio of the instrument should be standardized to permit comparison of test results.
(b) Protective Sheath.

- The protective sheath over the membrane should have the same radius of curvature as the instrument, i.e. the sheath should not be compressible.

- The intrusion of soil behind the lantern strips is undesirable. A sheath consisting of strips bonded to an outer membrane is preferred notwithstanding the resulting increase in membrane correction.

- Pore pressure measurements on the pressuremeter membrane are necessary to allow determination of effective stress in order that the magnitude of any compliance correction may be established.

(c) Strain Measurement System.

- There is no unique relationship between strains interpreted from volume measurement and those from displacement arms. The relationship varies with the stiffness of the soil.

- Cantilever strain arms appear to be preferable due to their simplicity. Detailed calibrations are required to ensure that limited and repeatable hysteresis can be allowed for in interpretation of very small movements and to detect any tendency towards interaction between the membrane, inflation medium and arm output. Any tendency towards drift of the electronics must be minimized. Where large deflections are to be measured, the spring load in the arms at maximum bending (i.e. prior to lift-off) may cause problems.
(d) Inflation Method.

- Hydraulic inflation systems impose rigorous requirements for saturation in order to limit compliance due to the compressibility of the system. This, in turn, requires fast data acquisition and inflation control systems.

- The use of a down-hole pressure developer eliminates many of the compliance problems inherent in surface-fed hydraulic systems. This speeds response of the system.

- The down-hole pressure developer will allow software-controlled inflation which will aid greatly in the study of rate effects.

- A disadvantage of the hydraulic system is that a leak cannot be tolerated unlike in pneumatic systems.

7.3 INSTALLATION METHOD

The extreme difficulty of installing a self-bored PM without creating disturbance and the additional difficulty of assessing the degree of disturbance make the use of full-displacement installation attractive. As was discussed in Chapter 5, results from self-boring pressuremeter tests in a calibration chamber suggest that, even under "ideal conditions", disturbance is likely. If the probe is pushed in, a consistent degree of disturbance should result, leading to easier distinction between equipment effects, disturbance and soil behaviour.
There is an apparent trend towards strain-controlled PM testing. It has been shown in this study that such testing can mask the existence of rate-dependent soil behaviour and stress-relaxation prior to unload-reload cycles in soils displaying creep can lead to difficulty in measuring modulus. During FDPM tests for this study, rate-dependent behaviour was particularly apparent at large cavity strains in stress-controlled tests in sands. Failure to take these effects into account in the interpretation of the test results can lead to erroneous conclusions.

Stress-controlled tests in soils displaying creep make methods of analysis involving curve-fitting difficult to apply due to the irregular nature of the expansion curves obtained. However, much extra information is obtained and better control of unload-reload cycles is possible if a constant stress holding phase is included in the test prior to unloading. Hence, it is concluded that some degree of stress control is required. A combination of strain-controlled inflation with constant stress holding phases prior to unload-reload cycles is believed to be the best approach. This approach is also likely to be well-suited to computer control.

A standard test procedure should be developed for sands. As stated above, the most suitable approach would be a combination of stress and strain control. The test should be stress-controlled until lift-off has been attained, and strain-controlled thereafter except prior to unload-reload cycles when creep deformations should be allowed to decay at constant cavity stress.

Standardized test procedures in sands (and in clays) will greatly assist research into pressuremeter testing as it is clear from the above that many effects hitherto attributed to soil behaviour may, in part, be due to variations in equipment and procedure.
7.5 METHODS OF INTERPRETATION

Pressuremeter test results are conventionally interpreted to give horizontal stress, stiffness and shear strength. Chapter 6 examined the applicability of existing methods of interpretation to FDPM results.

7.5.1 Horizontal Stress

The lift-off pressure in sands was shown to be extremely sensitive to instrument geometry. The values measured fitted the concept of increased lateral displacement leading to higher measured pressures which is embodied in the idea of the Iowa Stepped Blade instrument. With most instruments, the final value measured depends on the degree of unloading which occurs behind the tip of the instrument.

These observations suggest that, with a standard instrument and standard installation and relaxation procedures, it will be possible to derive empirical methods of interpretation to determine in situ stress in sands.

7.5.2 Stiffness

Current practice in PM testing is to derive shear modulus from unload-reload cycles carried out during a pause in inflation. This parameter has been observed to be insensitive to disturbance in SBPM testing. However, there is little consensus as to the relationship between the measured parameter and soil behaviour.

This investigation has shown that, provided the moduli obtained are interpreted with due regard to the effects of stress and strain level around the cavity, the values measured are in general agreement with those given by SBPM testing. Because of the variation of stress and strain level with radius, the
measured modulus reflects the average behaviour of the soil around the cavity. It is, therefore, necessary to apply some semi-empirical or empirical factor in order to relate the measured value to the in situ value.

In sands, the Robertson (1982) correction for stress level was found to result in an approximately constant modulus independent of the stage of the test at which it was measured. However, it was pointed out that this would not necessarily be true in all cases as the modulus is dependent on the test procedure. For SBPM tests from the literature and for the SBPM tests in this study, when the normalized corrected modulus was plotted versus about 15% of the shear strain increment over which it was measured, reasonable agreement was obtained with published data for attenuation of modulus with strain level. The corrected value of $G_{ur}$ was found to be between 30% to 50% of $G_{max}$. It is stressed that this does not mean that the measured stiffness is the in situ stiffness but that $G_{ur}$ is related to the soil stiffness and that $G_{ur}$ is an indicator of soil stiffness.

