MEASUREMENT OF IN SITU LATERAL STRESS DURING FULL-DISPLACEMENT PENETRATION TESTS

by

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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

September 1991

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The thesis considers the problem of the in situ measurement of horizontal stress with specific reference to the use of full displacement probes. It is generally accepted that correct measurement of the horizontal stress in situ should be performed under conditions of no disturbance since, depending on the soil characteristics, even small amounts of disturbance can significantly alter the in situ conditions. The self-boring pressuremeter is widely acknowledged to be the best available instrument for measuring horizontal stress but recent research has shown the data obtained to be very sensitive to the effects of probe installation. On the other hand full displacement probes cause repeatable degrees of disturbance and the induced stresses and pore pressures may provide a means of backfiguring the initial pre-penetration in situ stress condition.

The thesis presents the results of a detailed programme of in situ testing using both self-boring and full-displacement probes during which measurements of both stress and pore pressure have been performed. In addition these measurements have been performed at various locations on the full-displacement probes to evaluate the stress distribution. Both plate-like and cylindrical probes have been used in the study.

Reference profiles of lateral stress have been established for each of the research sites based on both in situ and laboratory test results. The stresses measured by the full-displacement probes and the interpreted in situ conditions are compared to the reference profiles. The data suggest that in soft to stiff clay and sands reliable estimates of the reference lateral stress profile can be obtained from the large strain measurements using semi-empirical techniques which are based on the results of published case histories.

Certain index parameters are also shown to provide consistent indicators of the variation in $K_0$ stress state as obtained from the reference tests. Using both calibration chamber and field data both the cone resistance and pore pressure gradient around the tip are shown to be dependent on the in situ horizontal stress.

Theoretical approaches for evaluating the stresses around full-displacement probes are considered and cavity expansion formulations are applied to measurements in both sand and clay. The data clearly show the inadequacy of the theoretical approaches to consider the unloading that occurs around the penetrometer tip. The degree of unloading is shown to be a function of both soil and probe characteristics.
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LIST OF SYMBOLS

1D One dimensional
3D Three dimensional
a, b Regression coefficients
A Dilatometer first reading
\( A_{LS} \) Lateral stress amplification factor
\( \Delta A, \Delta B \) Dilatometer correction values
B Dilatometer second reading
BAT Piezometer probe
\( B_q \) Pore pressure parameter
\( B_T \) Temperature coefficient
\( C_A \) Shear wave velocity constant in anisotropic stress plane
\( C_\alpha \) Rate of secondary compression
CEM Cavity expansion method
\( C_c \) Compression index
CC Calibration chamber, cylindrical cavity
\( c' \) Drained (effective stress) shear strength parameter
cc Cylindrical cavity (subscript)
C Dilatometer third reading
CCW Counterclockwise
\( c_h \) Coefficient of horizontal consolidation
\( C_I \) Shear wave velocity constant in isotropic stress plane
CPM(T) Cone pressuremeter (test)
CPT(U) Cone penetration test (with pore pressure measurement)
cv Constant volume
\( c_v \) Coefficient of vertical consolidation
CW Clockwise
\( C_0, C_1, C_2 \) Regression constants
D Diameter of cylindrical penetrometer
DAS Data acquisition system
DH Downhole
DH-XH Downhole-crosshole
DMT Dilatometer (test)
\( D_r \) Relative density (%)
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<tr>
<td>$e_o$</td>
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<td>$E_c$</td>
<td>Pressure cell stiffness</td>
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<td>$E$</td>
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<td>FD</td>
<td>Full displacement</td>
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<td>$f_s$</td>
<td>Sleeve friction on pile or penetrometer</td>
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<td>FVT</td>
<td>Field vane test</td>
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<td>$G$</td>
<td>Shear modulus</td>
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<td>$G_{o, max}$</td>
<td>Maximum (small strain) shear modulus</td>
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<tr>
<td>$G_{VH}$</td>
<td>Small strain shear modulus of anisotropic plane</td>
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<td>$H$</td>
<td>Horizontal</td>
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<td>HFT</td>
<td>Hydraulic fracture test</td>
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<td>Horizontal stress cone</td>
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<td>$I_D$</td>
<td>Material index from DMT</td>
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<td>$I_r$</td>
<td>Rigidity index</td>
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<tr>
<td>$I_{rr}$</td>
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<td>$k$</td>
<td>Coefficient of permeability</td>
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<td>$K_A$</td>
<td>Active pressure coefficient</td>
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<td>Coefficient of lateral stress</td>
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<td>$K_{FVT}$</td>
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<td>$K_{1},K_{2}$</td>
<td>$K$ determined from oedometer tests on oriented samples</td>
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<tr>
<td>L</td>
<td>Distance behind penetrometer tip</td>
</tr>
<tr>
<td>LBS</td>
<td>Laing Bridge South</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid limit (%)</td>
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<tr>
<td>LS</td>
<td>Lateral stress</td>
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<tr>
<td>LSC</td>
<td>Lateral stress cone</td>
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<tr>
<td>LSSCP</td>
<td>Lateral stress sensing cone penetrometer</td>
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<td>LS-CPTU</td>
<td>Lateral stress piezocone test</td>
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<td>LS-FS</td>
<td>Lateral stress sleeve friction</td>
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<td>m</td>
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<td>N</td>
<td>1D constrained modulus</td>
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<tr>
<td>$M_r$</td>
<td>Rebound factor</td>
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<td>MDF</td>
<td>McDonald Farm</td>
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<td>na, nb, nt</td>
<td>Exponents for $V_s-o'$ relationships</td>
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<td>NC</td>
<td>Normally consolidated</td>
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<tr>
<td>$N_h$</td>
<td>Cone resistance stress factor</td>
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<tr>
<td>$P_e$</td>
<td>Pressuremeter empirical parameter</td>
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<tr>
<td>$N_{\Delta u}$</td>
<td>Excess pore pressure factor</td>
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<td>OC</td>
<td>Overconsolidated</td>
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<td>OCR</td>
<td>Overconsolidation ratio</td>
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<td>OCR$_{\text{max}}$</td>
<td>Maximum past OCR at greatest unloading.</td>
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<td>OD$_{OED}$</td>
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<td>p($p'$)</td>
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<td>$P_0,P_1,P_2$</td>
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<td>$P_0$</td>
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<td>($P_0$)$_{av}$</td>
<td>Average lift-off pressure</td>
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<td>($P_0$)$_{OM}$</td>
<td>Lift-off pressure from PBPM</td>
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<td>$P_a$</td>
<td>Atmospheric pressure</td>
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<td>PBPM(T)</td>
<td>Prebored pressuremeter (test)</td>
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<td>$P_{cc}$</td>
<td>Cylindrical cavity expansion limit pressure</td>
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<td>$P_f(p'_f)$</td>
<td>Final mean stress (effective)</td>
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\( P_{hy} \)  Horizontal yield pressure from PMT
\( p_i(p') \)  Initial mean stress (effective)
\( PI \)  Plasticity index
\( PIPM(T) \)  Push-in pressuremeter (test)
\( P_L, P_L \)  Limit pressure
\( PL \)  Plastic limit
\( PM(T) \)  Pressuremeter (test)
\( PPD \)  Pore pressure difference parameter
\( PPSV \)  Pore pressure stress dependent parameter
\( P_{sc} \)  Spherical cavity expansion limit pressure
\( q_c \)  Cone bearing resistance
\( q_D \)  Dilatometer bearing resistance
\( r \)  Radius of cavity or probe, coefficient of correlation
\( r_o \)  Initial cavity radius
\( R_p \)  Radius of plastic zone
\( RDMT \)  Research dilatometer
\( R1 \)  Receiver 1 of XH SCPT
\( R2 \)  Receiver 2 of XH SCPT
\( SBLC \)  Self-boring load cell
\( SBPM \)  Self-boring pressuremeter
\( SBT \)  Stepped blade test
\( SCPT \)  Seismic cone penetration test
\( SPT \)  Standard penetration test
\( STR \)  Strong Pit
\( SVC \)  Source vane cone
\( S_t \)  Sensitivity
\( S_u \)  Undrained shear strength
\( S_{up} \)  Peak undrained shear strength
\( S_{ur} \)  Residual undrained shear strength
\( t \)  Blade thickness, time
\( txl \)  Triaxial
\( t_p \)  Time for end of primary consolidation
LIST OF SYMBOLS (Cont'd)

\[ T \quad \text{Temperature, DMT thrust, vane torque} \]
\[ T_{\text{TBT}} \quad \text{Tapered blade test} \]
\[ T_I \quad \text{In situ equilibrium temperature} \]
\[ T_R \quad \text{Reference temperature} \]
\[ TSC \quad \text{Total stress cell} \]
\[ u \quad \text{Pore pressure} \]
\[ u_o \quad \text{Equilibrium pore pressure} \]
\[ u_c \quad \text{Critical pore pressure} \]
\[ u_f \quad \text{Crack opening pressure} \]
\[ u_i \quad \text{Initial pore pressure during penetration} \]
\[ u_t \quad \text{Pore pressure at time } t \text{ during dissipation} \]
\[ u_{1,2,3} \quad \text{Penetration pore pressures at different locations around a probe} \]
\[ u_{\text{upper}} \quad \text{Upper pore pressure on UCB LS cone} \]
\[ u_{\text{lower}} \quad \text{Lower pore pressure on UCB LS cone} \]
\[ U(t) \quad \text{Degree of pore pressure dissipation at time } t \]
\[ UBC \quad \text{University of British Columbia} \]
\[ UCB \quad \text{University of California at Berkeley} \]
\[ V \quad \text{Vertical} \]
\[ V_{fs} \quad \text{Relative sleeve friction voltage} \]
\[ (V_{LS})_C \quad \text{Corrected relative lateral stress voltage} \]
\[ (V_{LS})_M \quad \text{Measured relative lateral stress voltage} \]
\[ V_s \quad \text{Shear wave velocity} \]
\[ (V_s)_A \quad \text{Shear wave velocity in anisotropic plane} \]
\[ (V_s)_I \quad \text{Shear wave velocity in isotropic plane} \]
\[ w \quad \text{Moisture content} \]
\[ w_N \quad \text{Natural water content} \]
\[ W_T \quad \text{Depth to water table} \]
\[ XH \quad \text{Crosshole} \]
\[ Z_m \quad \text{Gauge zero reading} \]
\[ \alpha \quad \text{Regression coefficient, reduction factor} \]
\[ \alpha_A \quad \text{Anisotropy factor} \]
\[ \alpha_D \quad \text{Disturbance factor} \]
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<tr>
<td>( \alpha_i )</td>
<td>Initial stress measurement at ( t=1 )</td>
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<tr>
<td>( \beta )</td>
<td>Power decay constant</td>
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<td>( \gamma_t )</td>
<td>Total (saturated) unit weight</td>
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<tr>
<td>( \gamma_d )</td>
<td>Dry unit weight</td>
</tr>
<tr>
<td>( \gamma_w )</td>
<td>Unit weight of water</td>
</tr>
<tr>
<td>( \delta )</td>
<td>Soil-steel interface angle of friction</td>
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<td>( \Delta )</td>
<td>Horizontal stress factor</td>
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<td>( \Delta u )</td>
<td>Pore pressure increment, excess pore pressure</td>
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<td>( \Delta \sigma )</td>
<td>Stress increment</td>
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<td>Horizontal strain</td>
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<td>( \varepsilon_v )</td>
<td>Vertical strain</td>
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<td>( \nu )</td>
<td>Poisson's ratio</td>
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<td>( \sigma_h(\sigma'_v) )</td>
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<td>( \sigma_v(\sigma'_v) )</td>
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<td>( \sigma'_{hm} )</td>
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<td>Octahedral stress</td>
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<td>Temperature corrected net total blade pressure</td>
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<td>( \sigma'_{vm} )</td>
<td>Maximum past vertical consolidation pressure</td>
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<td>( \sigma_1, \sigma_2, \sigma_3 )</td>
<td>Total principal stresses</td>
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<td>Shear stress</td>
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<td>( \tau_{vh} )</td>
<td>Shear stress on penetrometer surface (( f_s ))</td>
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<td>( \mu_c )</td>
<td>Microstrain</td>
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<td>( \phi' )</td>
<td>Shear strength parameter</td>
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<td>( \phi'_{cv} )</td>
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<td>( \phi'_{txl} )</td>
<td>Drained triaxial angle of friction</td>
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<td>( \phi'_{ps} )</td>
<td>Drained plane strain angle of friction</td>
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ACKNOWLEDGEMENTS

The author is grateful to Dr. R.G. Campanella for his encouragement and assistance throughout the period of this research. Discussions and suggestions from Drs. Byrne, Fannin, Finn and Vaid are gratefully acknowledged. The assistance of Dr. J.M.O. Hughes to improve the self-boring pressuremeter installation technique and Dr. A.B. Huang with the self-boring load cell is much appreciated. Dr. T. Masood assisted with the lateral stress cone testing at McDonald Farm as part of a UBC/University of California at Berkeley cooperative testing programme. The technical support of Scott Jackson, Art Brookes, Harald Schrempp, Jim Greig and Glenn Jolly has been invaluable in performing the research tests reported herein. Discussions with colleagues in the geotechnical group and assistance with field work are gratefully acknowledged, in particular Don Gillespie, John Howie, Ian Hers, Ross Hitchman, Ilmar Weemees and Dan Zavoral. The author would also like to extend his thanks to Dr. P. Robertson for critically reviewing Chapters 1 and 2. Thanks also to Kelly Lamb for typing the text and to Richard Brun for producing some of the figures. Financial support was initially provided by the Science and Engineering Research Council (UK) in the form of a NATO Overseas Fellowship. Subsequent funding from the University of British Columbia Graduate Fellowship Programme and INTEVEP, S.A. is gratefully acknowledged.

A special thanks to my wife Carmen for her support and understanding and to my sons Paul, Peter and Thomas for being such great guys.
CHAPTER 1

1. INTRODUCTION

1.1 Background

In situ testing has recently undergone rapid advances both in terms of the variety of techniques and methods of interpretation. In the early 1970's self-boring pressuremeters (SBPM) were introduced to the geotechnical community followed by cone penetrometers (CPT) capable of measuring pore pressures (Torstensson, 1975; Baguelin et al., 1972; Hughes, 1973; Wissa et al., 1975) and the flat expandable dilatometer (DMT, Marchetti, 1975). The traditional objectives of soil profiling for site characterization became more easily attainable as experience with the near continuous logging cone penetration test (CPT) and DMT methods was acquired. Soil behaviour type charts were produced based on samples obtained from contiguous borehole investigations which, when applied at other sites, allowed considerable stratigraphical detail to be inferred indirectly (Jones and Rust, 1982; Senneset and Janbu, 1984; Robertson et al., 1985; Olsen and Farr, 1986) without recourse to more traditional procedures (i.e., sample recovery and inspection).

With the trend towards increased in situ evaluation of soil parameters, interpretation techniques were developed and used to obtain estimates of soil behaviour for engineering design (Robertson and Campanella, 1984). Theoretical and empirical procedures were presented for deriving strength and stiffness parameters (Durgunoglu and Mitchell, 1975; Schmertmann, 1976; Sanglerat, 1979; Robertson and Campanella, 1983; Baldi et al., 1986). For in situ tests where specific parameters were being sought, theoretical interpretation techniques gave consistent (but not necessarily correct) parameter values. Empirical procedures, sometimes loosely based on theory, were
adopted to obtain similar parameters from the logging methods; the results generally being very erratic.

As the field of in situ testing developed, it became apparent that a consistent form of analysis of in situ test results could only be achieved if the following data were available:

(a) identification of soil types in profile,
(b) details of the in situ stress state of the soil,
(c) information pertaining to the stress history of the soil deposit,
(d) deformation characteristics (stress-strain response), and
(e) flow and consolidation characteristics.

Topics (a), (c) and (e) have received considerable attention and various techniques exist for evaluating the required information (Torstensson, 1972; Baligh and Levasoux, 1980; Jamiolkowski et al., 1985; Teh, 1987; Campanella and Robertson, 1988). The advances have been made possible through the presentation of case histories whereby in situ test results have been calibrated against behaviour measured in the laboratory (including calibration chambers) or in the field.

Topics (b) and (d) are presently the subject of intensive research and have many similar associated problems. The direct measurement of in situ horizontal stress and modulus is sensitive to disturbance, whether stress or strain related. Errors in measurement can be crucial for both parameters. Furthermore, while specific test methods exist for determining in situ stress and modulus, considerable discussion exists as to how to best measure each one and the validity of what is actually being measured. Equipment characteristics and method of insertion are extremely important and may so disrupt the initial in ground conditions that direct measurement is not feasible.
For evaluation of the in situ stress state, in particular the horizontal stress, it is generally accepted amongst experimentalists that in-place measurement is preferred wherever possible. The in situ state of stress is considered in the following section.

1.2 In Situ Stress State

The three components that define the in situ stress state at any depth within a soil deposit for level ground conditions are the vertical and horizontal stresses and the equilibrium pore pressure. If hydrostatic conditions exist, the equilibrium pore pressure, $u_0$, can be calculated if the depth to the water table is known; otherwise in situ piezometric measurements are necessary. The total vertical stress, $\sigma_v$, is usually computed from vertical equilibrium of the unit soil column taking into account the variation of soil density throughout the profile. No computational procedure exists to evaluate the total horizontal stress, $\sigma_h$, and if measurements are not available, correlations (usually based on laboratory test data) are employed. For a normally consolidated (NC) or overconsolidated (OC) soil the relationship between the effective vertical and horizontal stresses is given by:

$$K = \frac{\sigma_h - u_0}{\sigma_v - u_0} = \frac{\sigma'_h}{\sigma'_v}$$

(1.1)

where $K$ is termed the coefficient of lateral stress (Donath, 1891; Terzaghi, 1925). For the level ground situation where $\sigma'_v$ and $\sigma'_h$ are principal effective stresses, and where consolidation occurs under conditions of no lateral strain, the ratio of the stresses is termed the coefficient of lateral stress at rest and is denoted $K_o$. Where the ground surface is not horizontal, the principal stress directions are rotated. In this case $\sigma'_v \neq \sigma'_1$ and $\sigma'_h \neq \sigma'_3$ (for a
NC soil) and the lateral stress coefficient, by definition, is not \( K_0 \) but \( K \). To determine \( K_0 \) it would be necessary to evaluate \( \sigma \) at different directions so that the principal stresses and their direction could be calculated. In most cases in engineering, \( K \) is the required parameter and measurement of the horizontal stress is the objective. Inadvertently this is often designated \( K_0 \).

At the UBC research sites where tests have been performed, the ground surfaces are essentially horizontal and so the stress ratio obtained refers to \( K_0 \). Where a slight surface gradient exists, \( K \) has been assumed to represent \( K_0 \).

A unique stress-ratio relationship exists for a particular homogeneous isotropic soil under conditions of virgin or first time loading. The one-dimensional (1D) normally consolidated value of \( K_0 \) can be estimated based on the Jaky (1944) expression (derived from laboratory tests):

\[
(K_0)_{NC} = (1 + \frac{2}{3} \sin \phi') \left[ \frac{1 - \sin \phi'}{1 + \sin \phi'} \right]
\]  

where \( \phi' \) is the drained friction angle of the soil determined in the triaxial shear apparatus. In more general use, Eq. (1.2) is simplified to:

\[
(K_0)_{NC} = 1 - \sin \phi'
\]  

The stress path AB in Fig. 1.1 corresponds to the above situation. Upon 1D unloading of an ideal elastic soil the stress path should follow BA and the 1D overconsolidated \( K_0 \) value would be the same as for virgin loading. In reality \( K_0 \) increases as the overconsolidation ratio (OCR) increases and the soil follows stress path BCD when unloaded. The overconsolidation ratio,
OCR, is defined as the ratio of the maximum effective past pressure experienced by the soil ($\sigma'^{v}_{vm}$) to the present vertical effective stress ($\sigma'^{v}$), i.e.

$$OCR = \frac{\sigma'^{v}_{vm}}{\sigma'^{v}} \quad (1.4)$$

In Fig. 1.1, the effective stress at point B corresponds to $\sigma'^{v}_{vm}$ while that at any point along the unloading or reloading curves corresponds to $\sigma'^{v}$. At C, $(K^o)^{OC} = 1$ which usually corresponds to an OCR of about 4. Further unloading to D promotes passive failure with both OCR and $K^o$ increasing.

![Diagram](image)

**Fig. 1.1** Typical effective stress paths for 1D consolidated soil (modified after Wroth, 1975).

During reloading along path DEFB the $(K^o)^{OC}$ value decreases becoming equal to $(K^o)^{NC}$ somewhere between F and B. As pointed out by Wroth (1975), at points C and E the soil has the same OCR but different $(K^o)^{OC}$ values. $K^o$ is thus stress path dependent.
Where information regarding the in situ horizontal stresses is not available, estimates of \((K_o)_{NC}\) can be obtained from published data relating \(K_o\) and OCR (Brooker and Ireland, 1965; Mayne and Kulhawy, 1982). Several attempts at relating \((K_o)_{OC}\) and OCR have been suggested on the basis of laboratory data, the most popular being that suggested by Schmidt (1966):

\[
(K_o)_{OC} = (K_o)_{NC} (OCR)^{\alpha}
\]  

(1.5)

where \(\alpha\) is essentially constant for a particular soil and varies from 0.42 for low plasticity clays to 0.32 for high plasticity clays (Wroth and Houlsby, 1985). Mayne and Kulhawy (1982) reviewed published data and confirmed the validity of Eq. (1.5) for uncemented clays during 1D primary unloading. Schmidt (1966) suggested that:

\[
\alpha = \sin \phi'
\]  

(1.6)

which was also confirmed by Mayne and Kulhawy (1982) using a much larger data base.

The coefficient of passive earth pressure, \(K_p\), is an upper limit on \((K_o)_{OC}\) where:

\[
K_p = \frac{1+\sin \phi'}{1-\sin \phi'}
\]  

(1.7)

Similarly, the coefficient of active earth pressure, \(K_A\), where:

\[
K_A = \frac{1-\sin \phi'}{1+\sin \phi'}
\]  

(1.8)

is a lower limit for \((K_o)_{NC}\). For a purely frictional loose to medium-dense soil, the variation of \(K_A\), \((K_o)_{NC}\) and \(K_p\) as a function of \(\phi'\) are shown in Fig. 1.2.
Mayne and Kulhawy (1982) evaluated soil response during 1D reloading and suggested that:

\[
(K_o)^R_{OC} = (K_o)^{NC}_{OC} \left( \frac{OCR}{OCR_{max}} \right)^{1-a} + M_r \left[ 1 - \left( \frac{OCR}{OCR_{max}} \right) \right]
\]

(1.9)

where \((K_o)^R_{OC}\) is the 1D overconsolidated lateral stress coefficient during the first cycle of reloading, \(OCR_{max}\) is the maximum value of overconsolidation ratio experienced by the soil (point D in Fig. 1.1) and:

\[
M_r = \frac{3}{4} (1 - \sin \phi') = \frac{3}{4} (K_o)^{NC}_{OC}
\]

(1.10)

At any point during the reloading phase, \(\sigma'_v\) increases so that while \(\sigma'_{vm}\) remains constant the OCR is decreasing until a value of unity is again attained when the reloading reaches point B. The application of Eq. (1.9) is difficult in a field situation since generally \(OCR_{max}\) is not known.
The above relationships have been derived from laboratory test data on both undisturbed and reconstituted samples. Most of the data for undisturbed soil relates to clay due to the problems associated with recovery of quality samples in sand. Because the stress conditions and the effects of disturbance in the undisturbed samples at the start of the test are unknown, the $K_o$ values so defined are incremental in nature and it has been suggested that they provide a lower bound to the field situation. Aging in the field may also cause differences with the laboratory measured values. Furthermore, the stress history relationships have been evaluated for the condition of simple mechanical overconsolidation. Stress history characteristics can be influenced by many factors, some quantifiable, many not. Schmertmann (1985) discusses many of the non-tangible factors that may influence $K_o$, i.e. cementation, deposition environment, tectonic environment, strain history, stress history, environmental factors, dessication, aging, etc. The effect of any of the above, singularly or in combination, will cause the in-ground $K_o$ value to deviate markedly from that predicted based on the laboratory derived relationships. It is also to be expected that these relationships will be in error for conditions other than level ground. Hence where possible the horizontal stress should be measured in situ.