As the modulus obtained is very sensitive to the degree of unloading over which it is measured, standardisation of the test method is required before detailed studies can be made of this subject. If it is ascertained that the above relationships hold, it may prove necessary only to measure $G_{max}$ by seismic methods and use generally-accepted relationships to obtain the stiffness at the strain level of interest. Further research is required. Due to the variability of soil deposits, the profiling capability of a CPTU ahead of the PM and the determination of $G_{max}$ from down-hole shear wave velocity are important arguments in favour of the SCPM being used for this research.

7.5.3 Shear Strength

Existing methods of determining friction angle in sands from PM results were shown to be inapplicable to FDPM tests. The major drawback to the application of standard theories is that expansion to large
strains probably results in cylindrical cavity expansion being applicable to the early stages of expansion, with the remainder of the expansion curve reflecting a transition from cylindrical to spherical cavity expansion. Again, it appears that empirical adjustments will be required to the data obtained.

7.5.4 Rate-Dependent Behaviour of Sand

An apparent strain-rate dependence was observed during FDPM tests in sands to cavity strains of about 30%. The test data indicated that it would be possible to obtain a family of pressure-expansion curves by varying the rate of expansion. The instruments used in the study were neither sufficiently accurate nor sufficiently flexible to allow this to be confirmed. It is important to consider this strain-rate dependence when selecting test procedures. Several examples of potential errors due to rate effects were presented. The large amounts of creep observed for clean predominantly quartz sand suggests that the pressuremeter may provide a useful way of detecting granular material with friable grains.

7.6 CONCLUSION

This study has shown that the analysis and interpretation of the results of pressuremeter tests are greatly affected by many factors other than soil behaviour. The range of the relevant factors is so large that standardisation of equipment and testing procedures is urgently required. This is true of all PM tests regardless of method of insertion to the ground.

The findings of this research point to the likelihood that all methods of interpretation will require some empirical adjustment to the interpreted parameters to allow for the effects of departure from the assumptions of the theory applied. In SBPM testing, the degree of disturbance is difficult to quantify and
the time and expertise necessary to carry it out are unlikely to make such testing feasible for all but the most specialised applications.

Consequently, the FDPM and particularly the Seismic Cone Pressuremeter are tools worthy of further research and development. The additional information provided by the expansion curve will enhance our ability to predict the behaviour of the soil under load. At present, the most fruitful approach to the instrument would be to standardize the geometry and test procedures and concentrate upon the development of semi-empirical correlations with engineering parameters and soil behaviour.

7.7 SUGGESTIONS FOR FURTHER RESEARCH

7.7.1 Equipment

The equipment should be standardized. Given that some degree of empiricism is required in the interpretation of FDPM results, there may be merit to reducing the L/D ratio of the expanding portion. This would result in a shorter more manageable instrument. Although a fluid-filled membrane is attractive from an operational viewpoint, i.e. easy computer control and volume measurement, the consequences of leaks may require the use of gas inflation. In addition, the sensitivity of the lift-off pressure and other parameters to the distance of the pressuremeter unit behind the tip should be investigated before the final L/D is selected.

7.7.2. Procedures

Procedures should be standardized for PM testing as they have been for cone penetration testing. The repeatability of pressure-expansion curves and unload-reload moduli should be investigated using
procedures similar to that presented in Chapter 6. It will likely be necessary to have different standard procedures depending on the soil type and the purpose of the investigation.

7.7.3 Interpretation

Once the above have been carried out, improvements in methods of interpretation can be sought. Areas of study should include the study of the relationship between the individual parameters measured in both sands and clays.

The empirical approach to the use of in situ testing must use the fact that the soil will respond differently to different types of loading to extract information about the undisturbed state. The parameters measured using the SCPM (shear wave velocity, pressure-expansion curve including lift-off pressure, penetration resistance, pore pressure, friction, potentially lateral stress with a lateral stress cell) all depend on stress state and history, strain level, compressibility, strength, rate effects, etc. to varying degrees. It may, therefore, be possible to use combinations of the parameters measured, e.g. penetration resistance and pressure at 20% cavity strain, $G_m$ and $G_{max}$ with stress and strain level, to increase our ability to determine in situ properties of soil. For this type of study, it will be necessary to enforce very strict control on equipment tolerances and calibration and on procedures of installation and testing.

As $G_m$ should theoretically be a measure of properties in the horizontal plane and $G_{max}$ depends on the stresses in the vertical and horizontal plane, the relationship between the two parameters should be investigated as a possible source of information about anisotropy.

The strain-rate dependence of sands requires further investigation. An instrument allowing close control of rates of stress and strain change would be required. This study will require a pressuremeter capable
of expansion to strains in the order of 25% and may not be suited to an instrument with a very low L/D ratio. Creep rates may be another useful indicator of soil type.
LIST OF REFERENCES


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Hughes, J. M. O. (1986). Personal communication.


APPENDIX A. LABORATORY AND FIELD EVALUATIONS OF PRESSUREMETERS USED IN STUDY

A.1 THE HUGHES PRESSUREMETER

A.1.1 Calibration

The arms were calibrated by placing a micrometer against the head of the arm and recording the voltage output from the arm strain gauge. The micrometer was scribed in divisions of 0.01mm. The results were plotted and the slope of the line gave the arm calibration factor. A typical plot is shown in Figure A.1. Simulated unload-reload cycles were carried out to determine the nature of any hysteretic behaviour. The calibration curves were generally linear but there was some tendency towards electrical drift on the strain arms. Little hysteresis was observed on the unload-reload cycles.

The pressure transducer was calibrated against a reference pressure transducer which had in turn been calibrated against a dead-weight pressure tester. A linear calibration curve was obtained.