1.3 Laboratory Techniques for Evaluating $K_o$

Schmertmann (1985) listed seven techniques for laboratory determination of $K_o$, applicable mainly to cohesive soils. More recent techniques are included in the following list:

- Triaxial test using indirect or burette method (Bishop and Eldin, 1953)
• Triaxial test with lateral strain indicator (Bishop and Henkel, 1957)
• Triaxial test, capillary pressure measurement (Skempton, 1961; Burland and Maswoswe, 1982)
• \( K_q \) triaxial cell (Campanella and Vaid, 1972)
• Horizontal and vertical oedometer tests (Zeevaert, 1953; Poulos and Davis, 1972)
• Proportional loading tests in triaxial cell (Andrawes and El-Sohby, 1973)
• Hydraulic fracturing (Al-Shaikh-Ali, 1977)
• Triaxial deviator stress method (Chang et al., 1977)
• Lateral stress direct shear test (Dyvik et al., 1987)
• Lateral stress oedometer (Dyvik et al., 1985; Senneset, 1989)

For granular soils, these tests are usually performed on specimens formed in the laboratory. Also inherent in the above procedure is the assumption that the \( K_q \) value is uniquely related to overconsolidation ratio, at least during 1D unloading. This has been questioned by Jefferies et al. (1987) for recent offshore deposits. For the Beaufort Sea sediments studied, it may be that physiochemical processes contribute to the higher than expected \( K_q \) values, although some doubt also exists as to the actual OCR values used for the site. Undrained shear strengths measured in the Beaufort Sea sediments, which are usually reliable indices for estimating stress history, indicate OCR values higher than obtained from laboratory consolidation tests.

The effects of secondary compression and aging on \( K_q \) are areas of interest in geotechnics. Schmertmann (1983) first raised the question; Jamiokowski et al. (1985) reviewed available data and concluded that no definite change in \( K_q \) occurs with time.

It is the author's opinion that \( K_q \) in NC soils (\( K_q < 1 \)) increases with
geologic time to a value of unity and that in OC soils ($K_o > 1$) a reduction to unity occurs. The indications for this variation from published data are discussed in the following paragraph. For any soil, the variation of $K_o$ during loading and unloading will be controlled by the stress-strain characteristics of the material.

Based on a consideration of post-deposition history, one might conclude:

• In the slurry form, at the moment of deposition, $K_o$ is not easily defined (may be close to unity as for all non-viscous fluids) since both $\sigma'_v$ and $\sigma'_h$ are very small

• After deposition and with burial, the $K_o$ value approaches $(K_o)_{NC}$ over a short period of time, where

$$K_o = 1 - \sin \phi$$

• As the soil becomes progressively buried, the lateral strain condition is modified and stress redistribution as a result of anisotropic hardening occurs so that $K_o$ increases to a value of unity. This is consistent with stress measurements in NC sedimentary rocks unaffected by major tectonic activity (Brown and Hoek, 1978; Hergert, 1988).

The suggested variation of $(K_o)_{NC}$ with time after deposition is illustrated in Fig. 1.3. SBPM data from the Beaufort Sea presented by Graham and Jefferies (1986) concur with the idea of $K_o$ reducing from unity to $1-\sin \phi'$ during the initial period after deposition. Tests performed in a hydraulic sand fill one month after placement gave $K_o$ values around 1. Values measured one year later gave $K_o$ values in better agreement with the Jaky expression.

The $C_o/C_c$ concept proposed by Mesri and Godlewski (1977) can also be applied to evaluating $K_o$ variations. Mesri and Castro (1987) show that $K_o$
SLURRY CONDITION

RATE OF CHANGE DEPENDENT ON SOIL TYPE AND DEPOSITION ENVIRONMENT

TO CONDITION $K_0 \approx 1$
OVER GEOLOGIC TIME FOR NC STATE WITH INCREASING DEPTH OF BURIAL.

$$S = \frac{\Delta K_0}{\Delta t} = f\left(\frac{C_\alpha}{C_c}\right)$$

where: $t_p$ = time for end of primary consolidation

Fig. 1.3 Change in $(K_0)_{NC}$ with time after deposition.

increases with time for NC soils and that the $C_\alpha/C_c$ method can be used to adjust laboratory values to the field condition. However, much more field and laboratory data are required to evaluate the time dependence of $K_0$ in both NC and OC soils. Irrespective of the problems that may exist with the field application of laboratory determined $K_0$ values, the use of laboratory data is one of the few checks available on measured field data.

Calibration chamber (CC) testing is a large scale form of laboratory testing. The application of CC results to the interpretation of in situ measurements is considered in Chapter 2.

1.4 Conditions for the In Situ Measurement of Horizontal Stress

As discussed earlier, the ideal measurement of $\sigma_h$ in situ requires that no disturbance is imparted to the soil and that the measurements are made
under conditions of no lateral strain. Other considerations also exist.

Ideal criteria for measuring the true in situ $\sigma_h$ would include:

- installation of measuring apparatus with no lateral movement of soil, either inward or outward, $\varepsilon_h = 0$
- no shear stresses applied to the soil resulting from friction between the soil and instrument during insertion, $\tau_{vh} = 0$
- no induced pore pressures during or after probe insertion,
- measurement of $\sigma_h$ obtained by infinitesimal (ideally zero) expansion/contraction of probe,
- no change in $\sigma_v$, $\sigma_h$ or $\tau_{vh}$ due to measurement procedure,
- very sensitive measuring system for stress and pore pressure, and
- universal installation technique applicable for any soil type.

Ideally, the measurement of $K_0$ is a zero strain condition (by definition). However, this requirement is impossible to attain with presently available equipment where measurements are obtained at some minimal displacement. For this reason, the measurement of $K_0$ can be considered a small-strain technique.

The importance of the above conditions varies according to soil type, i.e. soil stiffness is important in determining the effect of stress or strain relief. In general, the stiffer the soil the more severe the effects of installation become. It is worth considering these ideal conditions in relation to the established techniques for measuring the in situ lateral stress.

The most widely used field techniques for measuring the in situ lateral stress are:

- self-boring pressuremeter (SBPM)
- self-boring load cell (SBLC)
- hydraulic fracture test (HFT)
- push-in total stress cells (TSC)

The individual test techniques and data interpretation are not considered here as detailed comments are contained later in the thesis. In relation to the seven ideal requirements listed above the following observations are made:

• The main problem associated with the self-boring probes is the disturbance caused during installation, the effects of which are indeterminate.

• Even if ideal installation of a self-boring probe were possible, the shearing at the probe-soil interface would modify the near-field stress distribution to various degrees depending on the characteristics of the soil being penetrated. Furthermore, only in normally consolidated soils would the effective lift-off pressure represent the in-situ horizontal effective stress. In overconsolidated soils, the lift-off pressure would correspond to an effective horizontal yield stress, indicative of the maximum past horizontal pressure to which the soil had been subjected.

• Both the HFT and TSC techniques require the installation of a probe or blade into a disturbed soil. The change in the in situ stress regime is corrected for in an arbitrary manner. Notwithstanding this, the initial stress field is modified and the results may be ambiguous.

The development of non-destructive techniques such as in situ shear wave velocity measurements may satisfy the conditions for the idealized requirements listed above. However, the evaluation rather than direct measurement of $\sigma_h$ from these techniques will depend on the uncertainty of any parameter relationships involved, i.e. the dependence of $V_s$ on both structure and stress. Similar conditioning relationships will also prevail for the inter-
pretation of full-displacement stress and pore pressure measurements. This is discussed later in this thesis.

1.5 Research Objectives

The self-boring pressuremeter is considered by many to be the best low displacement method for measuring \( \sigma_h \) in situ. However, the effects of boring disturbance, equipment characteristics and shear stresses induced at the soil-instrument interface and variations in test procedure cause problems in the interpretation of \( \sigma_h \) (Jamiolkowski et al., 1985; Mair and Wood, 1987). The problem is the determination of the often erratic disturbance that has occurred during probe insertion and adjusting/evaluating the data accordingly.

The application of full displacement probes to estimate \( \sigma_h \) has been developed with the idea of inducing large yet repeatable degrees of disturbance to the soil and then correcting for the resulting effects of the insertion on the measurement of \( \sigma_h \). This methodology relates the two differing soil responses according to the induced strain level, i.e. that the measurements of stress or pore pressure during full-displacement penetration testing are controlled to some degree by the pre-penetration horizontal effective stress. Furthermore, depending on the loading conditions and the interpretation technique employed to evaluate the pre-penetration lateral stress, the back-calculation technique is linked to both the initial stress state and the stress-strain response of the soil under the applied loading. The correction technique is based on empirical or semi-empirical correlations.

The objective of this research is to evaluate the soil response during the installation of full displacement probes. Comparisons of the horizontal
stresses measured in both sand and clay by various types of probe will be made to evaluate the factors controlling soil response to both self-boring and full displacement installation. The distribution of stresses and pore pressures around the different penetrometers are also considered. The measured stresses are also interpreted with the objective of predicting the in situ pre-penetration stress. The objective is to see if it is possible to interpret the large strain measurements in order to back-calculate the initial lateral stress condition and if so, how sensitive these techniques are to the algorithm used and soil parameters employed in the model. Reference values of horizontal stress (see Chapter 6 for discussion) are determined by both field measurements and laboratory tests for comparative purposes. Index parameters are defined as indicators for profiling variations in the stress history.

1.6 Thesis Layout

The conditions induced during the installation of full displacement probes are considered in Chapter 2 and relationships are presented which link soil behaviour at small and large strains. The stress distribution around a penetrating probe is considered in terms of the controlling influence of the pre-penetration stress state and cavity expansion methods are used to evaluate the measured stresses and pore pressures during full-displacement penetration testing. Laboratory methods for measuring and/or indexing the lateral stress conditions are briefly discussed.

The equipment used during the in situ and laboratory testing program is discussed in Chapter 3 and the research sites described in Chapter 4.

Chapters 5 and 6 present the results and interpretation of the field and laboratory tests performed. These results are discussed and concluding comments presented in Chapter 7.
CHAPTER 2

2. LATERAL STRESS FROM FULL-DISPLACEMENT PROBES: CONSIDERATIONS

2.1 Introduction

The evaluation of in situ horizontal stress can be classified into four main groups according to the type of measurement made:

1) Direct methods
2) Semi-direct or back-extrapolation methods
3) Indirect methods
4) Empirical methods

Direct methods include tests performed using the self-boring pressure-meter and self-boring load cell. Direct methods suffer from the often large effects of even small degrees of disturbance, the consequences of which become more important as the soil stiffness increases.

Semi-direct or back-extrapolation methods. Developments in this area include the stepped blade and wedge blade, both of which require additional calibration or correlations at specific sites prior to general use.

Indirect methods are used whereby a lateral stress value is measured during or after the installation of a full-displacement probe. In some cases, the dissipation of stress and pore pressure induced during insertion can be monitored with time so that an equilibrium value for the inserted probe can be obtained. Each of the full-displacement methods, i.e. lateral stress cone, causes significant but repeatable disturbance to the soil.

Empirical methods are an important source of information for evaluating the stress history of soil deposits. Existing correlations are generally
derived from laboratory or calibration chamber data and modified to incorporate field parameters, an example of this being the dilatometer $K_q$ correlations presented by Baldi et al. (1986).

A critical review of the current methods available for measuring or indexing the state of lateral stress is given in Appendix A.

As discussed earlier in Chapter 1, it is very difficult with presently available equipment to obtain a measurement of the true in situ horizontal stress, $\sigma_h$. Additional uncertainty arises as to the choice of a reference value against which measured values can be evaluated. In many comparisons of this type, the results from the self-boring pressuremeter test (SBPMT) are taken as the reference values, even though this technique has been shown to be unreliable, especially in sands and stiff clays (Jamiolkowski et al., 1985; Hawkins et al., 1990). Many of the problems associated with the SBPM as a method for measuring $\sigma_h$ have been attributed to the insertion of the probe into the ground, i.e. disturbance during drilling (Hughes, 1973; Denby, 1978; Ghionna et al., 1982; Benoit, 1983). Other effects due to membrane and equipment compliance, test procedures, etc., have also been recognized (Dalton and Hawkins, 1982; Howie et al., 1990; Mair and Wood, 1987). These are reviewed briefly in Appendix A.

In an attempt to evaluate qualitatively the effects of disturbance during the SBPM drilling process in sand, Bellotti et al. (1987) performed two types of SBPMT in a calibration chamber (CC). The "ideal installation" of the probe was attained by placing the probe in the centre of the CC before sample formation and subsequently air-pluviating the sample into the chamber to obtain the required relative density. Self-bored conditions were attained by drilling the probe into the already pluvially deposited sand. All PM tests were conducted strain controlled. The chamber stresses held constant
on both boundaries of the chamber ($\Delta \sigma_v = \Delta \sigma_h = 0$). The lift-off stress for each strain arm, $p_o$, was determined from visual inspection of the early part of the expansion curve. The results of the two types of test are summarized in Fig. 2.1.

For ideal installation, noticeable differences exist between $p_o$ from the SPBM and $\sigma_h$ applied to the chamber. Significant scatter also exists between the lift-off stress for each of the three strain arms, which may be due in part to individual strain arm compliance. The fact that most of the data gives $p_o > \sigma_h$ may indicate the existence of stress concentrations around the rigid probe induced during the 1D consolidation stage. Based on a single test using the self-boring load cell (SBLC) Bellotti et al. (1987) suggest that no induced stress concentration occurs for the ideal installation case. Huntsman (1985) evaluated the effect of stress concentrations around a lateral stress cone as a result of applied chamber stresses and suggested that the measured stress, $\sigma$, would approximate to:

$$\sigma = 1.5 \sigma_h + 0.04 \sigma_v$$  \hspace{1cm} (2.1)

From the limited published data, it would also appear the SBLC underestimates the true lateral stress (Charles and Watts, 1987; Tedd and Charles, 1983; Penman and Charles, 1985).

Irrespective of the individual arm scatter, for ideal installation the average $p_o$ values are reasonably consistent for $\sigma_h < 200$ kPa. Individual strain arm scatter increases as the relative density and stress level increase. From Fig. 2.1(a) it would appear that even for the case of ideal installation $p_o$ gives variation of $\sigma_h$ between -10% and +50%. Belloti et al. (1987) do, however, provide more consistent data after modifying the PM strain arm design.
Fig. 2.1 Comparison of lift-off pressures from SBPM and horizontal chamber stress for (a) conditions of ideal installation, and (b) installation by self-boring (data from Bellotti et al., 1987).
Figure 2.1(b) indicates that for the self-bored installation, intended to be indicative of the field situation, in almost all cases \((p_o)_{av}\) is less than \(\sigma_h\). Using only the average \(p_o\), Bellotti et al. (1987) quote:

\[
\frac{(p_o)_{av}}{\sigma_h(\text{CC})} = 0.47 \pm 0.28 \quad (2.2)
\]

The scatter in the individual strain arm data is similar to that for the ideal installation.

Based on the above, it would appear that the mechanical design of the SBPM strain arms is inadequate for determining accurately the small strain response required for measuring the true in situ lift-off pressure, even under ideal conditions. Even under controlled laboratory conditions, significant disturbance occurs as a result of self-boring. Considerable doubt must therefore exist with respect to the validity of data obtained in the field, especially at the low strain portion of the expansion curve.

This is evidenced by the field data presented by Robertson (1982) and Howie (1991) for pressuremeter tests in sand. A comparison of \(\sigma_h\) determined by various types of pressuremeter is shown in Fig. 2.2. The scatter in the SBPM data is larger than that associated with three types of full-displacement pressuremeter. The reduced scatter produced by full-displacement probes suggests that lateral stress measurements of this type may provide a more reliable basis from which to evaluate the in situ pre-penetration horizontal stress. The usefulness of this technique would depend on:

- A link between small strain and large strain behaviour of soil. This requires some interdependence on the two extremes of soil response.
Fig. 2.2 Comparison of measured horizontal stress from self-bored and full-displacement pressuremeters (adapted from Howie, 1991).
Fig. 2.3 Effect of varying $\sigma_h$ on SBPM pressure expansion curve according to Hughes (1989) interpretation.

- Development of interpretation techniques for full-displacement measurements to provide repeatable and reliable estimates of soil parameters.

These two points are considered separately below.

2.2 Correlation of Soil Behaviour at Varying Strain Levels

Several types of interpretation technique exist whereby the complete pressure-expansion curve obtained from the SBPM test is utilized for parameter determination. Hughes (1989) employs a four parameter model ($G, \phi_{cv}^\prime, \sigma_h^\prime, \nu$) to evaluate SBPM field data in sand whereby the influence of $\sigma_h^\prime$ is to define the overall position of the curve referenced to the stress origin (Fig. 2.3). Jefferies (1988) uses both the loading and unloading phases of
the test to interpret data in clay. The curve shape and position is determined by $G$, $S_u$, $\sigma_h$, and $u$ (Fig. 2.4). These methods, which have been reported to give good estimates of the conditioning parameters (Appendix A.2.3), illustrate the effect of $\sigma_h$ throughout the expansion curve, i.e. both at small and large strain. Both techniques have been developed to overcome the effects on the early part of the PM expansion curve caused by disturbance during self-boring installation.

![Fig. 2.4 Influence of variables for undrained SBPMT in clay (after Jefferies et al., 1988)](image)

Byrne et al. (1990) demonstrate the interdependence of large strain parameters on the initial state of soil by linking $G^*$, the unload-reload modulus from PM tests, to the small strain maximum shear modulus, $G_o(G_{\text{max}})$ for a wide range of loading and unloading conditions. $G^*$ is a parameter measured at any particular cavity strain; consequently it depends on both the
stress and void ratio changes induced in the soil during expansion. $G^*$ and $G_o$ are related through the factor $\alpha_p$ which was determined from the results of a finite element elastic plastic analysis incorporating nonlinear elastic behaviour during unloading (Fig. 2.5). $G_o$ values from SBPMT unload-reload loops ($G_{HH}$) and resonant column tests ($G_{VH}$) indicate the requirement to apply a disturbance factor ($\alpha_D$) and an anisotropy factor ($\alpha_A$) to obtain good agreement between two modulus measurements. The moduli are related according to:

$$G_{HH} \text{ (SBPM)} = \alpha_D [\alpha_p G^*(SBPM)]$$

(2.3)
\[ \alpha_p = \frac{G^*/G_o}{\alpha_D} \text{ (from Fig. 2.5)} \quad (2.4) \]

and \( \alpha_D \) is used to correct the SBPM \( G^* \) for disturbance effects. For seismic crosshole data:

\[ G_{VH} = G_{HH} \alpha_A = G^* \alpha_p \alpha_D \alpha_A \quad (2.5) \]

where \( \alpha_A \) considers the anisotropic stress conditions for the respective moduli. Byrne et al. (1990) also show that:

\[ \alpha_A = \frac{G_{VH}}{G_{HH}} = \left[ \frac{(1 + K_o)/2K_o}{2K_o} \right]^{0.5} \quad (2.6) \]

Hence the above equations suggest that the modulus \( G^* \) at any value of cavity strain can be related to the small strain value \( G_o \) and that both values are related to the in situ stress condition. This is also confirmed by the laboratory studies performed at UBC. Negussey (1984) showed that the initial unloading modulus in triaxial compression, performed at any axial strain value, corresponded well with the small strain low amplitude modulus determined in the resonant column test.

The three cases described above demonstrate the link between small and large strain behaviour based on theoretical/analytical considerations. In geotechnical engineering, where the use of relationships based on experience is widespread, many generally held tenets also demonstrate this same link between parameters determined at different strain levels. The Jaky (1944) expression which relates \( K_o \) to the large strain friction angle was described in Chapter 1. For clays, various authors have successfully related
plasticity index (PI) to \( (K_{o})_{NC} \) in a very empirical manner. Brooker and Ireland (1965), Alpan (1967) and Massarsch (1979) suggested relationships of the form:

\[
(K_{o})_{NC} = A + B(PI) \quad (2.7)
\]

where \( A \) and \( B \) are constants determined from laboratory tests on various soils. Sherif and Strazer (1973) relate \( K_{o} \) to the liquid limit as:

\[
K_{o} = \lambda + \alpha(OCR-1) \quad (2.8)
\]

where \( \lambda \) and \( \alpha \) are determined from Fig. 2.6.

Fig. 2.6 Liquid limit vs \( \lambda \) and \( \alpha \) (after Sherif and Strazer, 1973).
The dependence of the normalized strength ratio \( S_u/\sigma'_v \) on the overconsolidation ratio (OCR) in clays is a further example of the interdependence of small and large strain behaviour. Critical state soil mechanics (Wroth, 1984) and the SHANSEP principle (Ladd et al., 1977) indicate that:

\[
(S_u/\sigma'_v)_{OC} = (S_u/\sigma'_v)_{NC}^{\Lambda} \tag{2.9}
\]

where \( \Lambda \) is the plastic volumetric strain ratio. OCR is defined as:

\[
OCR = \sigma'_v/\sigma'_{vm} \tag{2.10}
\]

where \( \sigma'_{vm} \) is the maximum past effective vertical pressure experienced by the soil and \( \sigma'_v \) is the in situ vertical effective stress. \( \sigma'_{vm} \) can be considered as a small strain elastic response which occurs during reloading to the maximum past pressure experienced by the soil and thus indicates a change from elastic to plastic strains, whereas \( S_u \) is a large strain (plastic) response. Mesri (1989) has shown that for many clays \( S_u/\sigma'_{vm} \) is equal to 0.22. Windisch and Wong (1990) suggest a value of 0.27 for eastern Canada marine clays. If \( S_u/\sigma'_{vm} = (S_u/\sigma'_v)_{NC} \), it follows that:

\[
\frac{S_u}{\sigma'_{vm}}_{OC} = (S_u/\sigma'_v)_{NC}^{\Lambda} \tag{2.11}
\]

which agrees with Eq. (2.9) when \( \Lambda = 1 \). The typical range of reported values for \( \Lambda \) is from 0.8 to 1.35 (Jamiolkowski et al., 1985), with an average of around 0.98. For most soils, these constants can be used in Eq. (2.9) to provide reliable estimates of OCR from in situ field vane strengths (Mayne and Mitchell, 1988). Through OCR, \( S_u \) can also be related to \( K_o \).
Calibration chamber tests have provided another means by which soil response at small and large strain can be compared. Results have shown that cone penetration resistance ($q_c$) in sands can be correlated to $G_o$ since both parameters are governed essentially by $D_r$ and $\sigma'$ (Robertson, 1982; Belotti et al., 1986; Rix, 1984; Jamiolkowski and Robertson, 1988). Field data obtained in Po River sand generally confirm the trends suggested by CC results. Figure 2.7 demonstrates the well-defined relationship between $q_c$ and $G_o$ obtained from CC tests. The field data from Po River and Gioia Tauro sands and gravel are in good agreement with the trends from CC tests. It has also been demonstrated that the cone penetration resistance is almost completely governed by $\sigma'_n$ in sand and that the relationship is of the form (Houlsby and Hitchman, 1988):

![Diagram](image)

Fig. 2.7 CC data for $q_c$-$G_o$ correlation (after Baldi et al., 1989).
where \( A \) varies according to the sand state.

In conclusion, it appears that the large strain response of soil may be influenced to a great extent by the small strain properties and that backcalculation may be a feasible approach for evaluating one extreme of soil response from the other. The correlations presented above have also been applied to soil types ranging from clay to sand although the individual correlations are soil-type specific.

2.3 Stress and Pore Pressure Distribution Around Full-Displacement Probes

Section 2.2 has considered the link between soil behaviour at small and large strains and shown that many correlations exhibiting this interdependence are in widespread use. As stated in Section 2.1, the usefulness of the correlation of large strain behaviour to small strain properties also requires a method(s) of interpretation which can provide repeatable and reliable estimates of the parameters of interest. In terms of in situ measurement of lateral stress, it is important to identify the factors which affect the measured values so that a meaningful interpretation can be made. Hence information regarding the stress and pore pressure distribution around full displacement penetrometers is required for different stress and pore pressure measuring locations and differing probe geometries. This is considered in this section.

Full displacement lateral stress sensing probes were developed to induce repeatable degrees of disturbance; the problem then becomes one of relating the measured lateral stress to the pre-penetration value as opposed to evaluating whether or not the soil had been disturbed as is the case during SBPM.
installation. The idea of predicting small strain behaviour from large strain parameters has been considered above.

In the ideal case for undrained penetration, the penetration lateral stress, $\sigma_*$, measured by a full-displacement probe results from two components:

$$\sigma_* = \sigma_{ho} + \Delta \sigma$$  \hspace{1cm} (2.13)

where:

$\sigma_{ho}$ = in situ total horizontal stress

$\Delta \sigma$ = total stress increment caused by insertion

In any particular soil, the magnitude of the total stress increment caused by insertion is made up of both stress and pore pressure components and can be expected to be related to the displacement caused during penetration of a probe. The idealized change in the lateral stress coefficient (defined in terms of an effective stress ratio) for various in situ testing probes is shown schematically in Fig. 2.8. Although this simplified representation is instructive, it is, however, complicated by the fact that for each test method the stress/strain paths are very different and even under undrained conditions no single curve exists. The relative positions of the tests are also very subjective and dependent on individual probe characteristics.

A review of methods presently available for evaluating the stress distribution around full displacement probes is given in Appendix B. Many of the techniques have been specifically developed for piles but can be equally used for in situ testing results. Solutions exist for both drained and undrained conditions during insertion.

Penetration of a probe in clay gives rise to excess pore pressure as the soil is displaced both vertically and laterally. Cavity expansion methods
Fig. 2.8 Idealized change of lateral stress coefficient, $K$, caused by full-displacement probes (Sully and Campanella, 1989).

indicate that the magnitude of the excess pore pressure depends on the location of the pore pressure measurement and on soil parameters ($G$, $\sigma'_h$, $S_u$, $S_t$, OCR, etc.). This is confirmed by the more rigorous strain path approach described by Baligh (1986). In terms of soil response to undrained loading, the excess pore pressure ($\Delta u$) components close to the probe (plastic zone) are:

$$\Delta u = \Delta \sigma_{oct} + \Delta u_s$$  \hspace{1cm} (2.14)
where \( \Delta \sigma_{\text{oct}} \) is the change in octahedral stress and \( \Delta u_s \) is the pore pressure resulting from shear. Cavity expansion methods consider the shear related pore pressures in an empirical manner (Vesic, 1972). In the strain path method (Baligh, 1986), shear induced pore pressures are related to a yield shear strain. In both cases:

\[
\Delta u_s = \sigma' / \chi
\]

where \( \sigma' \) is a defined stress term and \( \chi \) is a defined pore pressure parameter. Comparison of Eqs. (2.13) to (2.15) indicates the dependence of the measured pore pressure on the in situ pre-penetration stress. Also, at a particular stress level, the theoretical solutions suggest that the magnitude of \( \Delta u \) in saturated clays depends primarily on \( S_u \) and to a lesser extent on \( G \).

It has also been established that a gradient of pore pressure exists around a penetrating cone (Robertson et al., 1986) and that the gradient can be qualitatively related to changes in normal and shear stresses as the soil moves around the cone tip (Baligh, 1986; Sully et al., 1988). As suggested by Fig. 2.9, measurement of the gradient around a penetrating cone should provide information related to the stress distribution during undrained penetration in clay. The trends in the data indicate that the soil is unloaded as it passes the tip and that the effect of the unloading is more pronounced as the soil stiffness increases.

For CPTU in clean sands, the penetration process can be considered as drained and no large excess pore pressures are generated. Gillespie (1990) demonstrates that, irrespective of relative density, for \( \sigma'_v < 200 \text{ kPa} \), the excess pore pressures behind the tip are zero or negative of static equilibrium and result from the rapid unloading that occurs due to the cone geometry.
in this region. Pore pressures on the face of the cone are approximately equal to hydrostatic if filter compressibility effects are not present. Consequently, in sands penetration pore pressures provide very little information regarding stress changes along the probe. Cone resistance ($q_c$) and sleeve friction ($f_s$) are generally more useful parameters for evaluating sand response during penetration.

Campanella and Robertson (1981) and Hughes and Robertson (1985) examined the possible variation of lateral stress around a penetrating cone with respect to changes in measured sleeve friction. Tests in sand show a marked increase in $f_s$ between 10 cm and 25 cm behind the tip (for 10 cm$^2$ cone). For larger distances (greater than 25 cm, or 7D, where D is the cone diameter), $f_s$ is essentially constant. For a constant soil-steel friction angle, a similar distribution for $q'_h$ can be determined. Hughes and Robertson (1985)
suggested the existence of high stress gradients, similar to the pore pressure gradients described earlier for clays, as the cone tip approaches and passes an element of soil.

Soil is thus unloaded as it passes the tip of the penetrometer. The shoulder of the cone tip, where this transition from loading to unloading behaviour occurs represents a singularity. Analysis using the strain path method in clays has shown that (Baligh and Levadoux, 1980; Teh and Houlsby, 1988):

- Difficulties exist in modelling soil response at the cone shoulder where large stress/pore pressure gradients exist.
- At the transition point, the vertical strain \( \varepsilon_v \) is close to zero. Below the tip \( \varepsilon_v \) is compressive, becoming slightly tensile behind the tip.
- Along the shaft, radial, circumferential and shear strain contours are concentric about the cone axis, similar to the condition of an expanding cylindrical cavity.

No analysis has been performed to define the strain contours that occur for penetration in sand, but similar strain contours to those predicted for clay could be expected. It must be noted that the strain pattern will not change much for different soils. The induced stresses and pore pressures, however, will depend very much on the properties of the soil being penetrated.

Levadoux and Baligh (1980) and Teh (1987) predict the existence of pore pressure gradients in soft clay. Simple cavity expansion methods suggest a ratio of 4/3 between the stresses and pore pressures at the tip and behind the tip, independent of OCR, if the same \( G \) and \( S_u \) values are applied to both
the spherical and cylindrical cavity formulations. Coop (1987), May (1987) and Gillespie (1990) compare the theoretical distribution of pore pressures determined by the strain path method with field data and obtained notable differences even for NC and lightly OC clays. The distributions shown in Fig. 2.9 suggest that departure from the theoretical solution increases as OCR increases.

Measurements in calibration chambers (Baldi et al., 1986; Huntsman, 1985) suggest that the lateral stress acting on the friction sleeve located immediately behind the tip is low compared to that on the tip and approximates to the pre-penetration horizontal stress for loose sand. This is confirmed by limited field data (Jefferies et al., 1987). As the sand becomes denser, stress amplification effects occur. This is reviewed further in Appendix A for the various types of full-displacement lateral stress probes. However, at locations further up the shaft no direct information is available. Theoretical solutions indicate a constant lateral stress condition behind the tip (Vesic, 1972; Carter et al., 1986).

Marchetti (1979) suggests that the disturbance caused during flat plate penetration (i.e. dilatometer) is less than that associated with a cone, simply based on the lower apex angle at the tip (20° for the DMT as opposed to 60° for the CPT). Studies by Baligh (1975) on the penetration of long wedges confirm this observation. However, the model tests reported by Baligh (1975) are 2D in nature and are misleading since the 3D effects of cone penetration are not considered. Cone penetration is axisymmetric and facilitates increased unloading effects; the generally available flat penetrometers cannot be considered as infinitely long and thus the deformation may lie somewhere between the 2D and 3D idealizations. The width to thickness ratio of the flat penetrometer may noticeably affect the stress measured at the centre
of the blade if stress concentrations due to edge effects are significant. No field or laboratory data are available concerning this.

A 3D strain path method has been developed by Huang (1989) which permits a complete evaluation of the flat plate penetration problem (see Appendix B.3). Huang (1989) concludes that the differences between the strain fields for cone and plate penetrometers are more than expected solely from apex angle variations. Strain levels around flat plates appear to be lower than those obtained with cones, but are much more complicated due to the finite width of the plate. The results of the analysis do, however, suggest that the stress behind the tip for a cone should be higher than behind a DMT tip due to the larger disturbance induced during insertion. This corroborates earlier results obtained by Davidson and Boghrat (1983) from laboratory measurements in sand. Approximate analysis of flat plate penetration problems has sometimes been performed by representing the blade as an equivalent cylindrical cavity, or by use of the elastic solution presented by Finn (1963).

Several methods exist to evaluate the stress distribution around penetrating probes. The validity of each approach depends on both soil and probe characteristics and these effects are examined in Chapter 6.

2.4 Application of Calibration Chamber Results to In Situ Measurements

Performing field tests in large calibration chambers where sample characteristics and boundary stresses can be controlled has provided invaluable insight into the dominant modes of soil response measured by full-displacement penetrometers. Many of the recent advances in interpretation techniques have been made possible by results from CC tests (Baldi et al., 1982, 1988). It is not the objective of this section to provide a review of
CC procedures and developments, rather to discuss some of the limitations involved when trying to evaluate lateral stresses from CC measurements.

It is recognized that chamber size and boundary conditions affect the results obtained and corrections to measured data need to be made. These corrections are well established for cone resistance in clean sands and have been derived from comparisons of results obtained in different size chambers (Parkin and Lunne, 1982; Parkin, 1988). Very little information exists for other measured parameters. For measuring locations behind the tip, the standard \( q_c \) correction may not be applicable since the boundary will have a different effect on the soil during unloading than during initial loading.

Parkin (1988) evaluates sleeve friction data in clean silica sand obtained at the Norwegian Geotechnical Institute (NGI) and suggests that for a 10 cm\(^2\) cone:

\[
 f_s = A (q_c)^{1.6} \tag{2.16}
\]

The relationship is dependent on the size of the cone being used, thus also suggesting that \( q_c \) and \( f_s \) are influenced differently by the applied CC boundary conditions. Masood (1990) compares the DMT bearing resistance and limit pressure (\( q_D \) and \( p_1 \)) from DMT tests in the Berkeley CC and concludes that the two parameters are influenced in a different way by soil compressibility. As a result \( q_D/p_1 \) would vary depending on soil type, \( D_r \) and chamber size.

Masood (1990) suggests that both \( p_o \) (DMT) and \( \sigma_{LS} \) obtained from lateral stress cone tests (LS-CPTU) are affected by chamber size but insufficient data are available to clarify the exact dependence. The author attempted a review of published CC data to evaluate the effect of chamber size on measured parameters at different locations on a probe. Insufficient data did
not permit any conclusions to be drawn. Jefferies et al. (1987) assume that the \( q_c / \sigma_{LS} \) ratio measured during lateral stress cone tests in a CC is a constant value for a particular chamber size. They then apply the same CC correction factors to both \( \sigma_{LS} \) and \( q_c \) to account for chamber size effects. Intuitively this would appear to be incorrect and is confirmed in part by the data presented by Masood (1990).

The application of correlations obtained from CC tests to a field situation should be performed with care. Certain well established relationships, i.e. \( q_c - D_r \) and \( q_c - \phi \) are available and work well provided soil compressibility is considered (Robertson and Campanella, 1983). However, in some situations the CC derived correlations can be in error. Baldi et al. (1986) developed a relationship between \( K_o, K_D \) and \( q_c / \sigma_v \) from CC tests which was found to over-predict the in situ \( K_o \) (see Appendix A.4.1). The CC relationship was revised based on field data in Po River sand. The field calibrated relationship suggested by Baldi et al. (1986) is compared in Fig. 2.10 against a similar CC derived function presented by Jamiolkowski and Robertson (1988).

The CC derived equation suggests that \( K_o \) is very sensitive to variations in \( K_D \) and hence not very useful for evaluating \( K_o \) from measurements of \( K_D \) and \( q_c \). The field correlation suggests a more acceptable interdependence of the parameters. If the Baldi et al. (1986) relationship is assumed to be correct (since it is field calibrated), then the difference in the trend for the two relationships shown in Fig. 2.10 would suggest that CC tests are unable to correctly simulate the in situ stress response of granular soils at locations behind the penetrometer tip where the soil has undergone some degree of unloading. As discussed earlier, this may result from the loading/unloading stress paths and the effects of the chamber boundary on parameters undergoing
stress/strain reversal. Fabric, environmental factors and other in situ effects not modelled in the chamber may also be important factors.

As a final point, the interpretation of CC data for sand is often indexed using the state parameter ($\psi$) approach (Been and Jefferies, 1985). This is ideal for CC tests where both $\sigma_h$ and void ratio are known. Application of the state parameter approach to in situ data appears promising but requires further confirmation. As suggested by Sladen (1989), the method may necessitate additional specialized in situ tests to confirm the void ratios evaluated on the basis of CPT data, i.e. nuclear density (Tjelta et al., 1985; Sully and Echezuria, 1988) or electrical resistivity (Zuidberg et al., 1987) tests. Furthermore, an a priori knowledge of $\sigma_h$ is required.
Data presented by Huntsman (1985) for CC tests using a lateral stress cone penetrometer are shown in Fig. 2.11. The range of chamber stresses ($\sigma'_v$ and $\sigma'_h$) used in the study varied between 100 kPa and 600 kPa, hence the results are valid for the depths of interest common to most geotechnical problems. More specifically, stress measurements performed as part of this research have been conducted to maximum depths of 30 m where $\sigma'_v$ attains a value of about 300 kPa. The amplification of lateral stress, $A_{LS'}$, is plotted against $D_r$ and $\psi$. The scatter in the two plots is identical and leads to the conclusion that for field measurements $D_r$ is a suitable index parameter and can be used for comparative purposes.

Fig. 2.11 Comparison of CC data indexing tests by (a) relative density, and (b) state parameter.
2.5 Conclusions

The points discussed above have emphasized the following important aspects which relate to the application of full-displacement test methods for determination of the in situ horizontal stress:

- Behaviour of soil at large strain is governed to a significant degree by small strain properties or conditions.
- Analytical techniques exist by which the large strain parameters can be interpreted to provide estimates of the controlling small strain conditions.
- Available interpretation techniques, irrespective of complexity, all rely to some degree on empirical inferences. The methods may be overly sensitive to the arbitrarily chosen soil parameters used in the models.
- Limited published data are available, especially in sand, for evaluating the distribution of stress around full-displacement probes.
- Data from calibration chamber testing have been instrumental in the development of specific interpretation techniques for full-displacement testing. More importantly, the idea that full-displacement techniques could be used to evaluate small strain parameters developed directly as a result of CC results.
- The direct application of CC derived relationships to a field situation has to be performed with caution as salient points related to the field response cannot be represented in chamber studies.
- Similarly, the above is true with respect to data from laboratory tests on both undisturbed and reconstituted samples. However, in laboratory tests the problems of boundary effects are not so pronounced as for a full-displacement test in a CC. The main concerns
are related to the initial stresses in the sample and the effects of disturbance, aging and other secondary factors on measured $K_0$ values.

- Laboratory tests do provide important information regarding possible limits for field $K_0$ values.
- For most soils under laboratory conditions of 1D unloading there appears to be a strong dependence between $K_0$ and OCR. This relationship has also been demonstrated between OCR and $K_0$ from field measurements.
3. DETAILS OF EQUIPMENT AND TEST PROCEDURES

3.1 Introduction

Several types of full-displacement and self-boring probe have been used during this research with the objective of comparing stress and pore pressure measurements for:

- Probes of varying magnitudes of displacement, i.e. different degrees of disturbance.
- Probes of varying geometry, i.e. cylindrical or plate-like.
- Differing locations along probe length.

A summary of the general characteristics of the probes used is given in Table 3.1 which also gives details on the method of lateral stress measurement. The equipment described in this chapter has not been designed by the writer but was available for this research. In some cases minor modifications were made to improve equipment response and data quality.

As outlined in Appendix A, results of more general tests such as piezo-cone soundings (CPTU) and in situ field vane tests (FVT) can also be used as lateral stress indicators. These tests are only discussed very briefly in the next section as they now represent a general state of practice for in situ testing.

3.2 Index Parameter Test Equipment

Brief details of equipment used to define index parameters are given below. For index test equipment, no direct horizontal stress measurement is obtained, rather a parameter is recorded which can be related to in situ lateral stress or stress history.
<table>
<thead>
<tr>
<th>Probe Type</th>
<th>Dimensions of Measuring Location</th>
<th>Condition Used for Lateral Stress Measurement</th>
<th>Stress-Displacement Measurement Technique</th>
<th>Distance Behind Tip to Stress Measurement Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>UBC Lateral Stress Cone</td>
<td>15 cm$^2$ 44 mm OD</td>
<td>Negligible movement</td>
<td>Strain gauged section of underreamed friction sleeve, 20 mm long</td>
<td>16.6D</td>
</tr>
<tr>
<td>UCB Lateral Stress Cone</td>
<td>10 cm$^2$ 35.7 mm OD</td>
<td>Stiff measuring system - negligible movement</td>
<td>Lateral stress on 25 mm long friction sleeve transmitted to inner strain gauged diaphragm</td>
<td>1D and 7.5D</td>
</tr>
<tr>
<td>UBC Seismic Cone Pressuremeter</td>
<td>15 cm$^2$ 44 mm OD [1]</td>
<td>Membrane lift-off</td>
<td>Total pressure electrical transducer and 3 strain-gauged feeler arms</td>
<td>29.8D [3]</td>
</tr>
<tr>
<td>Dilatometer</td>
<td>95 mm x 14 mm 60 mm diameter membrane</td>
<td>Back extrapolation from 0.05 mm displacement</td>
<td>Pneumatic pressure transducer and single membrane displacement sensor</td>
<td>6.8t [2]</td>
</tr>
<tr>
<td>Total Stress Cell</td>
<td>100 mm x 6 mm</td>
<td>Negligible diaphragm movement</td>
<td>Pressure transmitted by oil to hydraulic pressure transducer (pressure sensitive area 185 mm x 79 mm)</td>
<td>30.8t [1]</td>
</tr>
<tr>
<td>Self-Boring Pressuremeter</td>
<td>73 mm OD L/D = 6</td>
<td>Membrane lift-off</td>
<td>Total and effective stress electrical transducers and 3 strain-gauged feeler arms</td>
<td>5.6D [3]</td>
</tr>
<tr>
<td>Self-Boring Load Cell</td>
<td>80.4 mm OD 44.4 mm diameter load cell</td>
<td>Maximum load cell deflection of 9 µm at 280 kPa</td>
<td>Two bending web (electrical) load cells</td>
<td>3.5D [4]</td>
</tr>
<tr>
<td>UBC Research Dilatometer</td>
<td>95 mm x 14 mm 60 mm diameter membrane</td>
<td>Membrane lift-off</td>
<td>Effective stress electrical transducer and strain-gauged feeler arm</td>
<td>7.14t [2]</td>
</tr>
</tbody>
</table>

[1] Using distance to centre of pressure sensitive area and t = 6 mm
[2] Ratio calculated using blade thickness of 14 mm
[3] Using distance to middle of PM section
3.2.1 Piezocone Penetrometer

Two types of basic piezocone are available at UBC; the Hogentogler equipment and UBC designed equipment, both of which comply with the ASTM CPT standard. Test procedures have been elaborated upon by Campanella and Robertson (1988) and Gillespie (1990) and will not be discussed further.

As outlined in Appendix A, pore pressures during CPTU can be measured at various locations around a cone. The following nomenclature is used in this thesis to denote the measurement location (Fig. 3.1):

\[ L = \text{distance behind cone apex} \]
\[ D = \text{diameter of cone} \]

- \( u_1 \) denotes the pore pressure measured on the face of the cone. No distinction is made between pore pressures measured at the cone apex.
or along the face since the variation is relatively unimportant (Fig. 2.9).

- $u_2$ denotes the pore pressure measured immediately behind the cone tip, usually at a distance of 5 mm behind the shoulder.
- $u_3$ denotes the pore pressure measured at the back end of the friction sleeve.

All pore pressures were measured using 5 mm high, polypropylene filters, except where stated.

3.2.2 In Situ (Field) Vane

Two types of vane equipment were used during this study:

- Nilcon vane borer installed using a portable jacking frame. Three different sized tapered vanes are available depending on soil strength.
- Geonor vane borer installed by pushing with UBC Geotechnical Research Vehicle (GRV). Different sized rectangular vanes are available with a height to diameter ratio of 2.

The undrained shear strength of the soil is obtained from the torque required to rotate the vane. Peak and remolded strengths were determined at each test depth. All test procedures employed were those stipulated in ASTM D2573/D2573M.

3.2.3 Seismic Cone Penetrometer

The UBC seismic cone penetrometer is essentially a standard cone unit which has an accelerometer located 20 cm behind the tip (Campanella et al.,
1986). The accelerometer is oriented to be sensitive to horizontally polarized waves which are generated by striking the support pads of the GRV with a hammer. Striking the opposite sides of the pad generates two reversely polarized shear waves. The shear wave arrival times can be determined using either the reverse polarity (cross over) technique or the cross-correlation procedure (Campanella, Baziw and Sully, 1985) and the shear wave velocity calculated using the pseudo interval method. Shear wave velocity determinations were usually performed at 0.5 m or 1.0 m depth intervals. By striking each side of the pad twice, eight separate velocity determinations can be obtained.

The seismic cone is usually employed as a downhole technique, that is the horizontally polarized wave travels down from the surface to the cone which is located at some known depth. The direction of particle motion (horizontal) is perpendicular to the direction of wave travel (vertical). This is referred to a VH shear wave. The existing downhole configuration was modified to permit crosshole shear wave velocities to be calculated.

To shoot crosshole, a separate horizontally-displaced source at the same depth as the receiver cone is required. A 15 cm² vane cone was designed so that both HV and HH shear waves could be generated. In addition, two receivers are necessary to permit accurate interval time measurement. Details of the vane cone are given in Fig. 3.2 and the field test configuration schematically shown in Fig. 3.3 The downhole-crosshole (DH-XH) test procedure consisted of:

- performing DH seismic cone test at 1 m depth intervals at location of source vane cone (SVC) using a 10 cm² cone with no friction reducer
- the 15 cm² vane cone (source) is then installed in this hole to a depth of 1 m, measured to the centre of the vane section.
Fig. 3.2 Details of vane cone for generating crosshole shear wave signals.

Fig. 3.3 Configuration for downhole and crosshole shear wave velocity measurements.
• two receiver cones (R1 and R2) were installed along a common line at known distances from the source hole. Downhole shear wave velocities were determined at the receiver (R2) installed using the GRV; the other receiver (R1) was installed using the UBC In Situ Group trailer and downhole velocities were not determined at this location.

• all three cones (source and two receivers) were penetrated into the subsoil at 0.5 m intervals. At each depth, the following shear wave traces were recorded:
  - downhole traces from left and right hits on front pad of truck (VH) at R2 location
  - crosshole traces from vertical up and down hits generated by striking on source cone anvil (HV). Traces recorded at both receivers (R1 and R2).
  - crosshole traces from clockwise and anticlockwise hits generated by laterally striking vane cone anvil (HH). Traces recorded at both receivers (R1 and R2).

where the first letter in the (**) indicates the direction of wave travel and the second letter indicates the direction of particle motion (H = horizontal; V = vertical).

Multiple hits for each stage were used to check repeatability. As for the downhole technique, it was possible to calculate shear wave velocities from crosshole measurements using both the crossover and cross-correlation techniques with digitally filtered signals. The horizontal spacing between the cones for the crosshole set-up was between 2 m and 3 m.

3.2.4 Laboratory Index Tests

Laboratory index tests were performed on both disturbed and undisturbed samples obtained at each of the research sites where in situ tests were
carried out. The purpose of the tests was to provide details of soil type characteristics, index properties and state. Index parameters are useful for defining variations in the soil profile and for aiding the interpretation of in situ test data. Certain indices can also be used to estimate \((K_o)_{NC}\) as discussed in Chapter 2. The following tests were performed:

- grain size determinations by dry and wet sieving
- hydrometer tests
- natural water content determinations
- determination of liquid and plastic limits
- unconfined compression tests
- standard incremental oedometer tests on both horizontally- and vertically-cut samples

The results of these tests are discussed in Chapter 4.

3.3 Full-Displacement Probes: Cones

Direct lateral stress measurements using cylindrical axisymmetric penetrometers were obtained using two pieces of equipment designed and built at UBC, namely a lateral stress piezocone and a seismic cone pressuremeter. A second lateral stress cone designed and built at the University of California at Berkeley was also used at the UBC research sites and is briefly described here.

3.3.1 Lateral Stress Cone (LSC)

3.3.1.1 UBC LSC

The lateral stress (LS) piezocone designed and built at UBC comprises two separated measurement systems; a standard UBC piezocone unit followed by
a lateral stress module (Campanella, Sully, Greig and Jolly, 1990). The 8 channel cone has a tip area of 15 cm², a friction sleeve area of 225 cm² and allows the simultaneous measurement of the following parameters:
- cone resistance, \( q_c \) (bar)
- pore pressure on the face, \( u_1 \), or behind the tip, \( u_2 \) (m of water)
- sleeve friction, \( f_s \) (bar)
- pore pressure behind the friction sleeve, \( u_3 \) (m of water)
- temperature (°C)

The above channels operate over a 7.5 V range. The calibration factors for each channel are given in Table 3.2. The lateral stress module, which essentially consists of an instrumented friction sleeve, is located 0.69 m behind the tip shoulder and permits the following values to be recorded:
- sleeve friction, \( LS-FS \) (bar)
- pore pressure, \( u_{LS} \) (m of water)
- lateral stress, \( \sigma_{LS} \) (kPa)
- temperature (°C)

Table 3.2 Calibration Data for UBC LS Piezocone.