A.1.2 Observations on Installation

During the testing programme carried out at Lulu Island, relevant information about the drilling process was recorded. The information is presented in Table A.1. From examination of the "comments" column in Table A.1, it can be seen that much difficulty was encountered in the drilling process.

On the first and second field day, a jetting tool with four 0.25" diameter holes and a cutting shoe shaped as shown in Figure A.2 was used. The upper portion of the shoe had a diameter of 73 mm and the lower half was 75mm. No particular difficulty was encountered in penetrating the organic silty clay, but, in the sand, very high mud and pushing pressures were noted. Upon withdrawal of the instrument, the cutting shoe was found to be clogged apart from a narrow channel through which mud was likely escaping up the outside of the instrument causing disturbance of the sand.

For the second boring, a new jetting tool was used. The new tool had 6 jets, each 0.125" in diameter as shown. In addition, the cutting shoe was machined to 74mm O.D. on the lower half. The upper half had an O.D. of 73mm. The PM testing did not begin until the sand was reached. Penetration of the sand required a mud pressure of 690 kPa and a pushing head force of about half of the maximum to reach the first test depth. For the third test, the maximum pushing head force was required. Upon withdrawal of the probe, the jetting tool was found to be bent. Drilling to the next two test depths was straightforward, but on endeavouring to reach 27.1m depth, refusal was reached at 22.9m. The jetting tool was found to be cracked.

A third boring was attempted after repair of the jetting tool. Difficulty was encountered with blocking of the jets by gravel sucked from the base of the mud tank. No further testing was possible due to unavailability of the drill rig.

Table A.2 shows the drilling parameters recorded during the PM testing at McDonald Farm conducted by Hughes(1984). Comparison of the flow rates to those recorded at Lulu Island shows that considerably higher flow rates and higher mud pressures were used in the more recent work. There is no apparent reason for this. The penetration rates were similar. Some of the increase in mud pressure may have been due to smaller diameter holes in the jetting tool than in the previous work.
Figure A.1   Strain Arm Calibration Curve for Hughes Pressuremeter.
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*Distance Behind Cutting Shoe
Figure A.2 Jetting Tool and Cutting Shoe Configuration, Hughes SBPM (adapted from Campanella et al., 1990).
Table A.2  Hughes Pressuremeter Testing in Drilling Parameters - ADAPTED FROM HUGHES, 1984.

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* Remarks:...

Moyno Mud Pump.

Brahma Mud Pump
APPENDIX A.2 FUGRO FULL-DISPLACEMENT PRESSUREMETER (FFDPM)

A.2.1 Data Acquisition System and Test Control

As noted in Section 4.3.1.2, the instrument had been designed to work with the Fugro Data Acquisition System (DAS) which was not available to the author, some modifications to the connectors at the surface were required. A Connection Box was built to enable the analogue signals to be fed to a Compaq portable computer by ribbon cable and to Watanabe WX4403 XYY' Chart Recorders using BNC cables. The XYY' recorders allowed graphical display of the analogue data during the test. A typical trace of pressure and displacement for Arm 3 is shown in Figure A.3. Two displacement arms can be plotted against pressure on each XYY' recorder and so two recorders were required to monitor all three strain arms. This graphical output allowed close control of each test.

The analogue readings were also fed into a Compaq Portable computer fitted with a Data Translation Board #DT2801A. The board incorporates a 12 bit A/D converter, which was originally set to operate between -1.25V and +1.25V with a resolution of 0.6mV but was later modified to operate between 0 and 1.25V, improving the resolution to 0.3mV. The minimum time between data points was set at 1 second. The data were stored in RAM in integer format during a test and were copied to floppy disk when the test was completed. A hard copy, again in integer format, was output to the printer during the test.

The nitrogen gas supply for membrane inflation was provided by a bottle with a maximum pressure of 2000 psi (14 MPa). A coarse regulator on the bottle reduced the supply pressure to approximately 500 psi (3.5 MPa) to ensure that very high pressures were not applied to the probe in error. A more precise regulator was used to control the inflation rate. A 15 kPa to 800 kPa regulator was used for softer soils and a 35 to 3100 kPa regulator was used for tests in stiff soils. The system of valves illustrated in Figure A.4 was assembled to enable close control of unload-reload loops. The two-way valves between the probe and the regulator allowed the gas pressure to be shut off without adjustment of the regulator. The needle valves were then used to control the rate of pressure decrease or increase.

A.2.2 Laboratory Evaluation

A.2.2.1 Strain Arm Calibration

The arms were calibrated using the same procedure as was used for the Hughes SBPM. However, the arms on the FFDPM were subjected to a more rigorous examination. The arms were calibrated in 0.1 mm increments over the first 1.0 mm, i.e. where lift-off is to be measured, and thereafter at 0.2 mm intervals. This was done for the arm moving out and in. The data were plotted and a best-fit straight line was fitted. A typical plot is shown in Figure A.5.