<table>
<thead>
<tr>
<th>Channel No.</th>
<th>Parameter</th>
<th>Units</th>
<th>Calibration Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cone resistance</td>
<td>bar</td>
<td>0.13 bar/mV</td>
</tr>
<tr>
<td>2</td>
<td>Sleeve friction</td>
<td>bar</td>
<td>0.013 bar/mV</td>
</tr>
<tr>
<td>3</td>
<td>Lower pore pressure</td>
<td>m of water</td>
<td>0.23 kPa/mV</td>
</tr>
<tr>
<td>4</td>
<td>Upper pore pressure</td>
<td>m of water</td>
<td>0.23 kPa/mV</td>
</tr>
<tr>
<td>5</td>
<td>Temperature</td>
<td>°C</td>
<td>*</td>
</tr>
<tr>
<td>6</td>
<td>( LS-FS )</td>
<td>bar</td>
<td>0.013 bar/mV</td>
</tr>
<tr>
<td>7</td>
<td>( LS-PP )</td>
<td>m of water</td>
<td>0.23 kPa/mV</td>
</tr>
<tr>
<td>8</td>
<td>( \sigma_{LS} )</td>
<td>kPa</td>
<td>1.44 kPa/mV</td>
</tr>
</tbody>
</table>

*Temperature in degrees celsius is obtained from \((V_T\times4)-11\) where \( V_T \) is the voltage measured from a resistance temperature device (RTD).
Even though two temperature sensors are located in the LSC, only the temperature at the lateral stress module position is recorded when the cone is being used in this format.

The transducer ranges are again 7.5 V for all the channels except the lateral stress channel which operates on 1 V full scale. The choice of location for the lateral stress module requires some comment.

Design Considerations

Previous studies into soil behaviour have demonstrated that large gradients of both stress and pore pressure exist around a penetrating cone and that these gradients are related primarily to the geometry of the equipment. In effect, the singularity at the base of the cone tip causes a large normal stress reduction to occur as the soil passes the shoulder. The extent of the reduction has been experimentally evaluated with respect to pore pressures but little information exists with respect to lateral stress reduction and the relative importance of stress redistribution and creep. For sands, indirect evidence based on the variation of average sleeve friction, $f_s$, with distance suggests that at approximately $12D$ ($D =$ diameter of cone) behind the tip, the lateral stress should be essentially constant for any particular relative density.

Location of the lateral stress sensor close to the tip would require measurements in an area of highly variable stress. Furthermore, at this location dimensional tolerances may have unacceptable effects on the measured values, i.e., a slightly undersized friction sleeve will promote a larger stress reduction whereas an oversized sleeve will reduce the unloading effect. Strain rate changes near the tip and rotation of principal stresses may also be important. This aside, both Huntsman (1985) and Jefferies et al.
(1987) present data where the lateral stress measured during cone penetration by a sensor located 1D behind the tip correspond remarkably well to results of self-boring pressuremeter tests. This is surprising considering the disturbance caused by insertion of the cone and may well result from the loose nature of the soils tested.

The location of the sensor close to the tip is advantageous where reference tests are performed in calibration chambers. Calibration of an upper stress sleeve is not possible due to the limited penetration distance resulting from chamber size. No CC facility exists at UBC and consequently it was planned to calibrate the lateral stress cone initially in the laboratory and then in the field at sites where the $K_o$ condition was known. As such, the geometry of the cone in terms of sensor location was not a restriction.

Finally, with a view to developing some kind of theoretical interpretation, it is reasonable to expect that data obtained away from the tip may more closely represent conditions of cylindrical cavity expansion (Appendix B). Stress changes near the tip may cause significant deviation from the cavity expansion condition.

Details of LS Module

For the UBC lateral stress piezocone the sensor is located 0.69 m (15.6 D) behind the cone shoulder (Fig. 3.4). The lateral stress sleeve is 88 mm long and 44 mm in diameter (surface area of 121.6 cm$^2$), with a wall thickness of 3 mm. At the centre of the sleeve, a 20 mm long section has a reduced wall thickness of 1 mm. An arrangement of strain gauges is oriented at this location to measure the hoop stress in the section induced by the lateral
Fig. 3.4 Details of UBC lateral stress cone (Campanella, Sully, Greig and Jolly, 1990).
stress acting on the sleeve. Several different gauge arrangements were tested to optimize the lateral stress response and minimize both temperature and friction cross talk effects.

A full bridge configuration is mounted on the sleeve. Each arm of the bridge consists of two 1000 ohm strain gauges. The active arms are located on the thin walled section of the sleeve whereas the inactive arms are on the thicker section. The current design remains temperature sensitive to some degree and consequently a platinum RTD sensor has been installed in the sleeve to allow for temperature compensation corrections to both the lateral stress and sleeve friction measurements.

The differential signals from the lateral stress gauges are amplified in the cone to give a full scale output of 1 volt for an external hydrostatic pressure of approximately 1440 kPa. The analog signals are converted at the surface to a 12 bit representation of their voltage giving a sensitivity of 4.9 mV or 7.4 kPa of lateral stress. The IBM PC based data acquisition system (UBC DAS) consists of an analog to digital (A/D) converter, depth controller board, counter timer board and a battery backed-up power supply (Greig et al., 1987). A schematic layout of the UBC DAS is shown in Fig. 3.5.

The data acquisition program interfaces the various components of the system to provide a means of collecting and storing the data. Data storage is either on floppy or hard disk. The program operates in two modes: cone penetration and dissipation. The change to dissipation mode is automatic when penetration is halted.

Laboratory Calibration of LSC

Laboratory calibration of the load cells and pore pressure transducers for the piezocone unit were performed according to standard procedures
Figure 3.5 Details of UBC data acquisition system (modified after Greig et al., 1987)
adopted at UBC (Gillespie, 1990). Only the laboratory calibration of the lateral stress module is considered here.

Due to the nature of the design of the LS module it was necessary to calibrate the module for the following conditions:

- hydrostatically applied confining pressure
- lateral stress cross talk on friction sleeve due to axial loads.
- temperature sensitivity
- time-dependent stability of all channels.

During each of the calibrations performed, all eight cone channels were monitored to ensure the absence of channel interference.

Hydrostatic Calibration of LS Module

To calibrate the bridge output for applied hydrostatic pressure a special sleeve was fitted over the LS module and connected to a dead weight pressure tester. Pressure increments of 20 psi (\(\sim 138 \text{ kPa}\)) were used up to a maximum of 250 psi (1724 kPa) maintaining a constant temperature throughout. Hydrostatic loading and unloading sequences were performed for conditions of zero axial load. The results are shown in Fig. 3.6 which give a calibration factor of 0.000695 V/kPa or 1440 kPa/V, with little or no hysteresis effects and no baseline drift over full scale cycling. The factor was independent of temperature for the range of application (6-20°C).

Friction-Lateral Stress Cross Talk

Axial loading of the friction sleeve causes an output voltage on the lateral stress channel due to the Poisson effect. For the strain gauge arrangement employed, increasing sleeve friction on the LS sleeve causes a negative offset on the lateral stress channel.
The cone was set up in a frame so that the axially applied load was transferred by split rings to the LS friction sleeve. Data for both the lateral stress and sleeve friction channels were recorded by means of an HP7090A measurement plotting system. The load-unload was performed under zero confining pressure over a period of approximately 1 minute with readings taken every 0.1 sec. Linear regression of the data gave gradients of -0.2136 and -0.2150 for loading and unloading (Fig. 3.7). A correction factor of 0.53 is applied to the slope in Fig. 3.7 which takes into account the difference between the axial load distribution imposed for the laboratory calibration and actual field conditions.

An average value was used to correct the lateral stress data according to the equation:

\[ s = 0.000695 \text{ V/kPa} \]
Fig. 3.7 Evaluation of cross talk on LS channel due to axial friction load (Campanella, Sully, Greig and Jolly, 1990).

\[
(V_{LS})_C = (V_{LS})_M + [0.1135 \times V_{fs}]
\]  

(3.1)

where:

\( (V_{LS})_C = \) corrected relative lateral stress voltage

\( (V_{LS})_M = \) measured relative lateral stress voltage

\( V_{fs} = \) relative sleeve friction voltage

Calibration for Temperature Effects

To evaluate the temperature sensitivity of the LS module, the whole cone was immersed in a bath of ice water and was left to warm to room temperature.
over a 24 hour period during which time readings on all channels were taken every minute. The results for the lateral stress channel are shown in Fig. 3.8. The temperature coefficient \((B_T)^{LS}\) for the LS channel was calculated to be +3.6 mV/°C on cooling. (Similarly, temperature coefficients were also evaluated for the other channels.).

**Evaluation of Baseline Drift**

During the latter part of the temperature calibration, when the system had arrived at an equilibrium condition with the ambient temperature,
continued monitoring allowed baseline drift on each channel to be evaluated. For all eight channels of the piezocone and LS module, the time dependent drift (measured over a 16 hour period) was found to be negligible.

The calibration factors obtained as outlined above have been incorporated into the data acquisition program so that the output is given in corrected engineering units. The uncorrected raw data can be accessed if required.

Field Testing Procedures

Prior to performing the LS-CPTU, all pore pressure measuring systems were de-aired and saturated with glycerin. Saturation techniques used at UBC have been discussed extensively by Gillespie (1990) and will not be considered here. The LS piezocone was placed in a cold water bath to bring the probe temperature to an estimated equilibrium ground temperature. All connections to the data acquisition system (DAS) were made and the cone momentarily suspended just above the ground (zero load on all channels). Baseline voltage readings were taken on all channels. Having completed the baseline procedure, the cone was then pushed to a depth of 2 m (or just below the water table) and penetration halted while the cone came into temperature equilibrium with the ground. After a wait of approximately 15 minutes, the sounding was commenced. All tests were performed in accordance with the International Reference Procedure outlined in ISOPT-1.

A second set of baseline voltages on all 8 channels was also recorded on completion of the sounding. The two sets of baseline provided a check on the temperature corrections applied in the data reduction software.
3.3.1.2 Berkeley (UCB) LSC

The UBC LSC described above was developed following the original Berkeley design for the model II lateral stress sensing cone penetrometer, i.e. the measurement of hoop strain in a circular underreamed section on the friction sleeve. The Berkeley LSC (Model II) is described in Appendix A (Fig. A.17). A subsequent version (Model III) was developed by Tseng (1989) and employed by Masood (1990) for evaluating in-ground lateral stresses (Appendix A, Fig. A.19). The Berkeley (UCB) LSC was used at McDonald Farm as part of a cooperative research program between the Universities of British Columbia and California (at Berkeley).

The UCB Model III LSC measures the lateral stress in a different manner than the earlier Model II probe. The lateral stress section is 25 mm long and consists of a two-ring arrangement: an outer active ring and an inner passive ring, both being fabricated with stainless steel. Four identical arciform steel pieces 1.3 mm thick are joined by a polyurethane compound to form the outer flexible ring. The flexible ring is formed over the rigid inner passive ring which contains a strain gauged stainless steel diaphragm. The 6.3 mm diameter thin-walled diaphragm performs as a pressure transducer. A sealed rubber membrane isolates the inner and outer rings. The cavity between the membrane and the inner ring is filled with de-aired water. The saturation of the pressure cavity is vital to the performance of the measuring system. A schematic illustration of the lateral stress measuring system is shown in Fig. 3.9.

Two lateral stress measurement sections are incorporated into the UCB cone; one located 1D and the other 7.5D behind the cone tip. The DAS for the cone was also developed at UCB and essentially consists of an AT compatible
microcomputer interfaced with the downhole cone electronics. The following channels are recorded during a sounding:

- tip resistance (MPa)
- sleeve friction (kPa)
- lower lateral stress at 1D behind tip (kPa)
- upper lateral stress at 7.5D behind tip (kPa)
- lower pore pressure (kPa)
- upper pore pressure (kPa)
- temperature (°C)
All corrections to the basic field data are performed by the DAS and the output is given in engineering units.

3.3.2 Seismic Cone Pressuremeter (SCPM)

The seismic cone pressuremeter (SCPM) developed at UBC has been used at several of the Lower Mainland research sites considered during this study. A description of the equipment and test procedures followed with the SCPM have been reported by Hers (1989) and Howie (1991). A review of full displacement pressuremeters is given in Appendix A. Schematic details of the probe are shown in Fig. A.24.

The pressuremeter section is mounted behind a 15 cm² piezocone unit. The exact diameter of the PM unit was measured to average 43.6 mm. When compared to the cone diameter of 44 mm, this would suggest that the PM section is slightly undersized. This has important consequences in relation to the lift-off stresses measured with the probe. This is discussed later in the thesis.

SCPM tests were performed at 1 m intervals at several of the research sites. Details of the test procedures and data interpretation have been discussed by Hers (1989) and Howie (1991). For this study, the pressuremeter lift-off and limit pressures were of primary interest.

3.4 Full-Displacement Probes: Plates
3.4.1 Dilatometer (DMT)

Details of the dilatometer equipment and test procedures are given in Appendix A. Data reduction and definition of DMT index parameters are also outlined. The DMT tests performed were in accordance with the suggested ASTM procedure (Schmertmann, 1986). The data acquisition system used at UBC
differs slightly from that normally supplied with the DMT. The standard dilatometer control/readout box incorporates a pneumatic 0-40 bar pressure gauge. A second low range gauge (0-5 bar) can also be included to measure low lift-off pressures and to accurately monitor pore pressure changes via the closure reading. In the modified UBC system the pneumatic gauges have been replaced by a CEC flush-mounted electronic transducer with a full scale range of 0-200 bar. For low pressures (< 20 bar) the gain on the amplifiers can be switched to provide better signal definition. The sampling rate for the circuitry was initially 2 Hz, but this has been increased to 4 Hz. Rates of pressure increase during a test have to be controlled so that the sampling rate accurately captures the A, B and C readings. This is also true for the calibration constants ΔA and ΔB. Using the new system, test data have been found to be very repeatable. The electronic system also includes a pressure-value hold facility at each of the standard displacements during expansion/contraction, i.e., 0.05 mm, 1.1 mm and closure. In this way, true readings at each particular displacement are obtained, rather than estimating the value from a moving gauge point at each sounding of the buzzer.

3.4.2 Total Stress Cell (TSC)

Details of TSC

The spade-shaped push-in total pressure cells (TSC) used for this research were purchased from Solinst Canada Ltd. The spade cell is a plate 6.4 mm thick with a pressure cell of dimensions 100 mm x 200 mm. The rectangular oil-filled chamber is formed of two thin steel sheets welded at the edges. The pressure sensitive area is welded to a support plate. The cavity so formed is pressurized to maintain plate separation. The welded plates are strengthened by a solid metal strip which is welded on the cell
perimeter. The oil pressure in the chamber is connected via a short length of steel tube to a pneumatic transducer located on a connector boss behind the support plate (Fig. 3.10). A ceramic porous disc is also located on the support plate and connected hydraulically to a second pneumatic transducer which is tandem mounted behind the first. Both transducers are protected within a steel sleeve adaptor which connects the spade cell to the installation rod.

![Diagram of Solinst total pressure cell components](image)

**Fig. 3.10** Components of Solinst total pressure cell (after Soil Instruments Ltd., 1987).

A pre-set baseline (zero value) and calibration is supplied for each cell by the manufacturer. The zero reading corresponds to the oil pressure in the chamber. The manufacturers recommend an initial storage life to check that no baseline changes occur.

Twin nylon tubes, sheathed in polythene, are attached to the compression fittings located on each of the pneumatic transducers. Quick release coup-
lings are attached to one of the nylon tubes at the other end of the twin tubing. The quick release couplings are used to connect the down pressure-line to the pressure readout box. The twin tubing lines are usually cut at lengths determined by the depth to which the spade cell is to be installed.

The cell and pore pressure measurements are taken using a portable pneumatic readout box. The readout unit contains a compressed nitrogen pressure bottle. With the quick release coupling connected to the readout box, the pressure valve is opened and gradually increasing pressure is applied to the transducer. When the applied pressure just exceeds the pressure in the cell, the diaphragm in the transducer deflects and vents the applied pressure to the return line (the second nylon tube). The readout box then measures the gas pressure required to just maintain a continuous flow through the diaphragm chamber. The same technique is used for reading both the oil chamber and pore pressure transducers. The pressures are measured at the surface by a Druck electronic transducer with a 0 to 2000 kPa range. Resolution of the transducer is ±0.05% full scale, i.e. ± 1 kPa.

Prior to installation in the field, minor modifications were made to the cells and calibration checks made.

Because the cells are oil-filled and sealed, the differing temperature characteristics of the cell components will cause the baseline to be sensitive to variations in temperature. This is recognized by the manufacturer but no data have been presented to evaluate the effects. Furthermore, for none of the case studies reported in the literature are the pressure cell data corrected for temperature effects. To provide data on the in-ground ambient temperature and its variation during the period when the cells are installed, platinum RTD temperature sensors were installed in several of the cells. The RTD sensors were installed adjacent to the compression fittings
on the connector boss (Fig. 3.10). The electrical cables from the sensor were taken up through the return pressure line attached to the pressure cell transducer. The presence of the thin wires did not restrict the venting action required for diaphragm movement during readout.

**Pressure and Temperature Calibration**

Temperature and hydrostatic pressure calibrations were performed in the laboratory prior to field installation. For this purpose a testing chamber was constructed. Each cell was placed in the chamber with a RTD temperature sensor attached to the midpoint of the pressure sensitive cell area. The chamber was then water filled and sealed. An external pressure source was used to vary the chamber confining pressure. Temperature variations were achieved by immersing the complete chamber in a temperature bath. The stabilized temperature for each set of pressure calibrations was measured by the RTD sensor on the face of the blade. A temperature range of 0–20°C was used for both cooling and warming cycles. Typically, a series of cell and porewater pressures were taken at nominal chamber (confining) pressures of 0, 50, 100, 150 and 200 kPa for loading and unloading cycles. The results of the pressure and temperature calibration for one of the purchased spade cells are shown in Fig. 3.11.

From the results of the calibration it is evident that:

- the total stress cells have an internal pressure at zero applied stress which has to be subtracted from the actual reading to give the stress increase resulting from the increase in external pressure.
- an offset in the internal cell zero pressure occurs (baseline drift) as the temperature of the blade changes. This concurs with results presented for other types of pressure cell (Felio and Bauer, 1986).
Fig. 3.11 Typical temperature and pressure calibration for total stress cell (Sully and Campanella, 1989).

- the offset resulting from temperature change is essentially independent of the applied stress. This facilitates easy field correction for temperature effects since the correction does not vary with the recorded in-ground stress.

The temperature drift for all the blades purchased is shown in Fig. 3.12 for the condition of zero applied chamber pressure. The temperature coefficient, $B_T$, for the cells is listed in Table 3.3. Temperature coefficients of up to 1.35 kPa/°C were measured although average values are around 0.5 kPa/°C.
Temperature dependence of measured total blade stress for zero applied cell pressure

Fig. 3.12 Initial temperature dependence of baseline pressure for all spade cells used in this study (Sully and Campanella, 1989).

Table 3.3 Initial Calibration Data for Spade Cells (Before Installation)

<table>
<thead>
<tr>
<th>Spade Cell Number</th>
<th>Reference Temperature (°C)</th>
<th>Baseline Pressure, $p_b$ (kPa)</th>
<th>Factor, $B_T$ (kPa/°C on cooling)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSC 1350</td>
<td>2.2</td>
<td>130</td>
<td>0.45</td>
</tr>
<tr>
<td>TSC 1537</td>
<td>9.5</td>
<td>156</td>
<td>1.35</td>
</tr>
<tr>
<td>TSC 1538</td>
<td>8.0</td>
<td>179</td>
<td>0.58</td>
</tr>
<tr>
<td>TSC 1539</td>
<td>10.2</td>
<td>138</td>
<td>0.14</td>
</tr>
<tr>
<td>TSC 1540</td>
<td>9.3</td>
<td>146</td>
<td>0.14</td>
</tr>
<tr>
<td>TSC 1541</td>
<td>10.5</td>
<td>135</td>
<td>0.48</td>
</tr>
<tr>
<td>TSC 1542</td>
<td>9.1</td>
<td>139</td>
<td>0.91</td>
</tr>
<tr>
<td>TSC 1580</td>
<td>9.2</td>
<td>232</td>
<td>0.67</td>
</tr>
<tr>
<td>TSC 1581</td>
<td>9.9</td>
<td>160</td>
<td>0.95</td>
</tr>
</tbody>
</table>
Since temperature changes of 10°C or more may occur between the laboratory and field environments, the temperature corrections become appreciable, especially where low stresses are being measured.

Calibration measurements were carried out on the pressure cells before installation and again after the cells had been recovered from the ground. The latter calibration was used for data interpretation. The baseline pressure for the individual TSC's given in Table 3.3 is governed by the arbitrary choice of the reference temperature. All temperature corrections to the measured blade pressures were made with respect to the equilibrium ground temperature, as measured by the RTD temperature sensor installed on the cell. A sufficient number of cells were instrumented so that a representative temperature profile could be obtained at each site (usually from 3 or 4 cells). At depths where blades were not instrumented for temperature, the ground temperature was estimated by interpolation from other temperature measurements. Thus, the measured blade pressures from in situ measurements can be corrected according to:

\[
\sigma_{TSC} = \sigma_m - \sigma_b - [(T_R - T_I)B_T]
\]

where:

- \(\sigma_{TSC}\) = temperature corrected net total blade pressure (kPa)
- \(\sigma_m\) = measured total blade pressure (kPa)
- \(\sigma_b\) = baseline total pressure at reference temperature (kPa)
- \(T_R\) = reference temperature (°C)
- \(T_I\) = in-ground temperature (°C)
- \(B_T\) = cell pressure calibration factor for temperature (kPa/°C)
Similar baseline readings were also determined for the pneumatic pore pressure transducers; these transducers were not found to be temperature sensitive.

Installation Procedure

Total stress cells of the push-in type are normally installed in the base of an existing borehole; this reduces the risk of damaging the cell. Due to costs involved in the boring operation, and the availability of an alternative technique, minor modifications were made to the spade cells to facilitate installation using the UBC Geotechnical Research Vehicle. This involved machining of a steel sleeve adaptor to connect the spade cell to the installation rods. The adaptor also serves as a protective housing for the pneumatic transducers. One end of the adapter was screwed onto the cell connector boss (Fig. 3.10) while the other accepted the AWL casing (44.7 mm OD, 4.6 mm wall thickness) that was used to push the spade cell into the ground. High buckling strength rods were required to avoid rod damage due to the high loads required to push the cell assembly - most of the resistance resulting from the larger diameter sleeve adaptor. Unlike the borehole problem, where the spade cell is only advanced 0.5 to 1.0 m below the base, the use of the UBC GRV required the cells to be pushed from ground level to their final intended depth. To avoid buckling and breakage of the rods, it was decided to use AWL casing for installation. The 35 mm rod ID also permitted easy passage of the two lines of twin tubing from the cell to the surface.

The TSC blade is most susceptible to breakage, under axial loading, where the twin plates are welded to the support plate. To reduce the axial loads on the pressure cell, it was decided to prepush a dummy plate to a
final depth 0.5 to 1.0 m above the required depth prior to installing the TSC. In this way the TSC was only pushed in virgin soil for a depth of 0.5 to 1.0 m. The dummy push was performed using the dilatometer and standard DMT readings were taken every 0.2 m (Thrust, $p_0$, $p_1$, $p_2$). After installation of the TSC the lateral stress and pore pressure were monitored with time until a stable final equilibrium value was obtained.

3.5 Self-Boring Probes

Two types of self-boring probes were used during this study, namely a pressuremeter (SBPM) and a load cell (SBLC). While the intention of the thesis is to evaluate alternative methods for evaluating $\sigma_h$ in situ, it was felt that some comparison with SBPM data would be desirable at some of the sites studied.