Some difference was observed between readings at a particular deflection during expansion or contraction i.e. hysteresis was observed. This hysteresis is believed to have been partly due to friction between the arm and the spring. A graphite-based lubricant was applied to the area of contact in an attempt to reduce this effect. Further study of the hysteresis effect indicated that the degree of hysteresis varied depending on the deflection of the arm. This had serious implications for the accuracy of deflection measurements during unload-reload loops. A typical unload-reload loop with a range of deflection of 1 mm is shown in Figure A.6. It is clear that the calibration factor applicable to the conversion of arm output to deflection changes depending on the magnitude of the unload-reload loop. In Figure A.6, the applicable factor for an unloading of 0.2 mm would be 165 mV/mm as opposed to an overall arm factor of 105mV/mm. Calibrations were carried out for each arm where unload-reload loops of 0.04 mm were modelled at 1.0 mm intervals of arm deflection and equivalent values of calibration factor were
Figure A.3  Typical XYY' Recorder Trace, FFDPM.
Figure A.4  Inflation Control System for the Fugro FDPM.
Figure A.5  Strain Arm Calibration for the Fugro FDPM.
Figure A.6  Example of Simulated Unload-Reload Loop during Calibration-FFDPM, Arm 3.
determined. The values obtained are shown in Figure A.7. This allowed $G_{\text{eff}}$ values measured over loops of approximately 0.2% strain to be corrected in an approximate fashion for the hysteresis effect.

The above calibrations were carried out using a voltmeter to monitor the arm outputs. In the field, the data was digitized prior to storage. Unload-reload loops of 0.2% strain required accurate measurement of total movements of 0.044 mm. To adequately define the loop, a number of points is desirable and so a resolution of better than 0.01 mm is necessary. Such a deflection would give a voltage output of approximately 1 mV. The best resolution obtainable with the Compaq was 0.3 mV. In the absence of any electrical noise, therefore, the DAS was capable of the required resolution.

A further characteristic of the arms was an apparent drift with time at a constant displacement. Figure A.8 shows a case where the arm was allowed to move out to a deflection of 11 mm and then was maintained in that position for 20 minutes. The output of the transducer changed in such a way as to indicate an apparent inward movement. The change in output was about 0.03 V or an apparent movement of about 0.3 mm (1.3 % cavity strain). Similar overshooting was observed when the arm was returned to zero displacement. The reason for this behaviour was not detected but it is possible that the excitation voltage of 10 V on the gauges was too high resulting in excessive heating of the gauges.

A.2.2.2 Pressure Transducer Calibration

The pressure transducer in the pressuremeter was calibrated by inflating the expansion unit in a steel cylinder and comparing the transducer output to a reference transducer after equilibrium had been attained. The reference transducer had been previously calibrated against a Dead Weight Pressure Tester. The pressure tester was insensitive below 70 kPa (10 psi) but was accurate to 7 kPa 1 (psi) above that. The calibration constant for the transducer was approximately 8.8 kPa/mV or 11.4 mV/bar. The resolution of the DAS was 0.3 mV and so the minimum resolution of the pressure transducer was about 2.5 kPa. As the maximum capacity of the system was 10,000 kPa, this resolution was very good at 0.025 % of full scale. Unfortunately, the power supply of the Compaq was found to introduce noise to the data. This reduced the sensitivity of the probe in soft soils where typical total pressure increases of 150 kPa over the whole range of expansion were measured. The noise formed a large part of the signal as is shown in Figure A.9, the result of an expansion test in air. Even including the effect of electrical noise, the resolution was still very good as a change in pressure of 20 kPa represented 0.2 % of full scale. The effect was not so noticeable on the strain arms as a noise pulse 2.5 mV in magnitude only represented a cavity strain change of approximately 0.1 %.

A.2.2.3 Membrane Calibration

In all pressuremeter tests, the results must be corrected for the pressure required to inflate the membrane in the absence of soil. As noted earlier, it is desirable to keep the correction as small as possible to gain maximum accuracy in soft soils. Figure A.9 shows a typical pressure-expansion curve in air. The maximum pressure required to inflate the membrane to about 28% cavity strain was about 90 kPa. Figure A.10 shows the result of a Fugro FDPM test in clay at McDonald Farm. At a depth of 2.2 m, the pressure required to expand the probe to 25% strain was only about 500 kPa. The membrane correction therefore accounted for approximately 18% of the maximum pressure.

O'Neill (1985) encountered problems in determining the membrane correction because the correction was very sensitive to rate effects: the slower the rate of expansion, the lower the pressure required to inflate the membrane. The Adiprene membrane used on the Fugro probe was also susceptible to rate effects. As an in-depth study of this phenomenon was not deemed necessary at this stage of the probe's
Figure A.7  Variation of Unload-Reload Calibration Factor with Strain Arm Displacement, FFPDM.
Figure A.8  Drift of Strain Arm Output with Time at Constant Displacement = 11 mm, Fugro FDPM, Arm 3.

N.B. 0.03V represents 0.291mm (1.3% strain)
Figure A.9  Membrane Calibration Curve, FFDPM.
Figure A.10  Typical Test at Shallow Depth - FFDPM.
development, the membrane calibration was determined by inflating the membrane at a similar rate to that employed during the field tests.

A further problem encountered during inflation in air was a tendency for the membrane to inflate unevenly due to imperfections in the membrane material. A typical case is shown in Figure A.11 where arm 3 moved out much further than arm 2 at the same pressure. This occurred despite the effect of gravity - the probe was on its side with arm 2 downwards and arm 3 upwards. To minimize the effect of differences in individual arm movement, the average of the three arms was used for the membrane calibration.

A.2.2.4 Mechanical Design

The mechanical design of the probe was complex and, because the instrument was a prototype, a number of incompatibilities existed due to design changes during manufacture. A number of problems were encountered during assembly and disassembly which made field repair of the probe difficult. In particular, great difficulty was experienced in putting the membrane on to the instrument due to the membrane being approximately 3/4 of the probe outside diameter. In addition, limited clearance between the lantern and the membrane clamping rings caused problems whenever soil particles intruded between these two components.

Other comments on the mechanical design were passed to Fugro for consideration in design of future instruments.
Pressure (kPa).

Arm Displacement (mm).

Inflation in Air.
Probe horizontal, supported on blocks.
Arm 3 facing upwards.
Bulge close to Arm 3.

Figure A.11 Example of Uneven Inflation in Air, FFDPM.
APPENDIX A.3 UBC SEISMIC CONE PRESSUREMETER (SCPM)

A.3.1 SCPM Data Acquisition System

The data acquisition system consisted of a 8088 CPU, Data Translation Board DT2801A, two 360K Floppy Disk drives, 640K RAM and front panel analogue signal connectors. A high quality power supply was used in an attempt to minimize the noise introduced to the signals. In November 1987, one disk drive was replaced by a 720K 3.5" cartridge drive. The motor was controlled by an On-Off switch on the Connection Box.

A.3.2 Laboratory Evaluation

A.3.2.1 Arm Calibration

The calibration of the arms was carried out in the same manner as for the HPM and the Fugro probe. A typical calibration curve for Arm 2 is shown in Figure A.12. Unload-reload simulations were carried out throughout. No problems concerning hysteresis were observed.

A.3.2.2 Pressure Calibration

Pressure transducers were calibrated using a Dead-Weight Tester. The transducer used in the pressure developer, a Type 413-A with a range of (3500 kPa), was calibrated in a special calibration unit before placement in the probe. The calibration factor obtained was 433 kPa/V. As the least significant bit of the DAS was 0.005 V, a resolution of approximately 2 kPa was possible. The transducer used in the PM body was a Sensotech F/2893 transducer designed to work in the range (0 to 3500 kPa). This transducer is a thin disc which was placed in a slot in the body of the probe. The calibration was carried out with the transducer in place. A steel cylinder 44 mm in diameter was placed around the rubber membrane of the expanding section to prevent expansion. The Dead-Weight Tester was then connected to the pressure chamber. A pressure was applied and the system was allowed to come to equilibrium before the voltage output was read. The calibration factor obtained was 349 kPa/V leading to a maximum resolution of 1.75 kPa.

A.3.2.3 Probe Saturation

The use of a fluid for inflation of the pressuremeter unit introduces some additional problems in the assembly. To ensure that pressure changes are transmitted rapidly through the fluid, the system must be saturated. Initially, a complex system was devised to ensure such saturation. This involved assembly of the PM unit under oil followed by mating of the PM and PD in air. The connections required made mating under oil impossible. The mating process resulted in much air being trapped in the system. After some experimentation, the following procedure was devised. The PD was filled with oil, the PM was assembled and the PM and PD were connected together. The unit was attached to an oil reservoir and the PD slowly flushed oil back and forth through the PM until all air had been expelled. De-aired oil was used. This method seemed to result in an acceptable degree of saturation.

After the probe had been saturated the lantern strips were attached and a split-cylinder 44 mm in diameter was placed around the expansion unit to ensure that no excess volume of oil existed. The system was then sealed with a brass screw and O-ring. The pressure was monitored during the sealing and a small pressure increase (typically 5 kPa) was observed. When the split cylinder was removed, the pressure dropped due to the spring in the fully depressed arms causing the membrane to move out slightly thus increasing the volume of the system. The pressure measuring system was so sensitive that a change in
Figure A.12 Typical Strain Arm Calibration Curve, Arm 2 - SCPM (after Hers, 1989).
the pressure reading was also noted when the probe was moved from standing vertically to another position. All changes were small and so the initial reading on the transducer when the probe was in the upright position in air ready for penetration was taken to be the reference pressure.

A.3.2.4 Membrane Correction

In all pressuremeter testing, the pressure required to inflate the membrane in air must be subtracted from the test results to derive the true pressure-expansion behaviour of the soil. In the case of the SCPM, this proved useful as any error in the assumption that the transducer zero represented atmospheric pressure was cancelled when the membrane correction was applied. A typical expansion curve for inflation in air is shown in Figure A.13. The maximum pressure required to inflate the membrane was about 50 kPa.

A.3.2.5 Pressure Effects on Arms

Before the design of the arms was finalised, a mock-up of the arm configuration was made and studies were carried out to determine whether the arms were subject to pressure effects, i.e. whether the output of the strain arms changed due to the application of an all-round pressure. This phenomenon was believed to be a problem in early versions of the Cambridge SBPM (Ghionna et al., 1983). A pressure chamber was fabricated which allowed air pressure to be applied to the arm. The test set-up is illustrated in Figure A.14. A screw through the wall of the chamber allowed the deflection of the arm to be varied by a measured amount to simulate various stages of a pressuremeter test. Rubber and teflon tips were attached to the end of the screw to simulate the likely friction between the arm and the pressuremeter membrane. Brass rings were provided at each end of the chamber to clamp the O-rings used to seal the chamber. In tests before these rings were added, floating of the chamber on the O-rings led to apparent arm movements.

Work in this chamber led to improvements in the clamping mechanism, the choice of an optimum arm thickness and the determination of the most suitable location of the strain gauges on the arm. The typical variation in arm output during a pressure increase of 6900 kPa (1000 psi) was approximately 0.04 mV which corresponds to an arm movement of 0.027 mm. The screw controlling the arm deflection was found to move up approximately this amount when the chamber was pressurized. It was concluded that the arm behaviour was satisfactory.

A.3.2.6 Pressure Chamber Testing

In order to determine the ability of the arms to record accurate lift-off pressures, an oil-filled pressure chamber was constructed. It was designed to allow the PM to be surrounded by fluid which could then be subjected to a known pressure. The pressuremeter could then be inflated and the lift-off pressure determined. The testing in the chamber proved invaluable to the understanding of the behaviour of the probe.