Previous studies performed at UBC using SBPM data have been performed in association with Dr. J.M.O. Hughes using his probe. Recently a UBC SBPM has been designed (Campanella et al., 1990) and was used during this study. Tests with the self-boring load cell were conducted in conjunction with Dr. A.B. Huang (Clarkson University, U.S.A.) using a Cambridge in situ probe (Camkometer) operated by him. The two types of probes are briefly described below. Further details of the general methods and interpretation techniques are given in Appendix A.

3.5.1 Self-Boring Pressuremeter (SBPM)

Details of SBPM Design

The UBC SBPM design is based on the equipment developed by Hughes (1973) (Fig. A.1) with improvements in the areas of instrumentation/data processing, membrane and lantern characteristics and installation techniques (Campanella
et al., 1990). The overall length of the probe is 1.43 m with an external diameter of 73 mm. The monocell probe has an expandable membrane L/D ratio of 6. Three strain-gauged cantilever feeler arms track membrane movement at the centre of the PM section during inflation. Air pressure for probe expansion is supplied by a small compressor located at ground surface. The SBPM is installed using the UBC GRV by a combination of pushing and self-boring (jetting with drilling mud). The instrumentation in the SBPM system consists of 5 transducers, downhole electronics with A/D converter and microcontroller, a 12V DC power supply and a portable personal computer.

The transducers in the probe comprise 3 cantilever-type strain arm sensors and two pressure sensors. Each transducer has a separate amplifier which permits full use of the A/D converter for each channel. The strain arms monitor the membrane displacement using a full-bridge gauge arrangement to measure arm bending. The three strain arms are mounted at 120° at the centre of the inflatable membrane. The effective operating range of each arm is 8 mm which corresponds to a radial cavity strain of about 22%. One of the pressure transducers is used to monitor the air pressure inside the probe (inflation pressure) while the other measures the differential pressure between the internal air pressure and the external pore pressure. This differential corresponds to the effective pressure exerted by the soil. The difference between the two pressure measurements gives the porewater pressure. The effective stress transducer is mounted on the SBPM membrane and moves with it during expansion. A small porous filter mounted in the transducer cavity keeps soil away from the diaphragm but permits the transmission of pore pressures. The wires from the transducers are passed to the surface inside the 6.4 mm OD air return line. For the pressure sensors and displacement transducers, 12 and 13 bit resolution are used, respect-
ively. This gives a resolution of 4 microns for the strain arms and 2 kPa for the pressure sensors. The signals are converted to ASCII format and then sent to the surface PC via an RS232 serial link. The data acquisition program converts the raw data to engineering units and provides a real-time data listing for all channels and also a graphical representation in terms of the pressure-displacement response for either any one or all of the three strain arms.

Membrane and Lantern Characteristics

Commercially available Gooch rubber tubing, 58 mm diameter and 1 mm in thickness is used for the membrane material. This was preferred to thicker more robust membranes which usually have higher membrane correction and compliance factors, often being nonlinear, and which sometimes show marked hysteretic behaviour. Expansion of the probe in air with a Gooch membrane gave a bilinear envelope. Yield occurs at about 2% radial strain at a pressure of 17 kPa. Expanding from 2% to 22% radial strain the pressure remains essentially constant with little or no hysteresis. The low value of membrane correction, i.e. 17 kPa, is ideal for testing in soft soil.

Problems were experienced with the initial lantern design which consisted of a series of 24 overlapping strips 16 mm wide and 540 mm long. The strips were held in place by rivetting to a ring at each end of the lantern. Frequent blow-out of the membrane occurred at pressures ranging from 400 kPa to 600 kPa. A modified design, using overlapping spot welded 16 mm wide strips prevented any further blow-outs. Details of the original and revised lantern designs are shown in Fig. 3.13.
Details of Jetting System

Self-boring pressuremeters are frequently installed by means of a cutting technique with drilling mud being flushed through the system to remove the broken-up soil. Hughes et al. (1984) suggested the use of jetting in cohesionless soils and low-strength clays and this system has been adopted at UBC. Two types of jetting arrangement were used for the field studies; a central jetting system as shown in Fig. 3.14(a) and a "showerhead" system as in Fig. 3.14(b). The advantage of the central rod system is that the position of the exit points (jets) can be adjusted relative to the cutting shoe edge, although the adjustment can only be made while the probe is above ground. The jet holes are 2.4 mm in diameter with varying orientations (see
(a) Central jetting rod arrangement

(b) Showerhead arrangement

Fig. 3.14 Types of jetting arrangement used with UBC SBPM (Campanella et al., 1990).
Fig. 3.14). For the showerhead setup the jetting holes are 40 mm from the cutting shoe which can cause problems in stiff/dense soils since the jets only start to break up the material after the soil is penetrated by this distance. The internal spaces for both jetting arrangements limits the maximum particle size that will pass up the probe to about 10 mm.

Calibration of Transducers

The strain arms were calibrated using a micrometer to provide a relation between measured displacement and strain arm output voltage. With the membrane removed the strain arms are at their maximum deflection. By pushing back the arms to the zero strain position and mounting a micrometer screw over each arm in turn, the required calibration was performed. The strain arm calibration was found to be linear with very little hysteresis. The calibration factors obtained in this way were introduced into the data acquisition system. Details of the individual arm calibrations are given in Fig. 3.15.

The pressure transducers were calibrated by placing the probe inside a calibration chamber and applying known increments of pressure. This was performed in two ways:

- inflation of the probe in the empty chamber
- inflation of the probe in a water-filled chamber, where the water pressure could also be varied.

All pressure calibrations were performed using a dead weight tester. The pressure transducer calibration data are shown in Fig. 3.15 and the factors so-determined were also incorporated into the data acquisition program.
Fig. 3.15 Calibration data for strain arms and pressure transducers - SBPM.
Installation Procedure

Prior to installation in the field, the transducer operation was checked and one-point calibrations performed. The filter used in the effective stress transducer was saturated under vacuum using glycerin. To obtain a regular cylindrical shape of the membrane (with the same diameter as the probe), piston ring compressors were tightened over the lantern and left in place for 24 hours prior to installation.

The SBPM is installed in a prebored hole which is opened by pushing a large diameter cone to a depth of about 2m - 3m. The probe is assembled and connected to the DAS in the field, at which time the piston ring compressors are removed and the effective stress transducer cavity saturated and the filter positioned.

With the probe suspended in the prebored hole, baseline readings are taken on all transducers. The self-boring process then begins by first circulating drilling mud until the prebored hole is filled and the fluid exits at the ground surface. A synthetic drilling mud (WDS120L) is mixed at the surface and forced down the centre of standard cone rods by a hydraulically operated rotary mud pump capable of achieving pressures up to 2500 kPa. The cables exiting the SBPM are taped to the side of the pushing rods. The drilling mud passes down the push rods and is then channelled to the jets from a central jetting rod within the PM body.

Initial field trials with the UBC SBPM were conducted with the assistance of Dr. J.M.O. Hughes. The pushing force and rate of penetration were monitored and varied continuously according to soil type. At particular depths, usually at 0.5 m intervals, penetration was halted and the PM membrane expanded and then unloaded.
3.5.2 Self-Boring Load Cell (SBLC)

The self-boring load cell used during this study was on loan from Dr. A.B. Huang (Clarkson University, U.S.A.) who was involved in the initial test series. Subsequent tests were performed by the writer. The SBLC is the original camkometer designed at Cambridge University (Hughes, 1973) later described by Dalton and Hawkins (1982). The SBLC has a passive measuring system in the sense that it does not have an expandable membrane. The probe is fitted with two flush-mounted bending web load cells (C and D) located on opposite sides of the probe near its midheight. The probe diameter is 80.4 mm and the load cells have a diameter of 44.4 mm. Deformation of the pressure cells produces an electrical output proportional to the difference between the external applied pressure (on the cell surface) and the internal gas pressure which is controlled by a surface regulator. Two pore pressure sensors (A and B) are also located at the midpoint of the probe in between the pressure cells and these are also referenced to the internal gas pressure. Details of the probe are shown in Fig. A.32 (Appendix A).

The probe is installed by self-boring using a central cutter in the shoe of the instrument and a circulating slurry which removes the soil cuttings to the surface. The same drilling mud additive (WDS120L) was used as for the SBPM.

The SBLC readings were taken in voltages and then corrected in spreadsheet files to provide data in engineering units. It was not possible to connect the probe directly to one of the UBC data acquisition systems due to fundamental differences in system designs – the SBLC was originally designed in the early 1970's. The channel ranges were 200 mV full scale for the pore pressure transducers and 10 V (amplified) for the stress cells. For calibration purposes, a specially designed sleeve was placed over the probe which
could be pressurized when connected to a pressure tester. The results given in Table 3.4 were obtained from the laboratory calibration tests. Only voltages from total stress transducers C and D were amplified.

Table 3.4 Calibration Data for SBLC

<table>
<thead>
<tr>
<th>Transducer</th>
<th>Range</th>
<th>Calibration Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pore pressure A</td>
<td>0-200 mV</td>
<td>109.4 kPa/mV</td>
</tr>
<tr>
<td>Pore pressure B</td>
<td>0-200 mV</td>
<td>118.6 kPa/mV</td>
</tr>
<tr>
<td>Total stress C</td>
<td>± 10V</td>
<td>67.59 kPa/V</td>
</tr>
<tr>
<td>Total stress D</td>
<td>± 10V</td>
<td>67.28 kPa/V</td>
</tr>
</tbody>
</table>

The installation procedure for the probe was very similar to that used for the SBPM except that a cutter head with circulating fluid was used instead of simple jetting. Tests were performed at 0.5 m depth intervals. On reaching the prescribed depth, readings on all transducers were taken until equilibrium values were obtained. This usually required a wait of about 20 minutes. The results of the tests are presented in Chapter 5.

3.6 Lateral Stress Oedometer

3.6.1 Details of LS Oedometer

The LS oedometer test is performed in exactly the same way as a standard incremental test except that the oedometer ring is now instrumented to measure hoop deformation. The central area of the ring has a reduced wall thickness (0.5 mm) which is gauged to measure hoop strains in the same way as does the lateral stress cone friction sleeve. By calibration, the hoop strain is related to a total radial stress acting on the inside of the instrumented ring. A load increment ratio of 1 was used for both loading and
unloading. Each test comprised an initial reloading phase, subsequent loading under NC conditions and then unloading. The vertical load was applied via a Bellofram air loading piston and measured using a calibrated load cell. Displacements were measured using a dial gauge indicator reading to 0.001 mm. A maximum vertical stress of 1200 kPa was used during loading which was unloaded incrementally to between 25 kPa and 50 kPa. Data for each load increment were recorded on a displacement-root time plot. Void ratios at the end of 24 hours were used to plot the e-log $\sigma'_v$ relationships.

3.6.2 Calibration of Transducers

The calibration of the vertical load cell was performed using a strain indicator box according to standard UBC practice and gave a calibration factor of 0.0327 kg/µε. The lateral stress transducer was calibrated by sealing the inside area of the ring using an upper and lower plate held together by a central rod. The central rod was fixed to the lower plate and passed through the upper plate. A screw was lightly tightened against the top plate to hold the plates against the oedometer ring. O-rings seals were located on both plates (Fig. 3.16).

A pressure line was connected to the assembled unit and 5 psi (34.47 kPa) increments were applied. The output from the LS gauges was monitored in terms of microstrain (µε). A maximum cell pressure of 75 psi (517.11 kPa) was applied. Microstrain measurements were also taken during unloading. The calibration pressures were monitored and applied using a Druck DPI 600 digital pressure indicator with 300 psi maximum, reading to 0.01 psi.

During the calibration it became apparent that the strain gauges would be sensitive to axial load (the axial load being simulated by tightening the screw nut to varying degrees). In the test situation, the axial load will
result from shear along the soil-ring interface as the sample compresses. The nonlinearity of the axial load response of the LS transducers is shown in Fig. 3.17. The calibration in terms of load was obtained by placing weights on the top of the oedometer ring and monitoring the LS gauge output. As suggested earlier, it is also possible to apply an axial load by tightening the screw nut of the calibration unit. This produces a με baseline shift on the LS transducers but no actual load estimate is possible. The effect of varying the axial load by screw tightening is illustrated in Fig. 3.18.

It appears that the effect of axial load on the LS transducer is such that it not only causes a baseline shift but also a change in calibration
factor. The baseline shifts in Fig. 3.18 of 78 µε, 400 µε, and 777 µε, arise from tightening the screw nut that holds the upper and lower calibration plates in position. It appears that a change in the calibration factor only occurs once a baseline shift of around 400 µε occurs. This effect will have to be considered when interpreting the results obtained. Details of the tests performed and final results are given in Chapter 5.
Fig. 3.18 Effect of axial load on calibration characteristics of LS transducers.
4. GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS OF RESEARCH SITES

4.1 Introduction

The in situ test equipment described in Chapter 3 has been utilized at several research sites in the Lower Mainland of British Columbia where the soils are suitable for in situ testing. The five main research sites listed in Table 4.1 provided test data in soils ranging from soft NC clays to sands and stiff OC clays and silts. Additional data from other UBC research sites have been used to supplement the main data.

The location of all the UBC research sites is shown on Fig. 4.1. A description of the geological and deposition environment for each research site is as follows:

Table 4.1 Research Sites for In Situ Measurement of Lateral Stress

<table>
<thead>
<tr>
<th>Site</th>
<th>Abbreviation Used in Text</th>
<th>Location</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>McDonald Farm</td>
<td>MDF</td>
<td>Sea Island, Richmond</td>
<td>Sand &amp; soft clay silt</td>
</tr>
<tr>
<td>Laing Bridge South</td>
<td>LBS</td>
<td>Sea Island, Richmond</td>
<td>Sand &amp; soft clay silt</td>
</tr>
<tr>
<td>Lower 232nd Street</td>
<td>Lr. 232 St.</td>
<td>Langley</td>
<td>Soft to firm NC &amp; OC clayey silt (overconsolidation principally due to dessication)</td>
</tr>
<tr>
<td>Strong Pit</td>
<td>STR</td>
<td>Aldergrove</td>
<td>Firm to stiff OC clayey silt (overconsolidation due to unloading)</td>
</tr>
<tr>
<td>200th Street - #88th Ave.</td>
<td>200th St.</td>
<td>Langley</td>
<td>Soft to stiff NC &amp; OC clayey silt (overconsolidation due to minor unloading and dessication)</td>
</tr>
</tbody>
</table>
Figure 4.1 Location of UBC geotechnical research sites.
area is given below followed by detailed site information in terms of soils present and basic geotechnical parameters. The testing programme performed at each site is also described.

4.2 Geological History of the Lower Mainland

The geological history of the Lower Mainland has been studied by Blunden (1973) and Clague and Luternauer (1982). The Fraser River essentially controls the deposition environments throughout the region. The surficial deposits of the lowland are of Quaternary age and attain thicknesses of up to 300 m, overlying Pleistocene glacial deposits and Tertiary freshwater sediments. The Quaternary soils were deposited during periods of the last glaciation being influenced also by the contemporaneous isostatic and eustatic fluctuations. As a consequence of the complex deposition environment, the sediment types range from glacial to deltaic and demonstrate considerable heterogeneity both laterally and vertically.

The development of the Fraser River delta began about 11,000 years ago once the ice had retreated from the area. At this time the present location of Richmond was some 40 km out to sea and subject to deposition of fine sediments discharged into the sea by the Fraser River. Due to the large volume of sediments supplied to the river by the retreating ice sheet, the delta expanded rapidly and attained its almost present position about 5,000 years ago. Continued development was slowed by an 11 m rise in sea level. The Richmond area was now only 10 km out to sea and slightly coarser sediments were being deposited. Between 5,000 years ago and the present the delta grew to its present position as the sea rose a further metre. As the delta front approached Richmond sands were deposited over the finer grained clays and silts. Recent silt and clay overbank deposits were deposited
during annual flooding of the river as delta growth continued. The delta is still growing at rates of between 2.5 m/yr and 8.5 m/yr (depending on the depth of water).

Being post-glacial, the Holocene soils have not been ice-loaded and are generally normally consolidated. The generalized soil profile for the lowland area consists of fine grained marine sediments overlain by granular marine, deltaic and tidal flat deposits and then by fine overbank deposits (Wallis, 1979).

To the east of the Fraser River delta is the upland area where the Langley-Cloverdale basin is located. The sediments are Pleistocene glacial and post-glacial. The earlier glacial deposits consists of dense sands and gravels. The overlying post-glacial sediments have a glaciomarine origin having been deposited during a period when the Fraser River became dammed by ice. The soft clay silts are known locally as Capilano or Fort Langley sediments. Horizons of interbedded sand are common throughout, the frequency of which dies out towards the west.

4.3 Laing Bridge South, Richmond

4.3.1 Site Description

The site is located on the east side of Sea Island (Fig. 4.1) and is reached by means of the Arthur Laing Bridge to the north. The site is bounded to the north and east by McConachie Way overpass and by Airport Way to the south and west (Fig. 4.2).

The site is approximately 340 m long and 70 m wide and reasonably level. An average slope of 0.5° dipping to the northwest was measured. Surface drainage features on the site are visible. A main drainage ditch which runs parallel to Airport Way suggested a groundwater level about 1.2 m below
ground surface. The ground is covered by low grass. The study area had previously been part of the land incorporated for the construction of McConachie Way overpass and has a regraded surface relief with possible fill placement.
Overbank deposits with some fill comprise the surficial soils to a depth of 2 m which are underlain by 18 m of fine to medium sand. At 20 m a transition to a soft normally consolidated clay silt begins. The silt attains thicknesses of 40 m to 45 m at this location (Le Clair, 1988).

4.3.2 Testing Programme

Two test areas are indicated on Fig. 4.2. The testing programme at each location is outlined in Table 4.2. The legend used to denote each particular test is indicated in Table 4.3 and the layout of the tests at each location is shown in Fig. 4.3.

Table 4.2 Testing Programme at Laing Bridge South

<table>
<thead>
<tr>
<th>Test Performed / Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Area 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hogentogler CPTU, $u_2$ to 29.7m</td>
<td>24/09/87</td>
<td>C87-LBS2</td>
<td></td>
</tr>
<tr>
<td>UBC #7, $u_1$ &amp; $u_2$ to 29.7m</td>
<td>24/09/87</td>
<td>C87-LBS3</td>
<td></td>
</tr>
<tr>
<td>Dilatometer (#89) to 21m</td>
<td>28/09/87</td>
<td>D87-LBS1</td>
<td></td>
</tr>
<tr>
<td>Dilatometer (#74) to 29.8m</td>
<td>01/10/87</td>
<td>D87-LBS2</td>
<td></td>
</tr>
<tr>
<td>Dutch sampler to 4.75m</td>
<td>08/10/87</td>
<td>S87-LBS1</td>
<td></td>
</tr>
<tr>
<td>SPT in hollow stem auger to 23.7m</td>
<td>08/10/87</td>
<td>SPT-LBS1</td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7, $u_2$ &amp; $u_3$ to 17.9m</td>
<td>15/10/87</td>
<td>C87-LBS4</td>
<td></td>
</tr>
<tr>
<td>UBC Seismic cone pressuremeter to 7.0m</td>
<td>05/11/87</td>
<td>SCPMLBS1</td>
<td></td>
</tr>
<tr>
<td>UBC Seismic cone pressuremeter to 14m</td>
<td>04/12/87</td>
<td>SCPMLBS2</td>
<td>Subsequent test by Howie (1987)</td>
</tr>
<tr>
<td>Dilatometer (#74) to 19.2m</td>
<td>31/08/87</td>
<td>D88-LBS3</td>
<td></td>
</tr>
<tr>
<td>Lateral stress cone to 23.5m</td>
<td>05/09/87</td>
<td>LBS-LBS1</td>
<td></td>
</tr>
<tr>
<td><strong>Test Area 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7, $u_1$ &amp; $u_3$ to 20m</td>
<td>19/09/88</td>
<td>C88-LBS5</td>
<td></td>
</tr>
<tr>
<td>Lateral stress cone to 25.90m</td>
<td>20/07/89</td>
<td>LS2-LBS</td>
<td>$u_2$, $u_3$, $u_{LS}$</td>
</tr>
<tr>
<td>Self-boring load cell to 6.5m</td>
<td>23/07/90</td>
<td>CAM1-LBS</td>
<td></td>
</tr>
<tr>
<td>Self-boring load cell to 5.5m</td>
<td>24/07/90</td>
<td>CAM2-LBS</td>
<td></td>
</tr>
<tr>
<td>Self-boring load cell to 8.5m</td>
<td>27/07/90</td>
<td>CAM3-LBS</td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7 to 20m</td>
<td>21/08/90</td>
<td>C90-LBS6</td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7 &amp; #8 to 10m</td>
<td>22/08/90</td>
<td>C90-LBS7</td>
<td>Downhole and crosshole</td>
</tr>
</tbody>
</table>
4.3.3 Geotechnical Characteristics

Based on the results of the initial in situ and laboratory tests performed, the index properties and basic geotechnical parameters have been defined. These are briefly presented here and will be used later for interpretation of the more sophisticated in situ tests carried out.
Fig. 4.3 Layout of tests at Laing Bridge South.
Grading analyses were performed on disturbed samples recovered during the SPT drillhole. Figure 4.4(a) presents the range of grading curves obtained from 10 tests. The finer grading curves are obtained from samples above 3.5 m (overbank deposits) and below 19.5 m (transition to clay silt). The main body of sand between 3.5 m and 19.5 m is fine to medium grained having a \(D_{50}\) size of 0.2 mm. Peak friction angles in the sand of 41° are predicted using the CPTU results. These values are in agreement with the limited laboratory test data available (Zavoral, 1988). The dilatometer suggests a significantly lower friction angle (Fig. 4.4(b)). The underlying clay silt has a plasticity index of between 6% to 10% with an average undrained strength ratio \(\left(S_u / \sigma'\right)\) of 0.30. Stress history parameters indicate the clay silt to be normally consolidated. Also shown on Fig. 4.4(c) is the variation of maximum shear modulus with depth as determined from the seismic cone downhole test.

4.4 McDonald Farm, Richmond

4.4.1 Site Description

The McDonald Farm site is located on the north side of Sea Island some 4 km from the Laing Bridge South site. The site is approximately 250 m by 50 m sensibly level and covered by light vegetation of grass and shrubs. The general site area is bounded to the north and west by dirt roads and to the south and east by drainage ditches (Fig. 4.5). The site dips at about 5° towards the drainage ditch which was partially filled with standing water. Due to the proximity of the site to the coast, the groundwater is affected by tidal fluctuations. On average the phreatic surface is about 1.5 m below ground level.
Fig. 4.4 Geotechnical data for LBS deposits
The area has been raised and regraded during excavation of the drainage ditches and fill placement. The average elevation is +1.6 metres. A grid reference system has been established across the site based on a line of spaced angle-iron reference posts (Fig. 4.5).

The soil profile is very similar to that at Laing Bridge South except that individual layer thicknesses vary. The profile consists of up to 4 m of soft compressible silts underlain by medium to coarse sand to about 14 m. The sand, which has a variable density, is interbedded with silt layers
attaining maximum thicknesses of about 1 m. The sand is underlain by a soft
normally consolidated silt.

4.4.2 Testing Programme

The series of field tests performed at the McDonald Farm site area
detailed in Table 4.4 for both the test areas indicated on Fig. 4.5. The
locations of the individual tests are indicated on Fig. 4.6.

4.4.3 Geotechnical Characteristics

Grading analyses performed on samples recovered during SPT borehole
testing indicated the sand from 4 m to 14 m to be medium to coarse grained
with low percentages of fines (Fig. 4.7(a)). Good agreement exists between
friction angles determined from laboratory triaxial tests and those inter­
preted from CPT data using the Robertson and Campanella (1983) method. G_{max}
values obtained from the seismic cone downhole test are given in Fig.
4.7(c).