Figure A.15 shows the effect of an increase in the pressure of the fluid surrounding the probe. As the chamber pressure increased, so too did the pressure in the pressuremeter until a threshold was reached beyond which the latter pressure remained relatively constant. The chamber tests were carried out when pressure was being measured in the PD only. The pressure in the PD ceased increasing when the external pressure was sufficient to press the membrane down until it sealed the outlet to the channel between the PD and PM. The increase in pressure in the PD did not correspond to that in the chamber indicating poor saturation of the probe. The chamber testing was carried out prior to the adoption of the improved saturation procedure. As the pressure in the PD is influenced by the external pressure, the pressure in the probe at the start of a test in the ground will depend on the total pressure applied to the membrane by the soil. It is therefore necessary to define a reference pressure to which to compare the measured
Figure A.13  Typical Membrane Correction Curve for SCPM (after Hers, 1989).
Figure A.14  Schematic of Pressure Chamber for Study of Pressure Effects on Strain Arms.
Figure A.15 Variation of Strain Arm Output and Pressure Developer with Fluid Chamber Pressure, SCPM.
pressure. For probes inflated using gas, the pressure chamber is vented to atmosphere after each test and so the pressure reading can be zeroed at the start of each test. With a fluid-filled closed system probe this is not possible. The pressure reading in the probe prior to insertion in the ground was taken to be atmospheric as explained in section A.3.2.3.

Figure A.15 illustrates another characteristic of the strain arms. As the external pressure was increased, the arm appeared to move in initially but then appeared to move out again as the pressure continued to increase. This was believed to be due to the arms bottoming out as the arm heads were pushed downwards by the interaction of the external fluid and the membrane. The resulting flexing of the arm led to an output from the strain gauges and thus an apparent outward displacement. Figure A.16 shows the effect of increasing the probe pressure after the external pressure had been set, in this case to 170 kPa. As the internal pressure increased leading to a reduction of the differential pressure, the arm appeared to move inwards due to the relaxation of the initial flexing. When the internal and external pressures balanced, lift-off occurred. The amount of apparent movement depended on the pressure differential across the membrane. This introduced the problem of defining zero deflection of the strain arms in the ground.

A further difficulty in the selection of a reference value for the arms was that when the membrane was fully collapsed, the arms were fully flexed. The single membrane used in the chamber tests had insufficient stiffness to hold the arm fully down and so there was a tendency for the arm head to protrude beyond the body of the instrument. Therefore, the zero reading obtained prior to inflation was dependent on the external pressure.

As explained above, it was not possible to refer all readings to the readings at the start of each test, as can be done in a pneumatic system. Instead, a set of independent values had to be established. A procedure for establishing such reference values was developed during the initial field testing. A further point which became apparent during the chamber testing was that, in a fluid filled system, a very small movement of the PD led to a rapid rise in pressure and so the data acquisition system had to be able to record at a high rate.

A.3.3 Field Evaluation

A.3.3.1 Initial Field Testing

The first day in the field with the UBC probe was January 14, 1987. At this stage, the pressuremeter unit was used behind a steel "dummy" cone identical in dimensions to the final fully instrumented cone (i.e. 44 mm in diameter). The first sounding was at McDonald Farm, the same site at which the Fugro probe was tested.

Figure A.17(a) is a plot of the uncorrected pressure-deflection relationship for all three arms for the test at 3.0 metres depth and Figure A.17(b) shows the uncorrected pressure vs average cavity strain. The most striking feature of this figure is the S-shaped nature of the loading curve, reminiscent of the pre-bored or Menard PM curves. Tests were carried out at 2.0, 3.0, 5.0, 8.0 and 10.5 metres depth and all curves exhibited the S-shape. In addition, examination of the early portions of the curves indicated first movement of the arms at pressures close to the hydrostatic water pressure followed by an initial steep portion. At point B in Figure A.17(b), the curve becomes flatter. The relative magnitude of this feature decreased with depth. Later measurements of the probe diameter with the lantern strips flattened out using a hose clamp showed the probe to be undersized by approximately 0.4 mm or 1.0% cavity strain. This led to the tentative conclusion that the S-shape was due to the unloading experienced by the soil after passage of the slightly larger (0.4 mm) diameter cone.
Pressure Transducer Output (V)

Chamber Pressure = 170 kPa (approx.)

Calibration Factors: Pressure - 433 kPa/V
Arms 0.9 mm/V

Apparent Inward Movement ~ 0.1 mm

Arm Output (Volts)

Figure A.16 Pressure Expansion Curve in Fluid Calibration Chamber, SCPM.
Figure A.17 Uncorrected Pressure Expansion Curve for Test at Shallow Depth SCPM.
Also noted during the testing was that the initial reading on the internal pressure transducer was successively lower at the start of each test. This could have been due to either temperature effects or due to a suction being set up by the PD returning to its reference position. The latter behaviour might have been expected if a leak was occurring only at high pressure but not during deflation with the result that the volume of oil in the reservoir was reducing slightly during each test. Indeed, in the test at 10.5 m, the PD reached its full capacity at an average membrane deflection of 3.0 mm, substantially less than the 5.0 mm normally possible. This suggested that a leak had been occurring.

The effect of cooling of the probe on transducer response was investigated by comparing the pressure at which the unloading portion of the curve became horizontal, i.e. the closing pressure, to the hydrostatic water pressure. Wroth (1982) stated that these two pressures should be identical in sand as the soil arches around the probe during deflation. Examination of the SCPM data indicated that if the pressure reading prior to the first inflation in air was taken as reference, sensible closing pressures were obtained. The evidence was, therefore, that the change in pressure zero reading was primarily due to the effect of a slight leak. Despite intensive effort, the source of the leak was not detected.