The soft normally consolidated silt exhibits the following character­
istics:

10% sand; 70% silt; 20% clay

LL = 35%
\omega_n = 36%
PI = 10%
(S_u/c')_FV = 0.33
S_t(FV) = 3-5
\phi' = 35-36.5°
<table>
<thead>
<tr>
<th>Test Performed/Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test Area 1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC #7 CPTU, $u_1$ &amp; $u_3$ to 30m</td>
<td>22/09/88</td>
<td>C88-2MDF</td>
<td>Diaphragm facing east</td>
</tr>
<tr>
<td>Hogentogler CPTU, $u_2$ to 30m</td>
<td>22/09/88</td>
<td>C88-3MDF</td>
<td>Hammer and Buffalo Gun</td>
</tr>
<tr>
<td>Dilatometer (#74) to 30m</td>
<td>29/09/88</td>
<td>D88-1MDF</td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU (UBC #7) to 27.8m</td>
<td>06/10/88</td>
<td>C88-4MDF</td>
<td></td>
</tr>
<tr>
<td>UCB lateral stress cone ($u_2$ &amp; $u_{LS}$) to 30.12m</td>
<td>19/07/89</td>
<td>UCB-LS1</td>
<td>LS section at 1D and 7.5D</td>
</tr>
<tr>
<td>UCB lateral stress cone ($u_2$ &amp; $u_{LS}$) to 13.98m</td>
<td>19/07/89</td>
<td>UCB-LS2</td>
<td>LS section at 9D and 16.5D</td>
</tr>
<tr>
<td>UCB lateral stress cone to 25.75m</td>
<td>20/07/89</td>
<td>MDF-LS2</td>
<td>$u_1$, $u_3$, and $u_{LS}$</td>
</tr>
<tr>
<td>UCB Dilatometer to 13.8m</td>
<td>22/07/89</td>
<td>UCB-DMT1</td>
<td>Cone malfunctioned</td>
</tr>
<tr>
<td>UCB Dilatometer to 14.0m</td>
<td>23/07/89</td>
<td>UCB-DMT2</td>
<td>$u_1$, $u_3$, and $u_{LS}$</td>
</tr>
<tr>
<td>UBC self-boring pressuremeter to 6m</td>
<td>29/08/89</td>
<td>SBP3-MDF</td>
<td></td>
</tr>
<tr>
<td>UBC Lateral stress cone to 5m</td>
<td>13/11/89</td>
<td>MDF-LS3</td>
<td></td>
</tr>
<tr>
<td>UBC Lateral stress cone to 20.08m</td>
<td>19/07/90</td>
<td>MDF-LS4</td>
<td></td>
</tr>
<tr>
<td><strong>Test Area 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC #8, CPTU, $u_1$ &amp; $u_3$ to 28.02m</td>
<td>21/09/89</td>
<td>C89-2MDF</td>
<td>577 Course (1989)</td>
</tr>
<tr>
<td>Hogentogler CPTU, $u_2$ to 28.05m</td>
<td>21/09/89</td>
<td>C89-3MDF</td>
<td>577 Course (1989)</td>
</tr>
<tr>
<td>Dilatometer (UBC #2) to 27.0m</td>
<td>28/09/89</td>
<td>D89-1MDF</td>
<td>577 Course (1989)</td>
</tr>
<tr>
<td>Seismic CPTU (UBC #7) to 27.52m</td>
<td>05/10/89</td>
<td>C89-4MDF</td>
<td>577 Course (1989)</td>
</tr>
<tr>
<td>UBC self-boring pressuremeter to 8m</td>
<td>19/10/89</td>
<td>SBP4-MDF</td>
<td></td>
</tr>
<tr>
<td>UBC self-boring pressuremeter to 13m</td>
<td>28/11/89</td>
<td>SBP5-MDF</td>
<td></td>
</tr>
<tr>
<td>Self-boring load cell to 3.0m</td>
<td>19/07/90</td>
<td>CAM1-MDF</td>
<td>Pressuremeter lost</td>
</tr>
<tr>
<td>Self-boring load cell to 4.5m</td>
<td>20/07/90</td>
<td>CAM2-MDF</td>
<td></td>
</tr>
<tr>
<td>Self-boring load cell to 4.5m</td>
<td>21/07/90</td>
<td>CAM3-MDF</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 4.6 Layout of tests at McDonald Farm.
Fig. 4.7 Geotechnical data for McDonald Farm deposits.
4.5 Lower 232nd Street, Langley

4.5.1 Site Description

The 232nd Street site is located to the north side of the Trans-Canada Highway (Highway No. 1) near the City of Langley (Fig. 4.1). A lower and an upper site have been used at this site by the In Situ Testing Group although data reported here have been obtained from the lower site only (Fig. 4.8). The site is approximately 80 m long, 40 m wide, and triangular in shape being bounded on all three sides by public highways. The site slopes gently to the west. Drainage ditches run along the two long sides, being damp in the base but generally not containing standing water.

The stratigraphic profile at the site consists of a near-surface overconsolidated crust underlain by a low plasticity aged normally consolidated clayey silt. The glaciomarine clay is sensitive as a result of freshwater leaching after deposition. Below 5 m the clay silt is essentially normally consolidated and homogeneous. Between 13 m and 16 m occasional sand layers are present. Below 23 m the silt is interbedded with dense sand. The water table is subject to seasonal fluctuation, varying between 1 m and 1.5 m below ground level. The pore pressure distribution is essentially hydrostatic.

4.5.2 Testing Programme

A list of the field tests performed at the site is given in Table 4.5. The locations of the tests are indicated on Fig. 4.9. Also shown on Fig. 4.9 is the location of the test pit from which undisturbed block samples were recovered.

4.5.3 Geotechnical Characteristics

The results of the hydrometer and Atterberg limit tests are shown in Fig. 4.10. Below 2 m the clay silt is of low plasticity containing an
Fig. 4.8 Lower 232nd Street - general site details.
Table 4.5 Field Testing Programme at Lr. 232 St.

<table>
<thead>
<tr>
<th>Test Performed/ Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Area 1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nilcon in situ vane test to 22m</td>
<td>12/11/87</td>
<td>232-1V87</td>
<td>Peak &amp; remoulded</td>
</tr>
<tr>
<td>Hogentogler CPTU, ( u_2 ) to 29.4m</td>
<td>12/11/87</td>
<td>232-1C87</td>
<td>2.5mm filter</td>
</tr>
<tr>
<td>UBC #7 CPTU, ( u_1 ) &amp; ( u_3 ) to 28.7m</td>
<td>19/11/87</td>
<td>232-2C87</td>
<td>5mm filter</td>
</tr>
<tr>
<td>Hogentogler CPTU, ( u_2 ) to 29.7m</td>
<td>19/11/87</td>
<td>232-3C87</td>
<td>5mm filter</td>
</tr>
<tr>
<td>Dutch sampler to 15m</td>
<td>26/11/87</td>
<td>232-1S87</td>
<td></td>
</tr>
<tr>
<td>BAT pore pressure &amp; permeability</td>
<td>26/11/87</td>
<td>232-BAT4</td>
<td>Installed @ 7.32m</td>
</tr>
<tr>
<td>UBC #7 CPTU, ( u_1 ) &amp; ( u_3 ) to 12m</td>
<td>29/04/88</td>
<td>232-4C88</td>
<td>Long pore pressure dissipations ( u_3 ) not working</td>
</tr>
<tr>
<td>Lateral stress cone to 5m</td>
<td>14/01/89</td>
<td>232-LS1</td>
<td>Pore pressure transducer from 1540 with temper-</td>
</tr>
<tr>
<td>Total stress cell installed at 5.0m</td>
<td>16/02/89</td>
<td>TSC1580</td>
<td>ature sensor</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed at 7.5m</td>
<td>25/02/89</td>
<td>TSC1581</td>
<td>Temperature sensor on blade</td>
</tr>
<tr>
<td>Total stress cell installed at 12.5m</td>
<td>20/04/89</td>
<td>TSC1538</td>
<td>Temperature sensor on blade</td>
</tr>
<tr>
<td>Total stress cell installed at 10m</td>
<td>21/04/89</td>
<td>TSC1537</td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed at 2.5m</td>
<td>22/04/89</td>
<td>TSC1541</td>
<td></td>
</tr>
<tr>
<td>Lateral stress cone to 25.9m</td>
<td>04/08/89</td>
<td>232-LS2</td>
<td>( u_3 ), ( u_1 ), &amp; ( u_{LS} )</td>
</tr>
<tr>
<td>Extension of total stress cells</td>
<td>22/09/89</td>
<td>TSCs</td>
<td></td>
</tr>
<tr>
<td>1580, 1581, 1538 and 1537 by 1m;</td>
<td>23/09/89</td>
<td>TSCs</td>
<td></td>
</tr>
<tr>
<td>1541 advanced 0.5m deeper</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trial pit excavation to 3.0m</td>
<td>25/09/89</td>
<td>232-TP1</td>
<td>Recovery of block samples</td>
</tr>
<tr>
<td>Dilatometer (UBC #2) to 15m</td>
<td>03/10/89</td>
<td>232-1D89</td>
<td>Membrane facing south</td>
</tr>
<tr>
<td>Dilatometer (UBC #2) to 15m</td>
<td>04/10/89</td>
<td>232-2D89</td>
<td>Membrane facing west</td>
</tr>
<tr>
<td>Lateral stress cone to 16.33m</td>
<td>11/10/89</td>
<td>232-LS3</td>
<td>( u_3 ), ( u_1 ), &amp; ( u_{LS} )</td>
</tr>
<tr>
<td>Lateral stress cone to 16.05m</td>
<td>12/10/89</td>
<td>232-LS4</td>
<td>( u_3 ), ( u_1 ) &amp; ( u_{LS} )</td>
</tr>
<tr>
<td>Lateral stress cone to 16.08m</td>
<td>12/10/89</td>
<td>232-LS5</td>
<td>( u_3 ), ( u_1 ) &amp; ( u_{LS} )</td>
</tr>
<tr>
<td>Self-boring load cell to 17m</td>
<td>25/07/90</td>
<td>232-CAM1</td>
<td>C cell to NE</td>
</tr>
<tr>
<td>Self-boring load cell to 5.05m</td>
<td>26/07/90</td>
<td>232-CAM2</td>
<td>C cell to NE</td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7, ( u_1 ) &amp; ( u_3 ) to 11m</td>
<td>08/08/90</td>
<td>232-5C90</td>
<td></td>
</tr>
<tr>
<td>Crosshole &amp; downhole seismic to 11m</td>
<td>08/08/90</td>
<td>232-XH1</td>
<td>( u_3 ) &amp; ( u_3 )</td>
</tr>
<tr>
<td>Total stress cells removed from site in November 1989</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
TEST AREA FOR LATERAL STRESS EVALUATION

TEST PIT (Recovery of Block Samples)

Fig. 4.9 Layout of field tests at Lr. 232 St.
average 5% fine sand. The fine material comprises clay and silt in the ratio 40:55%. Natural water contents range from 33% to 42%. The range of measured plasticity limits obtained was:

\[
\begin{align*}
\text{LL} &= 40-50\% \quad \text{Avg: } 44\% \\
\text{PL} &= 17-21\% \quad \text{Avg: } 20\% \\
\text{PI} &= 21-30\% \quad \text{Avg: } 24\%
\end{align*}
\]

Above 2 m, the liquid limit is higher (73-83%) giving rise to a higher PI (46-61%) and natural water content (55-70%). The results of the in situ shear vane tests are presented in Fig. 4.11. The figure includes tests performed by the writer supplemented with data from results obtained over the last 6 years (Greig, 1985; Campanella, Sully & Robertson, 1988; Hers, 1989).
In terms of $S_u$, the effect of the near surface overconsolidated crust is not clearly apparent. However, in terms of the normalized strength ratio, $S_u / \sigma'_v$, the trend in OCR with depth is apparent. Below 14m the scatter in the results increases primarily as a result of the presence of thin sand lenses within the clay silt.

The tabulated results of standard oedometer tests performed on horizontally-cut undisturbed samples are presented in Table 4.6. Due to the orientation of the samples, the vertical preconsolidation pressure ($\sigma'_v$) is obtained from which the overconsolidation ratio (OCR) can be calculated.
Table 4.6 Oedometer Test Results for Horizontally-Cut Undisturbed Samples - Lr 232 St.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\gamma_f$ (Mg/m³)</th>
<th>$\varepsilon_0$ (in situ)</th>
<th>$\omega_m$ (%)</th>
<th>$\sigma'_v$ (kPa)</th>
<th>$\sigma'_{vm}$ (kPa)</th>
<th>OCR ($\sigma'_{vm}/\sigma'_v$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.93</td>
<td>1.69</td>
<td>1.47</td>
<td>49.9</td>
<td>15.8</td>
<td>205</td>
<td>12.9</td>
</tr>
<tr>
<td>1.5</td>
<td>1.66</td>
<td>1.590</td>
<td>75.7</td>
<td>19.7</td>
<td>61</td>
<td>3.1</td>
</tr>
<tr>
<td>1.86</td>
<td>1.63</td>
<td>1.663</td>
<td>55.8</td>
<td>21.7</td>
<td>62</td>
<td>2.9</td>
</tr>
<tr>
<td>2.0</td>
<td>1.69</td>
<td>1.790</td>
<td>74.7</td>
<td>22.5</td>
<td>131</td>
<td>5.8</td>
</tr>
<tr>
<td>2.0</td>
<td>1.59</td>
<td>0.980</td>
<td>75.2</td>
<td>23.0</td>
<td>116</td>
<td>5.0</td>
</tr>
<tr>
<td>2.4</td>
<td>1.70</td>
<td>1.424</td>
<td>53.6</td>
<td>24.9</td>
<td>90</td>
<td>3.6</td>
</tr>
<tr>
<td>3.9</td>
<td>1.90</td>
<td>1.953</td>
<td>32.4</td>
<td>32.7</td>
<td>138</td>
<td>4.2</td>
</tr>
<tr>
<td>4.2</td>
<td>1.87</td>
<td>1.005</td>
<td>37.2</td>
<td>34.6</td>
<td>176</td>
<td>5.0</td>
</tr>
<tr>
<td>5.0</td>
<td>1.58</td>
<td>1.10</td>
<td>25.0</td>
<td>38.8</td>
<td>115</td>
<td>3.0</td>
</tr>
<tr>
<td>6.1</td>
<td>1.76</td>
<td>2.28</td>
<td>48.2</td>
<td>45.1</td>
<td>63</td>
<td>1.4</td>
</tr>
<tr>
<td>7.7</td>
<td>1.78</td>
<td>1.207</td>
<td>46.7</td>
<td>54.2</td>
<td>45</td>
<td>1.0</td>
</tr>
<tr>
<td>8.0</td>
<td>1.51</td>
<td>1.320</td>
<td>33.8</td>
<td>55.7</td>
<td>90</td>
<td>1.6</td>
</tr>
<tr>
<td>9.1</td>
<td>-</td>
<td>1.180</td>
<td>49.4</td>
<td>62.5</td>
<td>72</td>
<td>1.2</td>
</tr>
<tr>
<td>11.0</td>
<td>1.83</td>
<td>2.10</td>
<td>32.5</td>
<td>73.7</td>
<td>76</td>
<td>1.0</td>
</tr>
<tr>
<td>14.7</td>
<td>1.76</td>
<td>1.233</td>
<td>46.5</td>
<td>96.9</td>
<td>100</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Several samples were also cut vertically from both the block and tube samples and tested in the oedometer to obtain the horizontal preconsolidation pressure, $\sigma'_h$ (Jefferies et al., 1987). The results are presented in Table 4.7. Two values of lateral stress coefficient can be defined from these tests, namely:

$$K_1 = \frac{\sigma'_h}{\sigma'_v}$$  \hspace{1cm} (4.1)

$$K_2 = \frac{\sigma'_h}{\sigma'_{vm}}$$  \hspace{1cm} (4.2)

The profiles of OCR obtained from laboratory oedometer and field vane tests are shown in Fig. 4.12. The results are discussed later in Chapter 5.

Shear wave velocities obtained from downhole seismic cone tests are shown on Fig. 4.13.
Table 4.7 Oedometer Test Results for Vertically-Cut Undisturbed Samples - Lr 232 St.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\gamma_t$ (Mg/m$^3$)</th>
<th>$e_o$</th>
<th>$\omega_n$ (%)</th>
<th>$\sigma'_v$ (kPa)</th>
<th>$\sigma'_h$ (kPa)</th>
<th>$K_1$ ($\sigma'_h/\sigma'_v$)</th>
<th>$K_2$ ($\sigma'_h/\sigma'_v$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.93</td>
<td>1.73</td>
<td>1.295</td>
<td>49.8</td>
<td>15.8</td>
<td>170</td>
<td>10.8</td>
<td>0.83</td>
</tr>
<tr>
<td>1.86</td>
<td>1.59</td>
<td>1.663</td>
<td>62.9</td>
<td>21.7</td>
<td>46</td>
<td>2.1</td>
<td>0.74</td>
</tr>
<tr>
<td>2.0</td>
<td>1.61</td>
<td>1.75</td>
<td>64.5</td>
<td>23</td>
<td>63</td>
<td>2.7</td>
<td>0.48</td>
</tr>
<tr>
<td>3.9</td>
<td>1.93</td>
<td>1.880</td>
<td>32.0</td>
<td>33</td>
<td>105</td>
<td>3.18</td>
<td>0.70</td>
</tr>
<tr>
<td>4.2</td>
<td>1.78</td>
<td>1.141</td>
<td>38.6</td>
<td>34.6</td>
<td>102</td>
<td>2.9</td>
<td>0.58</td>
</tr>
</tbody>
</table>

Fig. 4.12 Profile of OCR determined from laboratory oedometer and field vane tests - Lr 232 St (Campanella, Sully and Robertson, 1988).

4.6 Strong Pit, Aldergrove

4.6.1 Site Description

The site is located at the edge of the hummocky kame-and-kettle terrain near the town of Aldergrove (Fig. 4.1). The surface gravel in the area is
fluvio-glacial in origin having been transported seaward by meltwater streams during the three major glaciations which occurred in the area (Clague and Luternauer, 1982). The poorly sorted sand and pebble-cobble gravels, which irregularly overlie marine or glaciomarine clayey silt, have been assigned to the Sumas Drift, and attain thicknesses of 30 m in the area. At the location of Strong Pit the gravel is about 15 m thick but has been excavated to a depth of approximately 14 m at the location of the in situ tests.

The underlying stony silty clay could be part of the Sumas Drift but is more likely to be a deposit of the Fort Langley Formation (Pleistocene), the fines being deposited from meltwaters and seawater, with the stones and some finer material being transported by floating ice and deposited as it melted.
Sediments are dominantly massive to weakly stratified containing dropstones (Clague and Luternauer, 1982). Stones can be up to cobble size. Lenses of sand and gravel up to several metres thick may be present. Discontinuous stringers of fine sand are numerous. Shells are usually common but none have been found at this location. Sampling shows the clay to be intact with no indications of fissuring.

As mentioned previously, gravel has been removed from the area where testing has been performed. Since the deposits have not been overridden by ice, reasonable estimates of the stress history of the silty clay are possible.

The present profile consists of 1 m to 1.5 m of gravel underlain by up to 9 m of stony clayey silt. Below this level, the silt is interbedded with dense fine sands and gravel.

Groundwater conditions at the site are somewhat unusual in that the measured pore pressure throughout the upper clay silt varies between 0-10 kPa. These pressures arise due to the clay layer being underdrained; a perched water table is present in the gravel at the top surface of the clay and maintains saturation of the clay. Piezometer measurements in the underlying granular layer indicate that the pore pressure distribution becomes hydrostatic at the base of the clay which coincides with the location of the water table.

4.6.2 Testing Programme

The surface gravel and cobble at the site was so dense that penetration of any testing tool was difficult with the obvious likelihood of breakage. To facilitate testing, a series of trenches were excavated to the base of the gravel at various locations across the site. The trenches were then back-
filled with a fine loose gravel, through which most of the in situ probes could be penetrated. The location of the test trenches is indicated on Fig. 4.14. The testing programme is outlined in Table 4.8.

4.6.3 Geotechnical Characteristics

The results of the hydrometer and Atterberg limit tests are presented in Fig. 4.15. The results suggest the soil is poorly sorted but homogeneous with depth. Slight changes in composition appear to be reflected by changes in PI. The following parameter ranges were measured for the clay silt:

- 19% Clay; 52% Silt; 29% Sand
- $LL = 26-37\%$ Avg: 32\%
- $PL = 15-19\%$ Avg: 17\%
- $PI = 11-20\%$ Avg: 15\%
- $\omega_n = 15-20\%$ Avg: 17.4\%
- $\left(\frac{S_u}{\sigma_v'}\right)_{NC}^* = 0.31-0.38$ Avg: 0.35

*Calculated from $\left(\frac{S_u}{\sigma_v'}\right)_{FM}$ where $\sigma_v'$ is obtained from oedometer tests.

The undrained strength profile (Fig. 4.16) was obtained from Nilkon field vane tests. The vane tests were correlated with cone data to evaluate the $S_u$ at greater depths where vane penetration was not possible.

The results of standard incremental oedometer tests are reported in Table 4.9. A profile of shear wave velocity determined by the downhole seismic cone technique is indicated in Fig. 4.17. The small strain shear modulus, $G_o$, can be obtained from the following relationship:

$$G_o = \rho \frac{V_s^2}{s}$$ (4.3)

where $\rho$ is the unit weight of the soil.
Fig. 4.14 Strong Pit - General site details and layout of test locations.
Table 4.8 Field Testing Programme at Strong Pit

<table>
<thead>
<tr>
<th>Test Performed/Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Trench 1</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hogentogler CPTU ($u_3$) to 14.62m</td>
<td>12/11/87</td>
<td>C87-1STR</td>
<td>CIVL 445 Project</td>
</tr>
<tr>
<td>Hogentogler CPTU ($u_3$) to 6.58m</td>
<td>28/12/87</td>
<td>C87-2STR</td>
<td>BAT installed at 7.2m</td>
</tr>
<tr>
<td>UBC #7 CPTU ($u_1$ &amp; $u_3$) to 3.7m</td>
<td>28/12/87</td>
<td>C87-3STR</td>
<td>BAT installed at 4.2m</td>
</tr>
<tr>
<td>UBC #7 CPTU ($u_1$ &amp; $u_3$) to 9.6m</td>
<td>29/12/87</td>
<td>C87-4STR</td>
<td>BAT installed at 9.72m</td>
</tr>
<tr>
<td>Nilcon FVT to 4m @ 0.5m intervals</td>
<td>28/12/87</td>
<td>V87-1STR</td>
<td>11x5cm vane</td>
</tr>
<tr>
<td>Nilcon FVT to 5.5m @ 0.5m intervals</td>
<td>28/12/87</td>
<td>V87-2STR</td>
<td>11x5cm vane</td>
</tr>
<tr>
<td>*Dutch Sampler - no recovery</td>
<td>29/12/87</td>
<td>S87-1STR</td>
<td>Equipment damaged</td>
</tr>
<tr>
<td>Swedish Sampler - continuous to</td>
<td>26/01/88</td>
<td>S88-2STR</td>
<td></td>
</tr>
<tr>
<td>recovery to 8.6m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed to 8.4m</td>
<td>05/07/88</td>
<td>TSC1350</td>
<td>Vane damaged</td>
</tr>
<tr>
<td>*Geonor vane to 2.5m</td>
<td>06/07/88</td>
<td>V88-STR4</td>
<td></td>
</tr>
<tr>
<td>Hogentogler CPTU ($u_3$) to 14.42m</td>
<td>07/07/88</td>
<td>C88-5STR</td>
<td></td>
</tr>
<tr>
<td>Dilatometer to 12.4m (membrane fac-</td>
<td>08/07/88</td>
<td>D88-1STR</td>
<td>East side of TSC</td>
</tr>
<tr>
<td>ing S.W.)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dilatometer to 10.0m (membrane fac-</td>
<td>26/07/88</td>
<td>D88-2STR</td>
<td>West side of TSC</td>
</tr>
<tr>
<td>ing S.E.)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7 ($u_1$(ceramic)</td>
<td>11/02/88</td>
<td>C88-6STR</td>
<td>Stopped at 5.5m</td>
</tr>
<tr>
<td>and $u_3$)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7 ($u_1$(ceramic)</td>
<td>11/02/88</td>
<td>C88-7STR</td>
<td></td>
</tr>
<tr>
<td>and $u_3$) to 9.62m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC #7 ($u_1$(ceramic) and $u_3$)</td>
<td>26/02/88</td>
<td>C88-9STR</td>
<td>Dissipation data</td>
</tr>
<tr>
<td>to 9.68m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Trench 2</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell (TSC) installed</td>
<td>14/11/88</td>
<td>TSC1537</td>
<td>Parallel to trench</td>
</tr>
<tr>
<td>at 2.25m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed to 4.5m</td>
<td>14/11/88</td>
<td>TSC1538</td>
<td>Parallel to trench</td>
</tr>
<tr>
<td>UBC Dilatometer to 6.4m - hole used</td>
<td>15/11/88</td>
<td>D88-3STR</td>
<td></td>
</tr>
<tr>
<td>to install TSC1542</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed to 7.0m</td>
<td>15/11/88</td>
<td>TSC1542</td>
<td>Parallel to trench</td>
</tr>
<tr>
<td>Total stress cell installed to 5.5m</td>
<td>16/11/88</td>
<td>TSC1540</td>
<td>TSC damaged</td>
</tr>
<tr>
<td>UBC Dilatometer to 5.0m prior to</td>
<td>16/11/88</td>
<td>D88-4STR</td>
<td></td>
</tr>
<tr>
<td>installing TSC</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC Dilatometer to 8.0m prior to</td>
<td>17/11/88</td>
<td>D88-5STR</td>
<td></td>
</tr>
<tr>
<td>installing TSC</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total stress cell installed at 8.5m</td>
<td>17/11/88</td>
<td>TSC1539</td>
<td>TSC damaged</td>
</tr>
<tr>
<td>Total stress cell installed at 4.0m</td>
<td>18/11/88</td>
<td>TSC1541</td>
<td>Dummy DMT to 3.25m</td>
</tr>
</tbody>
</table>

Continued...
Table 4.8 Field Testing Programme at Strong Pit (Continued)

<table>
<thead>
<tr>
<th>Test Performed/Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Trench 3</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seismic CPTU, UBC #7 to 8.98m</td>
<td>10/01/88</td>
<td>C89-10STR</td>
<td>(u_2 ) &amp; (u_i); seismic (@ ) 0.5m intervals</td>
</tr>
<tr>
<td>Lateral stress cone LS-CPTU to 8.1m</td>
<td>12/01/89</td>
<td>STR-LS1</td>
<td>(u_2), (u_3), (u_{LS})</td>
</tr>
<tr>
<td>Lateral stress cone LS-CPTU to 8.1m</td>
<td>14/01/89</td>
<td>STR-LS2/189-SSTR</td>
<td>(u_1), (u_3), (u_{LS})</td>
</tr>
<tr>
<td>Geonor in situ vane test to 3.9m</td>
<td>26/01/89</td>
<td>STR-TP1</td>
<td>No tests possible</td>
</tr>
<tr>
<td>Excavation of test pit to 3.0m</td>
<td>25/09/89</td>
<td></td>
<td>Block samples recovered</td>
</tr>
<tr>
<td>Geonor in situ vane to 4.8m</td>
<td>13/10/89</td>
<td>V89-6STR</td>
<td>(u_1), (u_3), (u_{LS})</td>
</tr>
<tr>
<td>Lateral stress cone LS-CPTU to 8.78m</td>
<td>13/10/89</td>
<td>STR-LS3</td>
<td>(u_2), (u_3), (u_{LS})</td>
</tr>
<tr>
<td>Lateral stress cone to 9.4m</td>
<td>13/10/89</td>
<td>STR-LS4</td>
<td></td>
</tr>
<tr>
<td><strong>Trench 4</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hogentogler CPTU ((u_2)) to 6.97m</td>
<td>26/07/89</td>
<td>C88-11STR</td>
<td>Refusal (@) 6.97m</td>
</tr>
</tbody>
</table>

*Not shown on site plan.

TSC1350 and TSC1540 left in place due to rod breakage; all other TSCs removed from site in February in 1989 after 3 months in ground.

Table 4.9 Oedometer Test Results for Undisturbed Samples – Strong Pit

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(\gamma_t) (Mg/m³)</th>
<th>(\varepsilon_0)</th>
<th>(\omega_n) (%)</th>
<th>(\sigma_y') (kPa)</th>
<th>(\sigma_{vm}') (kPa)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Horizontal Samples</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.5</td>
<td>2.18</td>
<td>0.479</td>
<td>19.8</td>
<td>26</td>
<td>370</td>
<td>14.2</td>
</tr>
<tr>
<td>1.5</td>
<td>2.19</td>
<td>0.489</td>
<td>20.4</td>
<td>26</td>
<td>357</td>
<td>13.7</td>
</tr>
<tr>
<td>2.0</td>
<td>2.09</td>
<td>0.563</td>
<td>19.7</td>
<td>36</td>
<td>350</td>
<td>10.3</td>
</tr>
<tr>
<td>2.0</td>
<td>2.10</td>
<td>0.541</td>
<td>20.1</td>
<td>36</td>
<td>365</td>
<td>10.1</td>
</tr>
<tr>
<td>2.65</td>
<td>2.10</td>
<td>0.613</td>
<td>20.3</td>
<td>49</td>
<td>360</td>
<td>7.4</td>
</tr>
<tr>
<td>3.55</td>
<td>2.10</td>
<td>0.594</td>
<td>20.3</td>
<td>67</td>
<td>500</td>
<td>7.5</td>
</tr>
<tr>
<td>5.3</td>
<td>2.09</td>
<td>0.635</td>
<td>20.1</td>
<td>102</td>
<td>450</td>
<td>4.4</td>
</tr>
<tr>
<td>6.3</td>
<td>2.19</td>
<td>0.498</td>
<td>17.3</td>
<td>122</td>
<td>480</td>
<td>3.9</td>
</tr>
<tr>
<td>8.3</td>
<td>2.13</td>
<td>0.575</td>
<td>18.2</td>
<td>162</td>
<td>420</td>
<td>2.6</td>
</tr>
<tr>
<td><strong>Vertical Samples</strong></td>
<td></td>
<td>(\sigma_{vm}') (kPa)</td>
<td>(K_o)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>2.05</td>
<td>0.587</td>
<td>20.3</td>
<td>36</td>
<td>80</td>
<td>2.2</td>
</tr>
</tbody>
</table>

\[ K_o \]
4.15 Classification and index test results for Strong Pit.

Fig. 4.15 Classification and index test results for Strong Pit.

4.16 Undrained vane strength ratio profile - Strong Pit.

Fig. 4.16 Undrained vane strength ratio profile - Strong Pit.
4.7 200th Street, Langley

4.7.1 Site Description

The site is located to the north of the Trans-Canada Highway to the east of the 200th Street overpass (Fig. 4.18). The site was recently made available to UBC as a new interchange (Carvolth Road Interchange) is planned at this location. During the rainy season, the site is very soft and surface water is often present. To obtain access it was necessary to place a gravel pad over the area where testing was to be performed. The pad has dimensions 21 m x 13 m, being approximately 1 m thick. The site is located some 7 km
from the Lr. 232 St. site and the geological map suggests a similar stratigraphical sequence.

An earlier borehole investigation performed at the site by Golder Associates suggests the following soil profile (BH 107):

- G.L. - 0.3m: Gravelly fill
- 0.3m - 4.3m: Stiff silt
- 4.3m - 12m: Soft clayey silt
- > 12m: Till

No indication was given of the water table position. Considerable seasonal fluctuations exist at the site.
4.7.2 Testing Programme

Initial testing at the site was performed in November 1989 as part of the field work for the 577 Graduate Course in Geotechnical Engineering. Based on these initial studies it became apparent that from a stress history approach the site possessed interesting characteristics. Subsequent sampling and laboratory testing by Crawford (1989, 1990) confirmed the presence of an upper heavily OC layer and a lower lightly OC layer. It was decided to evaluate the horizontal stress variation at the site. The testing programme carried out is outlined in Table 4.10. The layout of the tests is illustrated in Fig. 4.19.

Fig. 4.19 Layout of tests at 200th St.
Table 4.10 Field Testing Programme at 200th Street

<table>
<thead>
<tr>
<th>Test Performed/Equipment Installed</th>
<th>Date</th>
<th>Designation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>UBC #7 Seismic CPTU to 12.24m (577)</td>
<td>02/11/89</td>
<td>C89-2001</td>
<td>(u_1) &amp; (u_3), BAT (u_2)</td>
</tr>
<tr>
<td>Hogentogler CPTU to 7.95m (577)</td>
<td>02/11/89</td>
<td>C89-2002</td>
<td>Medium vane (577)</td>
</tr>
<tr>
<td>Nilcon FVT to 11m at 1m intervals</td>
<td>02/11/89</td>
<td>V89-2001</td>
<td>Small vane (577)</td>
</tr>
<tr>
<td>Nilcon FVT to 11m at 1m intervals</td>
<td>09/11/89</td>
<td>V89-2002</td>
<td>Membrane facing south</td>
</tr>
<tr>
<td>Dilatometer to 12.15m (577)</td>
<td>09/11/89</td>
<td>D89-2001</td>
<td></td>
</tr>
<tr>
<td>Geonor FVT to 8m at 1m intervals</td>
<td>07/12/89</td>
<td>V89-2003</td>
<td>Facing west</td>
</tr>
<tr>
<td>Dilatometer to 8.4m</td>
<td>10/05/90</td>
<td>D90-2002</td>
<td>Facing west</td>
</tr>
<tr>
<td>Total stress cell installed at 9m</td>
<td>10/05/90</td>
<td>TSC1538</td>
<td>Facing west</td>
</tr>
<tr>
<td>Total stress cell installed at 6.5m</td>
<td>10/05/90</td>
<td>TSC1541</td>
<td>Facing west</td>
</tr>
<tr>
<td>Total stress cell installed at 3.5m</td>
<td>15/05/90</td>
<td>TSC1581</td>
<td>Facing west</td>
</tr>
<tr>
<td>Total stress cell installed at 5m</td>
<td>15/05/90</td>
<td>TSC1537</td>
<td>Facing west</td>
</tr>
<tr>
<td>Crosshole &amp; downhole seismic CPTU</td>
<td>04/07/90</td>
<td>XHDH-1</td>
<td>Aligned east-west</td>
</tr>
<tr>
<td>Removal of TSCs from site</td>
<td>11/07/90</td>
<td>SBLC1</td>
<td></td>
</tr>
<tr>
<td>Self-boring load cell to 4.0m</td>
<td>26/07/90</td>
<td>200-XH1</td>
<td>Aligned east-west</td>
</tr>
<tr>
<td>Downhole &amp; crosshole seismic CPTU</td>
<td>23/08/90</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

4.7.3 Geotechnical Characteristics

Results of Atterberg limit tests are shown in Fig. 4.20. Plastic limits in the range 22-33% (Average = 27%) and liquid limits in the range 33-57% (Average 47%) were measured. Average PI from these limits was 20%. The undrained strength ratio profile for the site is shown in Fig. 4.20 as well as the variation in shear wave velocities with depth.

Both the \(S_u\) and \(V_s\) profiles indicate a similar stress history variation as determined from laboratory constant rate of strain consolidation tests (Table 4.11).
Fig. 4.20 Geotechnical data for 200th St.
Table 4.11 Oedometer Test Results for Undisturbed Samples - 200th St. (Modified after Crawford, 1989).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(e_o)</th>
<th>(w_n) (%)</th>
<th>(\sigma_{V}^{11}) (kPa)</th>
<th>(\sigma_{VM}^{1}) (kPa)</th>
<th>OCR</th>
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</thead>
<tbody>
<tr>
<td>1.08</td>
<td>1.10</td>
<td>39.5</td>
<td>18</td>
<td>300</td>
<td>16.7</td>
</tr>
<tr>
<td>2.13</td>
<td>1.40</td>
<td>47.0</td>
<td>26</td>
<td>330</td>
<td>12.7</td>
</tr>
<tr>
<td>*2.20</td>
<td>1.13</td>
<td>39.3</td>
<td>27</td>
<td>197</td>
<td>7.3</td>
</tr>
<tr>
<td>*2.23</td>
<td>1.07</td>
<td>38.7</td>
<td>27.3</td>
<td>190</td>
<td>7.0</td>
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<td>4.43</td>
<td>1.03</td>
<td>35.3</td>
<td>45</td>
<td>224</td>
<td>5.0</td>
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<tr>
<td>5.20</td>
<td>1.32</td>
<td>-</td>
<td>50.6</td>
<td>115</td>
<td>2.3</td>
</tr>
<tr>
<td>5.25</td>
<td>0.89</td>
<td>31.3</td>
<td>51</td>
<td>95</td>
<td>1.9</td>
</tr>
<tr>
<td>6.23</td>
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<td>59.1</td>
<td>56</td>
<td>121</td>
<td>2.2</td>
</tr>
<tr>
<td>7.12</td>
<td>1.93</td>
<td>67.4</td>
<td>60</td>
<td>120</td>
<td>2.0</td>
</tr>
<tr>
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<td>2.02</td>
<td>72.4</td>
<td>63</td>
<td>95</td>
<td>1.5</td>
</tr>
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<td>8.14</td>
<td>1.46</td>
<td>-</td>
<td>64.5</td>
<td>90</td>
<td>1.4</td>
</tr>
<tr>
<td>9.93</td>
<td>1.17</td>
<td>42.1</td>
<td>74</td>
<td>130</td>
<td>1.8</td>
</tr>
<tr>
<td>10.75</td>
<td>1.26</td>
<td>46.0</td>
<td>80</td>
<td>235</td>
<td>2.9</td>
</tr>
</tbody>
</table>

* Water table taken at 1m below ground level (at base of gravel pad).
* Results from lateral stress oedometer tests (this study).
5. COMPARISON OF MEASURED STRESSES AND PORE PRESSURES

5.1 Introduction

The usefulness of any technique to predict soil state or engineering parameters depends on the test repeatability and consistency of the interpretation method. Both of these should be judged with due consideration given to the inherent variability of natural materials.

Full displacement probes cause significant but repeatable degrees of disturbance to the soil during insertion. This has been confirmed by numerical studies using the strain path approach (Baligh, 1985; Teh and Houlbysy, 1988; Huang, 1989) for both cylindrical and plate-like probes. However, even though the strain domain may be very similar for any soil, the stresses induced will vary considerably and will depend on the soil type behaviour characteristics. In coarse granular soils, no excess pore pressures will be generated during penetration and the total stress increase will closely mirror the effective stress change. In fine grained soils, where drainage is very slow, the penetration-induced excess pore pressures will be related to the total stress changes. The magnitude of the measured total and effective stress changes will be related to various factors such as:

- geometry of the penetrometer
- location of the measurement sensor
- soil behaviour characteristics

These factors are considered below in relation to the results obtained from tests performed using the probes available at UBC (Chapter 3) and the soil
types encountered at the various research sites (Chapter 4). In sections 5.2 and 5.3, only the pore pressures and lateral stresses measured during penetration and the resulting initial effective stresses are considered. Dissipation of the excess pressures are considered in a subsequent section.

5.2 Measured Pressures in Fine Grained Soils

Detailed measurements of both pore pressures and lateral stresses from three research sites (McDonald Farm, Lr. 232 Street, Strong Pit) are presented below. The fine grained soils at McDonald Farm are normally consolidated (NC) clay silts. Surficial OC and deeper NC sensitive clay silts are present at Lr. 232 Street while only OC soils are present at Strong Pit.

5.2.1 Penetration Pore Pressures in Clays

The measurement of pore pressures during penetration of both self-boring and full-displacement probes has been a topic of considerable interest especially since the introduction of piezocones in the mid-1970's. Campanella and Robertson (1988) consider the following factors to be of importance when evaluating the results of soundings:

a) mechanical design of the probe
b) pore pressure element location
c) type and size of porous element
d) saturation of porous element
e) rate of penetration

Campanella, Robertson and Sully (1988) discussed the relevance of field techniques to the interpretation of piezocone data by considering the above factors and their effect on measured pore pressures. In terms of evaluating
the distribution of pore pressure around a penetrating probe, only item (b) will be discussed in detail, although reference to items (a) and (c) will be made.

McDonald Farm

The normally consolidated silt below 15 m at McDonald Farm is ideally suited for comparing results of adjacent tests due to its relative homogeneity. Combining the results of the two types of lateral stress cone discussed in Chapter 3, it is possible to obtain pore pressure measurements at six different locations around a penetrating cone. The results are presented in Fig. 5.1. Four measurements are from the 44 mm diameter UBC LS cone while two additional results are from the 36 mm diameter UCB LS cone. The excess pore pressure induced due to penetration is essentially independent of the cone diameter. The data have been classified in terms of an L/D ratio where L is the distance from the tip to the centre of the porous filter and D is the diameter of the penetrometer. For the range of results obtained, L/D varies from 0.5 to 17.6.

The pore pressure measured at the $u_1$ location (on the tip of the penetrometer) is the maximum and occurs at L/D values of between zero and 0.9. At the $u_2$ location the soil has passed the cone shoulder and a reduced pore pressure is recorded for L/D of around 1.0. The pore pressure measured at the location L/D = 1.25 with the UCB cone agrees well with the L/D = 1.0 data. At the $u_3$ location (L/D = 3.9) a further noticeable reduction in pore pressure occurs. To date it has been suggested that this reduction is due to both dissipation of excess pore pressures and stress relaxation. Fig. 5.1 also illustrates the very small variation in pore pressures measured at L/D ratios of 3.9 ($u_3$), 7 ($u_{upper}$) and 17.6 ($u_{LS}$); in effect the three measure
Figure 5.1 Comparison of pore pressures measured at different locations on penetrating cones in NC clay silt.
ments are identical. This would suggest that for porous filters located at L/D ratios of 4 or greater, the measured pore pressures in NC soils are not affected by stress changes induced in the region of the tip.

It has been demonstrated by Robertson et al. (1988) that the DMT $p_2$ closure pressure is identical to the CPTU $u_3$ pore pressure at this site.

Lr. 232 Street

A similar pattern of measured pore pressures was obtained at this site (Fig. 5.2) using a standard UBC piezocone. To evaluate the gradient of pore pressure close to the tip, a 2.5 mm high filter was used in the $u_2$ location as opposed to the standard 5 mm high filter. The results are shown in Fig. 5.3 and illustrate that in the NC clay silt the pore pressures measured using the two filter thicknesses are identical. This would suggest that in soft NC soils, the gradient of pore pressure change is such that the unloading as the soil passes the tip is not instantaneous but occurs over some small finite distance (say 5-10 mm). This is confirmed by tests reported by Campanella et al. (1986) in a stiff OC clay. Their data suggests that the maximum effect of the unloading is measured 5 mm to 10 mm behind the cone shoulder, and that the magnitude of the unloading depends on the soil stiffness. This is also evident in Fig. 5.3. At several depths in the profile substantial reductions in pore pressures are generated which correspond to dense sand/silt layers of high cone resistance. Recovery of the pore pressure is especially sluggish when those pressures are negative of hydrostatic and probably result from partial loss of saturation and in some cases cavitation occurs. The delay in recovery to the maximum penetration value on re-entering the soft clay silt depends on the ease with which water can flow back into the measuring system as indicated by comparing the response of the 5 mm and 2.5 mm high filters.
Figure 5.2 Pore pressures measured at three locations during CPTU at Lr. 232 Street (Campanella, Sully and Robertson, 1988).
Figure 5.3 Effect of filter size on \( u_2 \) pore pressure response for soft NC cohesive soil (Campanella, Sully and Robertson, 1988).
The 5 mm filter recovers very rapidly, but some delay occurs with the 2.5 mm filter.

Repeatability of pore pressure measurement was a problem at this site for the \( u_1 \) location in the upper 3 m of the deposit. On occasions, the measured pore pressures were considerably lower than that expected, based on the stress history of the deposit, and thus also inconsistent. However, at all other locations, very consistent measurements of penetration pore pressure were obtained over the period of testing. Figure 5.4 illustrates this for the \( u_1 \) measurements obtained using the UBC lateral stress cone. The reason for the highly variable and inconsistent data measured at the \( u_1 \) location became apparent when a trial pit was excavated at the site for the recovery of undisturbed block samples. To the excavated depth of 2.5 m the overconsolidated crust was seen to contain numerous, mainly vertical, open mineralized root holes which influence both the excess pore pressure measured during cone penetration and its subsequent dissipation if penetration were to be halted. As would be expected, it is the \( u_1 \) pore pressure measurements that are most affected by the presence of the open root holes resulting in lower than expected values. Once the tip of the penetrometer had destroyed the structure of the root hole and soil, the succeeding pore pressure sensors did not experience the same effects on the measured pore pressure.

A comparison of the pore pressures measured on cylindrical probes for both self-boring and full-displacement installation is shown on Fig. 5.5(a). The pore pressures measured during installation of the self-boring load cell (SBLC) or Camkometer are close to hydrostatic and indicate the small amount of disturbance induced during the self-boring process. However, due to the insensitivity of the pore pressure sensors on the SBLC (see Chapter 3), the measured values presented in Fig. 5.5(a) can only be used as rough estimates.
Figure 5.4 Comparison of $u_3$ pore pressure profiles from LS-CPTU at Lr. 232 Street.
Figure 5.5 Comparison of measured pore pressures at Lr. 232 Street.

(a) Cylindrical probes

(b) Flat probes
of actual values. The distribution of hydrostatic pore pressure, $u_0$, gives a reasonable approximation to the average pressure measured on the SBLC sensors A and B for depths greater than 3.5 m.

The LS-CPTU pore pressures are much higher than the SBLC values as expected and present the same trend as discussed earlier for McDonald Farm. In the surficial overconsolidated crust the $u_1$ values are proportionately much higher than the $u_2$ values when compared to the results in the deeper NC clay silt.

Figure 5.5(b) compares the pore pressure measured using flat probes (dilatometer and total stress cell) at the same site. Also shown for comparison the pore pressure measured at the lateral stress sleeve ($u_{LS}$) on the LS cone. The standard DMT pore pressures are higher than those recorded by the TSC pore pressure sensor. The pore pressures measured by the UBC research DMT are lower than those recorded by the standard DMT since redistribution of the excess pore pressures occurs on stopping penetration. The lateral stress cone $u_{LS}$ values agree better with the TSC ($u_{TSC}$) than the DMT values even though the TSC data are subject to considerable scatter. Robertson et al. (1988) suggest that $p_1$ (DMT) is very similar to $u_3$ (CPTU). Since it was shown earlier that $u_3 = u_{LS}'$ the DMT and LSC pore pressures should agree, which is not the case. Intuitively one would expect the TSC pore pressures to be lower than those generated by insertion of the DMT since the TSC blade is thinner, hence the relative magnitude of these measurements appears correct. The difference probably results from local site variations. Robertson et al. (1988) show $p_3$ (DMT) slightly larger than $u_3$ for McDonald Farm but $u_4 > p_3$ for Lr. 232 Street. Of the two pore pressure measurements $u_3$ is considered by the writer to be more reliable than $p_3$. It is also interesting to note that $p_2$ measured with the standard DMT is higher than the
pore pressure measured with the research DMT. The CPT $u_3$ pressure is somewhere between the two DMT pore pressures.

Comparison of $u_{LS}$ with $p_2$ and $u_{TSC}$ is however complicated by the different modes of measurement. The LS cone value is obtained during penetration, as is the $u_{DMT}$ obtained from the research DMT. Both $p_2$ and $u_{TSC}$ are recorded once penetration has halted and a finite time has passed. As has been demonstrated in the literature, pore pressures measured behind the penetrometer tip may increase once penetration is halted due to stress redistribution and pore pressure equalization (Campanella, Sully and Robertson, 1988). This effect is most pronounced at the $u_2$ and $u_3$ locations, i.e. close to the tip shoulder. The $u_{LS}$ values should hence only be compared with $u_{DMT}$ and $u_{TSC}$ compared with $p_2$. As expected, $u_{TSC} < p_2$ but contrary to expectations, is the result that for both the OC and NC soil $u_{LS} < u_{DMT}$.

Strong Pit

The pore pressure profiles to a depth of 9 m measured at the four possible locations using the UBC LS cone are presented in Fig. 5.6(a) for the stiff OC clay silt at this site. It was not possible to penetrate the LS cone to depths greater than 9 m due to the presence of very dense sand and gravel layers below this level. Because of the high strength and stiffness of this fine grained deposit, the pore pressures measured at all locations are much higher than recorded at Lr. 232 St. Furthermore, the reduction in pore pressure as the pore pressure measurement location is distanced from the tip is also larger than that recorded at Lr. 232 St. The presence of free draining layers within the profile is clearly indicated by the pore pressure response, being most marked for the $u_1$ location.
Figure 5.6 Comparison of measured pore pressures at Strong Pit.
Figure 5.6(b) compares the pore pressures measured by the plate penetrometers using the $u_{LS}$ as a reference to Fig. 5.6(a). At this site, the higher variability does not afford as clean a correspondence between $u_{TSC}$ and $u_{LS}$ as obtained at Lr. 232 St. A certain degree of agreement between $u_{TSC}$ and $u_{LS}$ can be observed but insufficient data exist for any definite conclusion to be drawn. It was not possible to realistically obtain further TSC measurements at this site due to the gravel and cobble dropstones within the silt. Of the nine total stress cells installed at this site only six functioned satisfactorily, the remainder suffering damage during installation. This illustrates the problem of using the thin TSC's in this type of deposit; the possibility of breakage is high and the instruments are costly. On the other hand no problems were experienced using the cylindrical probes.