A further difficulty indicated during the first day in the field was in the choice of zeroes for the measurement of strain arm deflections. As noted earlier, at low external pressures with only one membrane over the probe, the spring load in the strain arms resulted in them standing proud of the probe diameter. As the probe went deeper, the arms were pushed progressively inward by the increased external pressures. However, at 10.5 m, when the motor was switched on and the pressure in the membrane increased, the arms first appeared to move inward and then outward as the lift-off pressure was reached as had been observed in the chamber testing. The choice of zeroes is important because the initial stage of the membrane correction curve is very steep. As the correction is applied as a function of strain, the reference values for calculation of strain become critical. In weak soils where the correction is a substantial portion of the total pressure, a slight change in the zeros used can affect the initial portion of the curve by a considerable amount as is shown in Figure A.18 which shows the test at 3.0 m after correction for membrane resistance. This would have serious implications for any derivation of modulus from the initial portion of the PM curve. In stiff soils, such effects are less important.

Another feature of the tests was that the pressure dropped when the PD was switched off prior to an unload-reload loop. This is in contrast to the continuing strain observed at constant pressure in tests with the Fugro instrument. When the motor stopped, the strain remained constant as the volume of oil could not change. In the absence of additional oil to maintain the pressure, the pressure dropped. Other possible reasons for a drop in pressure are equipment-related. For example, air in the oil due to incomplete saturation may have gone into solution causing a slight decrease in volume which would not necessarily result in a noticeable strain arm displacement. Alternatively, slight play or "backlash" in the PD when the motor was switched off would have had the same effect. The rate of pressure drop was allowed to slow before the unload-reload loop was carried out. Attempts were made to determine the reason for the pressure drop by inflating the pressuremeter inside a steel cylinder. However, due to the compressibility of the system combined with the extreme sensitivity to small volume changes, the results were inconclusive.

For the next field day, two membranes were put on the PM section in an attempt to bring the probe up to 44 mm in diameter. Prior to insertion into the ground, the probe was allowed to cool in air in an attempt to minimise any temperature effects. One test was carried out at 8.0m in the sand in order to assess whether the increase in diameter due to the second membrane had eliminated the S-shaped curve. Figure A.19 illustrates that it had not.
Figure A.18  Corrected Pressure-Expansion Curve, Depth = 3 m, McDonald Farm, SCPM.
Figure A.19  Pressure Expansion Curve, SCPM with 2 membranes, Depth = 8 m, McDonald Farm.
Tests were also carried out in the silty clay which underlies the sand at McDonald Farm. A typical curve is shown in Figure A.20(a) for the test at 17.0 m. The effect of the high porepressures set up during cone penetration (typically about 450 kPa at this depth) on the lift-off behaviour of the arms can be seen. The initial portion of the curve is shown to a larger scale in Figure A.20(b). All three strain arms appeared to move in and then out as the differential pressure across the membrane reversed. The requirement for very fast data acquisition is also shown. The points shown were taken at the rate of three per second. The shape of the curve is very different from that in sand with no S-shape apparent. Based on previous FDPM tests in clay the curve is very close to that expected for clays apart from the lift-off behaviour.

A.3.3.2 Assessment of Compliance

In an effort to understand the behaviour at lift-off, the probe (including the lantern) was inflated inside a 44 mm diameter steel split cylinder. The expansion curve is shown in Figure A.21. At the start of inflation, there was little resistance to the movement of the arms but, after an apparent average cavity strain of approximately 1.8% (the exact deflection is subject to the usual uncertainty as to the choice of strain arm zero), the resistance began to increase. The curve for an inflation in air is also shown. The two curves diverge at about 0.5% strain but the major divergence occurs at about 1.6% strain. Above a pressure of about 500 kPa, the relationship between pressure and deflection was effectively linear with a very steep slope. The observed compliance is believed due to a combination of the following effects:

a) flattening out of the lantern strips which had an initial radius of curvature greater than that of the probe;
b) spreading of the rubber membranes (two membranes were on the probe during this testing) as the head of the strain arm indented them.

The implication of the observed behaviour is that as the probe is pushed into the ground, the amount of initial compliance observed during the subsequent pressuremeter test will depend on the initial effective stress in the soil surrounding the PM element i.e. if the effective stress against the lantern is less than about 60 kPa, very little compression of the lantern would take place. Thus the observed initial portion of the expansion curve would be flat even if the soil was relatively stiff due to the flattening of the lantern strips. This was the behaviour observed in the test at 3.0 m depth shown in Figure A.18. However, if the prevailing effective stress was 300 kPa or greater, the initial portion of the expansion curve would be likely to be much steeper since the steel lantern strips would already be flattened. This is shown to have been the case for the test carried out at 8.0 m depth shown in Figure A.19. It is likely, therefore, that a portion of the S-shape observed in the SCPM tests was due to "compliance" of the lantern. Any attempt to correct the test curves for this effect required an accurate knowledge of the initial effective stress. This would be possible in free-draining soils, but in clays no correction is possible unless the pore pressure around the membrane can be measured.

No S-shaped pressure expansion curves were observed in the tests in soft clay for two possible reasons:

a) the increase in effective stress against the membrane was not great during the early portion of a PM test;
b) the clay was very much softer than the sand and probably much closer in stiffness to that of the instrument.
Figure A.20  Pressure-Expansion Curve in Silty Clay, Depth = 17 m, McDonald Farm.
Figure A.21  Inflation of SCPM in a 44 mm Steel Cylinder.
Point (a) is based on behaviour observed during the expansion of a research DMT in soft clay as shown in Figure A.22. Two points are illustrated in this figure:

- the total stress against the membrane is dominated by the pore pressure;
- the effective stress remains very small and effectively constant during inflation of the membrane.

An idea of the initial effective stress acting on the membrane in the test shown in Figure A.20 can be obtained by considering the pore pressures measured during cone penetration. At a depth of 17 metres, the pore pressure recorded behind the tip during penetration was typically about 450 kPa. The estimated total vertical stress at this depth is 325 kPa. The value of pore pressure relevant to the PM test depends on the effect of distance of the PM element behind the tip and of the time between stopping pushing and starting inflation. Nevertheless, the effective pressure could be small. Similar results were obtained by Azzouz and Morrison (1988) by monitoring the variation of total stress and pore pressure after a piezo-lateral stress cell was pushed into Boston Blue clay. Figure A.23 shows their measurements of total stress and pore pressure over 30 hours after installation. Immediately after insertion, pore pressure made up almost 90% of the total stress. As the pore pressure dissipated, the total stress also reduced. However, the ratio of horizontal effective stress to vertical effective stress reduced at first to around 0.05 before increasing greatly until it reached a value of about 0.6 which was within the range of $K_0$ at that site.

As a consequence of the above, the assessment of the effective stress at the location of the membrane in order to make corrections for compliance will depend on the pore pressure induced and on the elapsed time between the end of penetration and the start of expansion and makes it very desirable that the pore pressure at the membrane be measured.

A.3.3.3 Correction for System Flexibility

In order to deal with the effects of compliance, it is necessary either to eliminate the causes or to quantify the effects and correct the pressure-expansion curves. The compressibility of the oil can be reduced by effective de-airing. Elimination of the effects of lantern compressibility would require redesign. This would be the preferable solution. Alternatively, correction of the pressuremeter curves for compliance effects requires the following:

a) an accurate assessment of the prevailing lateral effective stress;

b) the determination of the relationship between pressure and deflection due to compliance.

As no pore pressure measurement was available, only results in sands could be corrected. In an effort to be consistent, a split cylinder of 44mm internal diameter was constructed. This was used to establish reference readings for the strain arms before commencement of penetration. The procedure developed was first to inflate the probe in air and then in the split cylinder. The initial pressure from the inflation in air, measured after the probe had cooled in the ambient air temperature, was taken to be the reference pressure and the initial readings on the strain arms observed in the split cylinder were used in the reduction of all other tests on that particular field day. A typical curve for inflation in the split cylinder is shown in Figure A.21. The average deflection of the three strain arms is presented. The curve has been corrected for membrane resistance. An equation was fitted through the data and was then used to correct the field data.
Figure A.22  Research DMT Pressure-Expansion Curve Showing Effective Stresses (adapted from Campanella and Robertson, 1985).
a) Variation of Total and Water Pressures with Time after Installation of Piezo-Lateral Stress Cell

b) Variation of Effective Stress Ratio with Time

Figure A.23 Variation of Stresses around Piezo-Lateral Stress Cell After Installation in Boston Blue Clay (adapted from Azzouz and Morrison, 1988).
The procedure adopted was as follows:

a) the pressure at lift-off was estimated for each arm from a plot such as Figure 4.14 showing total pressure vs deflection. The initial use of digital data presented problems in that lift-off commonly occurred between scans of the transducer outputs by the DAS. For example, Arm 3 moved at a pressure of between 90 and 128 kPa during the 1.6 seconds the data was being written to disk. The data acquisition software was subsequently altered to store data in RAM with the complete file being written to disk at the conclusion of the test. In assessing the lift-off pressure for Arm 3, the average of the pressure before and after lift-off was used;

b) the lift-off pressure was taken to be the average of the lift-off pressures measured for each of the three arms;

c) if the membrane inflation in air showed an initial resistance to expansion, the pressure from (b) was corrected for this;

d) the effective lift-off pressure was calculated using the assumed water pressure (only in sands);

e) the compliance of the membrane and the stainless steel lantern already taken up was then assessed from the curve obtained from inflation in the split cylinder;

f) the additional compliance due to the further increase in effective pressure could then be calculated;

g) the pressure-deflection curve could then be corrected for membrane resistance and for compliance.

Figure 4.14 shows the final curve for a test at 8.1m at McDonald Farm. Despite the corrections, some S-shape is still apparent. It has still to be determined whether this was due to inaccuracies in the measurement of and correction for compliance, to the unloading caused by the slightly undersized PM section or to the membrane correction.

The compliance effect is also relevant to the measurement of unload-reload modulus. For the test shown in Figure 4.14, a typical measured modulus before correction for compliance was 67 MPa for unloading from approximately 1400 kPa to 1100 kPa. This is equivalent to a strain increment of 0.22%. From Figure A.21, system compliance for this unloading accounts for 0.07% strain or approximately 30% of the measured movement. The corrected modulus is therefore 90 MPa. This illustrates the critical importance of accurate calibration and assessment of any compliance effects in any measurement of soil stiffness.

A.3.3.4 Improvements to the Strain Arms

In a further effort to improve the behaviour of the arms at lift-off, steps were taken to eliminate the "bottoming-out" due to external fluid pressure. An adjusted head configuration ensured that the arm bottomed out in a consistent manner. Figure 4.15 shows a comparison of the behaviour of two arm designs in tests in clay. Very sharp lift-off pressures were discernible after alteration of the arm indicating that the membrane-arm head interaction had been largely eliminated.