In stiff soils, pore pressures measured at the tip can be affected by:
- mechanical load transfer
- filter squeeze

The mechanical design of the probe should ensure that when the cone is loaded no transfer of the load to the pore pressure measuring system should occur. The procedure for verifying if the problem exists involves loading an assembled cone and recording the response of both the bearing and pore pressure which normally forms an integral part of the cone calibration. Results of such a test have been presented by Battaglio et al. (1986). If mechanical load transfer onto the pore pressure measuring system occurs the magnitude of any induced pore pressure will also depend on the filter location and soil stiffness. UBC cones are designed so that no load transfer to the pore pressure transducer occurs. The calibration procedure is
designed to provide checks on this by monitoring all channels as the tip transducer is loaded.

Filter squeeze or compression can be important for cones that measure pore pressures at the tip. During penetration into stiff layers with high cone resistance the filter element can become compressed and generate a pore pressure in addition to that which would have been measured with no compressibility effects. This may occur unless the filter element has a very low compressibility or if filter and soil are of sufficient permeability to rapidly dissipate the pore pressure due to filter element compression. Based on the results presented by Battaglio et al. (1986) it was thought that filter compression may be partly responsible for the high \( u_1 \) pore pressures recorded at Strong Pit. To check this, separate soundings were performed using a standard UBC porous polypropylene filter and an aerolith 10 (ceramic) filter element located on the cone face at midheight \( (u_1) \). The \( u_1 \) and \( q_T \) profiles are shown in Fig. 5.7. Allowing for the depth offset between the two profiles (0.25 m) the two pore pressures traces are essentially identical to 5 m. Below 5 m the pore pressure measured with the ceramic filter is lower than that measured with the plastic one, as is the cone bearing. However, the difference between the pore pressures is not constant but decreases with depth within a particular horizon. The pore pressures measured with the ceramic filter increase gradually between 5 m and 7 m whilst the plastic filter records an almost constant pore pressure. The response of the ceramic filter appears somewhat sluggish. When the piezocone was retrieved from the ground the ceramic filter had broken up and was lost during the test. This created an undersized sensing area on the face. Clogging of the pore pressure outlets (no longer protected by the filter stone) and unloading caused by the space behind the tip where the filter has
Fig. 5.7 Comparison of pore pressures measured at the u, location with plastic and ceramic filter elements in OC clay silt
been located could explain the poor response of the ceramic filter below 5 m. Above 5 m, the results would suggest that compressibility of the filter material does not affect the measured pore pressure response for this site. To date, comparisons of filter compressibility on the face have not resulted in any changes in interpretation of results. There is no noticeable difference in pore pressures measured at locations behind the tip when comparing ceramic and porous polypropylene filter response.

5.2.2 Lateral Stress Measurements in Clays

Little work has been published in relation to fundamental measurements of lateral stresses around penetrating probes and how the stress varies along the length of the probe. Measurements made with model piles suggest that the initial stresses during penetration are dependent on soil characteristics (Azzouz and Morrison, 1988). Experience at UBC also suggests that probe characteristics are a controlling factor. As part of this research, lateral stress measurements were conducted at several research sites using both flat and cylindrical probes. The results obtained at three sites comprising fine grained deposits are discussed in this section.

McDonald Farm

The results of lateral stress measurements using the various pieces of equipment available are presented in Fig. 5.8. The lateral stress cone data from both the UBC and the Berkeley (UCB) instruments shown in Fig. 5.8(a) are in excellent agreement and fall below the DMT lift-off pressure, $p_0$. In all cases the measured total lateral stress is greater than the total overburden pressure, $\sigma_v$. The measured lift-off and limit pressures from self-boring and
Figure 5.8. Comparison of lateral pressures measured using cylindrical and flat probes in NC fine grained soils at McDonald Farm.
full-displacement pressuremeters reported by Hers (1989) are presented in Fig. 5.8(b). The DMT 1.1 mm expansion pressure, \( p_2 \), is also indicated on the figure.

Comparing the two plots, it is apparent that the lateral stress measured during LS-CPTU coincides with the lift-off pressures from the two full-displacement pressuremeters (Fugro CPM and UBC SCPM). The good correspondence between \( \sigma_{LS} \) and \( p_0 \) (PM) suggests that the same lateral stress is measured whether the probe is stationary or moving. Lift-off pressures from the self-boring pressuremeter vary considerably as demonstrated earlier in Chapter 2. The two DMT pressures \( p_0 \) and \( p_1 \) are sandwiched between the SBPM \( p_0 \) and \( p_L \) such that:

\[
(p_0)_{SBPM} < p_0 \text{ (DMT)}
\]

and

\[
(p_L)_{SBPM} > p_1 \text{ (DMT)}
\]

The L/D ratios quoted on Fig. 5.8(b) refer to the ratio of the distance to the centre of the PM section (L) with respect to probe diameter (D). For the DMT, the ratio involves the blade thickness, \( t \), and the distance to the center of the membrane, L. For the DMT, \( L/t = 6.8 \) which may explain the high total pressures when compared to the much higher L/D ratios for the other cylindrical probes.

**Lr. 232 Street**

The repeatability of the LS cone data is demonstrated in Fig. 5.9(a). Four of the five soundings performed at the site are in good agreement. The slightly higher \( \sigma_{LS} \) measured during test LS5 is considered to result from lateral variations in soil characteristics. LS5 was performed on the same
Penetration lateral stress (kPa) DMT lift-off (kPa)

0 200 400 600 0 200 400 600

(a) LS cone data (b) Flat penetrometer lift-off values

Figure 5.9 Measured lateral stresses at Lr 232 St.
day as LS4. At the start and end of both tests very similar baseline values on the lateral stress channel were recorded suggesting that the difference (down to a depth of 12 m) was not equipment related. Below 12 m all five soundings give very similar lateral pressures.

The LS cone measures the in situ horizontal stress acting on a cylindrical instrumented sleeve; for this reason no directional stress measurement is possible. However, with the DMT it is possible to measure pressures in specified directions since the expandable membrane is mounted on one side of a flat penetrometer. Two DMT soundings were performed at Lr. 232 Street with different membrane orientations, one facing south-east (membrane pointing along contour) and the other facing south-west (membrane pointing downslope). The lift-off pressure (0.05 mm movement) in the SE direction is lower than that in the SW direction, which suggests a higher lateral stress condition in the downslope direction. At depths of 10 m or more, the two lift-off pressures are very similar; the difference that exists may be due to natural soil variations.

Also shown on Fig. 5.9(b) is the $p_0$ profile obtained from the UBC research DMT (Tsang, 1987), which plots below the standard Marchetti DMT data. As mentioned above, the DMT lift-off pressure does not represent a true lift-off condition since it corresponds to a membrane movement of 0.05 mm. The UBC R-DMT on the other hand provides a true lift-off pressure, i.e. the pressure (after corrections for membrane compliance) at which the membrane begins to move off of its rest position. Logically then the R-DMT will provide lower lift-off pressures. The initial design of the Marchetti standard DMT was such that the buzzer sounded at membrane lift-off; all the empirical correlations were developed for this design. Due to problems with the original design, the concept was changed so that the buzzer sounded after
0.05 mm movement and the "actual lift-off" obtained by back-extrapolation (see Appendix A, Section A for more details). As indicated by the data in Fig. 5.9(b) the back-extrapolated $p_0$ values appear to be noticeably higher than the actual values measured with the UBC R-DMT. The initial lateral stresses (immediately after penetration) measured by the total stress cells installed at this site are also included on Fig. 5.9(b). The increase in measured lateral stress can be seen to increase substantially as the blade thickness increases. Soundings have been performed at a site close to Lr. 232 St. using the offshore DMT developed at the Norwegian Geotechnical Institute (By et al., 1987). The NGI offshore DMT has a blade thickness of 16 mm. Results of soundings at the Langley site and at other soft clay sites in the Lower Mainland indicate higher lift-off pressures from the 16 mm blade than from both the 13.7 mm blades (UBC research DMT and standard Marchetti DMT). Assigning a blade thickness of 13.75 mm to the standard DMT to consider the 0.05 mm expansion we have:

$$\sigma_{TSC}^{(t=6\text{mm})} < p_0^{(\text{UBC RDMT, 13.7mm})} < p_0^{(\text{DMT, 13.75mm})} < p_0^{(\text{NGI DMT, 16mm})}$$

The plate penetrometer stress measurements are compared to the cylindrical probe data in Figs. 5.10(a) and (b), using the LS cone measurements on each as a reference. The lowest stresses measured correspond to the self-boring load cell (SBLC). As mentioned earlier the pore pressures recorded during installation of the SBLC were approximately hydrostatic indicating the low degree of disturbance caused during installation. The SBLC measured total lateral stresses are greater than $\sigma_v$ at the surface reducing with depth to about 5 m. Below 5 m, the pressures increase linearly with depth. One of the lateral stress sensors records a pressure essentially equal to $u_0$ (C)
Figure 5.10 Lateral stress measurements at Lr. 232 Street using cylindrical and flat penetrometers.
whereas the second sensor (D) gives pressures between $u_0$ and $\sigma_v'$. The SBLC lateral pressures were initially low on halting penetration of the probe. The pressures presented in Fig. 5.10(a) are those measured approximately 20 minutes later, once the stress and pore pressure had stabilized.

The rest of the data presented in Fig. 5.10 are for full displacement probes where comparison with Fig. 5.5 illustrates the predominance of pore pressure on the measured stresses in the NC soil. The TSC and seismic cone pressuremeter (SCPM) lateral stresses are very similar. It is interesting to note that for the TSC the $L/t = 30.8$ whereas for the SCPM the ratio $L/D = 29.8$. It must be remembered, however, that the comparison is not strictly valid since the SCPM values are slightly lower than would be expected due to:

- The PM section is slightly undersized which gives rise to unloading of the soil in the area of the PM lantern. This aspect is not thought to be important in soft clay as evidenced by the good agreement between SCPM and LS-CPTU data at McDonald Farm.
- More importantly, relaxation periods of up to 30 minutes were allowed prior to commencing inflation of the PM which would permit significant dissipation of excess pore pressures to occur. SCPM lift-off pressures at other sites indicate that the pressures measured at Lr 232 St. are lower than expected. Since the SCPM does not have a pore pressure transducer on the lantern it is not possible to verify this idea.

Considering the pore pressure distribution around a penetrating cone and the absence of large gradients along the shaft away from the tip, it should be possible to estimate a pore pressure value corresponding to the location of the pressuremeter module. However, the pore pressure measured at the lateral stress module (0.775 m behind tip) is larger than the total stresses
at the PM location (1.31 m behind tip). It would appear then that either the pore pressure has also been reduced due to the unloading or that significant pore pressure dissipation has occurred during the 30 minute relaxation period. In addition, undrained creep may have caused a stress redistribution around the probe also resulting in lower measured stresses. This emphasizes the importance of pore pressure measurement in the vicinity of the PM section if rational interpretation of measured total stresses is to be attempted. The fact that the PM section is slightly undersized does not preclude at least an empirical interpretation of measured stresses provided the corresponding pore pressures are also known.

The \( \sigma_{LS} \) values are slightly higher than the \( \sigma_{TSC} \). The variation in \( \sigma_{LS} \) with depth is almost linear and fairly consistent. The DMT \( p_0 \) values lies above \( \sigma_{LS} \) as witnessed earlier for McDonald Farm.

For comparison and completeness, three limit pressures are included on Fig. 5.10(b) namely \( p_1 \) (DMT), \( p_L \) (SCPM) and \( q_T \) (CPT). It is interesting to note that the initial decrease and subsequent increase in \( q_T \) suggests the presence of an overconsolidated surficial crust (with higher \( K_0 \)) which then becomes normally consolidated at about 5 m depth. Below 5 m, \( q_T \) increases linearly with depth. With the possible exception of the TSC data, none of the direct lateral stress measurements show the same pronounced near surface changes as does \( q_T \). This, of course, assumes that similar pore pressures (i.e. positive) are developed at all locations. Whereas the tip pore pressures are likely to be very high even in the partially saturated near-surface soil, at locations further behind the tip they may well be negative. Consequently, consideration of only the total stress may be misleading especially when interpreting data in near-surface soils where stress history
has been imparted due to dessication, as is thought to be the case for Lr. 232 St.

Finally it is also very apparent from Fig. 5.10 that stress gradients exist along the probe. The reduction in measured lateral stress as L/D increases is evident from the figure. While most of this is due to pore pressures, it is likely that effective stresses may also vary but to a much lesser degree.

**Strong Pit**

Similar to Lr. 232 St., directional DMT soundings were performed at this site to evaluate the existence of horizontal stress anisotropy. The results are presented in Fig. 5.11(a). Apart from variations in $p_0$ below 8 m the two soundings are consistent with isotropic $\sigma_h$. Due to the nature of the soil at this site (see Chapter 4) it is difficult to be certain as to the meaning of Fig. 5.11(a). The results of three DMT soundings all performed in the N-S direction prior to the installation of the total stress cells (TSC) are presented in Fig. 5.11(b). The scatter in the $p_0$ values increases with depth as also suggested in Fig. 5.11(a). The TSC data also show considerable variation as was also apparent from the pore pressure measurements. Due to the stress history of the site, it is probable that these variations are a result of lithological rather than stress changes.

Figure 5.12 compares the representative DMT and TSC lateral pressures with results from cylindrical probes. The variation in $q_T$ has also been included on the figure. At this site the agreement between $\sigma_{LS}$ and $\sigma_{TSC}$ is good, with the $p_0$ (DMT) pressures being considerably higher than the $\sigma_{LS}$ results. Based on the results obtained at McDonald Farm and Lr. 232 St. it would appear that the disturbance caused by the flat penetrometers increases
Figure 5.11 Flat penetrometer data from Strong Pit illustrating, (a) directional DMT $p_0$ values, and (b) variation in DMT lift-off pressures in N-S direction.
Figure 5.12 Lateral stress measured using full displacement probes at Strong Pit (Sully and Campanella, 1990).
relative to that caused by the cylindrical probes as the soil stiffness increases. This may result from a larger non-uniform zone of stress (associated with the finite blade width) induced by the increased soil stiffness such that the stress concentration zone impinges on, or effects the area of measurement of the flat penetrometer.

Finally, it is evident that the variation of $q_T$ with depth is similarly indicated by the TSC, DMT and LS cone results, even though the scale of variation is somewhat reduced, probably as a result of the lower excess pore pressures. This confirms the idea that pore pressure effects were responsible for the lack of correspondence at Lr. 232 St. rather than the fact that the full displacement probes had obliterated all evidence of the stress history of the soil.

5.2.3 Initial Effective Lateral Stresses in Clays

The above sections have compared the pore pressures and total lateral stresses measured by different types of in situ testing equipment in fine grained soils. In terms of interpreting the data to evaluate the effect of in situ state on the measured parameters it is also instructive to examine the initial effective lateral stress acting on the probes. This is possible for probes where both lateral stress and pore pressures are measured. For some of the pressuremeter data where only total stress is recorded, this is not possible. In some cases, however, estimates of pore pressures can be obtained using the distributions obtained from the measurements presented in Section 5.2.1.

The initial effective lateral stress is defined as the value obtained by subtracting the penetration pore pressure from the penetration lateral stress. For the case of the LS cone:
where both $\sigma_{LS}$ and $u_{LS}$ are located essentially at the same point and are measurements taken during penetration of the lateral stress cone. For other probes such as the DMT, TSC and SBPM, the stress and pore pressure measurements are obtained once penetration has been halted and either an active or passive type of measurement is performed, but no dissipation of excess pressures is allowed prior to measurement. For the DMT, the effective lift-off is given by:

$$p'_0 = p_0 - p_2$$  \hspace{1cm} (5.2)

Conceptually it would appear that $p_2$ is not the correct pore pressure to use in the equation since it is measured only after first fully expanding the diaphragm and then unloading to the lift-off position. The definition of $p'_0$ is consistent for soft clay where the effective stresses on the membrane are very small and remain almost constant during the expansion and deflation phases of the test (Campanella and Robertson, 1991). The applied total stress increment ($p_1 - p_0$) during undrained expansion of the membrane in soft NC soil is matched by a corresponding rise in pore pressure; this is also true for the unloading phase from $p_1$ back to $p_0$. Only in soils of increased permeability would some change in the pore pressure response be obtained due to dissipation.

In stiff clay, Campanella and Robertson (1991) show that contrary to the soft clay results, the effective stresses on the membrane change in a similar manner to the total stress. They also conclude that the closure pressure, $p_2$ is unrelated to either the penetration or equilibrium pore pressure. This
latter point may be explained by the fact that during contraction of the membrane, separation of the soil and membrane occurs in stiff soil due to the inability of the soil to follow the membrane back. This would account for the negative and often erratic $p_2$ values measured in stiff fine grained soils.

For self-boring pressuremeter tests, the results presented above suggest that in fine-grained soils where little disturbance occurs, the measured pore pressures are very close to hydrostatic, hence:

$$\sigma'_{SBPM} = \sigma_{SBPM} - u_0 \quad (5.3)$$

$$\sigma'_{SBLC} = \sigma_{SBLC} - u_0 \quad (5.4)$$

This assumption can be used where pore pressure measurements are not available and may provide some insight into the quality of the SBPM lift-off pressures measured ($\sigma_{SBPM}$) and the excess pore pressures generated during self-boring.

The variation of initial effective lateral stress from various in situ probes in soft clay silt at McDonald Farm is indicated in Fig. 5.13. Also shown on the figure are the variations of $\sigma'_v (K_0 = 1)$ and $\sigma'_h$ for the condition $K_0 = 0.5$. In Figure 5.13(a) the results from the UBC and UCB (Berkeley) LS cones are compared. For both cones the pore pressure used to obtain $\sigma'_L$ from Eq. (5.1) is that measured closest to the lateral stress sleeve. The UBC LS cone data shows an initial high of effective stress between 15 m and 17 m before reducing to a value which increases slightly with depth. The effective stress varies somewhat due to the sensitivity of the measuring system, as discussed in Chapter 3. The average effective
Figure 5.13 Initial effective lateral stress variation from various in situ probes in soft clay silt at McDonald Farm.
lateral stress below 17 m approximates to a value slightly below the calculated $\sigma_n'$ for the condition $K_0 = 0.5$. Between 15 m and 17 m the increased $\sigma_n'$ is considered to result from drainage in the transition zone from the overlying sand to the underlying silt.

The UCB (Berkeley) LS cone data show approximately the same trend with depth as the UBC cone data, with a lower average value. In the range 15 m to 17 m the UCB cone data do not indicate the presence of the interbedded sand and silt transition zone and the calculated effective lateral stresses are unexpectedly low.

As stated earlier the lower lateral stress sensor on the UCB cone was damaged in the overlying sand and no data were obtained. Based on the measured data the estimated lateral stress coefficient for the two types of LS cone used can be derived from:

$$K_{LS} = \frac{\sigma_{LS}'}{\sigma_V'}$$

For the soft clay silt at McDonald Farm, the data in Figure 5.13(a) give average values of $K_{LS}$ of 0.4 (UCB LS cone) and 0.45 (UBC LS cone).

Figure 5.13(b) compares the UBC LS cone data with measurements obtained with the DMT and SBPM. The SBPM effective lift-off data is very scattered throughout the profile suggesting that variable and sometimes significant excess pore pressures were induced as a result of self-boring. The lower data values which fall slightly above the $K_0 = 0.5$ line are taken from Konrad et al. (1987) and suggest very little disturbance during installation. Consequently, subtracting the hydrostatic pore pressure to obtain $\sigma_{SBPM}'$ (Eq. 5.3) does not cause large errors in the final result, if as supposed the $\sigma_n'$ variation is close to that given by $K_0 = 0.5$. The remaining SBPM data
indicate effective lateral pressures close to or greater than the $K_0 = 1$ condition.

The disturbance suggested by the high $\sigma'_{SBPM}$ values is confirmed by comparison with the DMT effective lift-off pressures, $p'_0$ (Eq. 5.2). The $p'_0$ values decrease rapidly in the upper transition layer to a minimum at 18 m depth before linearly increasing with depth, almost parallel to the LS cone data. Also shown on Fig. 5.13(b) are the DMT effective limit pressures, $p'_1$, defined as:

$$p'_1 = p'_1 - p'_2$$  \hspace{1cm} (5.6)

By definition, $p'_1$ is not truly an effective stress since during the expansion from $p'_0$ to $p'_2$ excess pore pressures are generated. The comparison with the SBPM data does however serve to illustrate the notable degree of disturbance that has occurred in some of the tests. Even for the lower bound SBPM data, which agree fairly well with the $p'_0$ values, some disturbance has occurred since the DMT is a full displacement probe. For the lower bound SBPM and DMT $p'_0$ data, the calculated $K_0$ averages 0.65. $K_0$ equal to 1.0 is obtained for the $p'_1$ results.

Data from four types of in situ probes obtained at Lr. 232 St. are shown in Fig. 5.14(a). The following points can be noted:

- The effective $p'_0$ and $p'_1$ pressures are very similar indicating that in this sensitive soils, the $p'_0$ to $p'_1$ membrane expansion requires very little stress increase. It would appear that in this case $p'_0$ is essentially a limit pressure.
Figure 5.14 Initial effective lateral stress results from Lr. 232 Street.
• The relative magnitudes of the total lift-off pressures presented earlier in Fig. 5.9(b) gives TSC values much lower than DMT values which is to be expected based on the degree of disturbance induced relative to the blade thicknesses. The small difference in effective pressures for the TSC and DMT suggests the unreliable measurement of pore pressures with the TSC equipment. This is confirmed by the inconsistent pore pressures measured as indicated in Fig. 5.5(b).

• Below 5 m, the SBLC gives effective pressures very close to the calculated pressures corresponding to $K_0 = 0.5$. Above 5 m the $\sigma'_{SBLC}$ values increase towards the ground surface consistent with the presence of a surficial overconsolidated crust.

• The LS cone data are very scattered ranging from $\sigma'_{LS}$ close to zero to as high as 0.5 $\sigma'_{v}$. In the fine sand layers $\sigma'_{LS}$ values are very peaked. As with the SBLC results, the presence of an overconsolidated crust is indicated to a depth of about 5 m. Below 5 m, the average value of $\sigma'_{LS}$ increases linearly with depth.

• The scatter on all the data appears to be fairly uniform whether obtained from the pneumatic/mechanical type (DMT, TSC) or the electronic type (LS-CPTU, SBLC) of transducer.

In terms of the effect of measurement details on recorded data, Fig. 5.14(b) compares the $p'_0$ and $p'_1$ (effective DMT pressures) values obtained using the standard Marchetti and UBC research dilatometers. As mentioned earlier, the standard DMT measures lift-off at 0.05 mm (and then corrects to zero movement by linear back-extrapolation) whereas the research DMT measures true lift-off. The difference in the two values of $p'_0$ is significant and illustrates the importance of small movements on measured pressures. On the other hand,
very little difference in the effective $p_1$ pressures can be seen. This would further suggest that by using a true lift-off pressures, reasonable estimates of $K_0$ could be obtained using the $p_2$ reading as an indication of the excess pore pressure, i.e.

$$K_0 = K_{DMT} \quad (5.7)$$

$$K_{DMT} = \frac{p_0 - p_2}{\sigma'_v} \quad (5.8)$$

In soft clays this expression might be used to replace the present definition of $K_D$ and the required semi-empirical correlation to $K_o$ (see Appendix A, Section A.4.2). The data presented in Fig. 5.15 for stiff clay would suggest that this relationship is not valid in overconsolidated materials.

The results obtained from measurements in the overconsolidated soil at Strong Pit are shown in Fig. 5.15. The initial effective lateral stress determined using the total stress cells is reasonably consistent with depth considering the fairly heterogeneous nature of the soil. Both $p'_o$ and $p'_1$ values from the DMT show the same consistency. The difference between $p'_o$ and $p'_1$ suggests that large excess pore pressures are generated during membrane expansion. The effective lateral stress determined by the LS cone is lower than that from the DMT ($p'_o$) and generally higher than the results of the TSC measurements. This is, however, a function of the unreliable TSC measurements of pore pressure as the two values could be expected to be very similar. For all the in situ testing probes installed at Strong Pit, the measured initial lateral effective stress are significantly higher than obtained at the other sites. Furthermore the ratio of excess pore pressure to effective stress is lower for the stiff overconsolidated material than
Figure 5.15 Initial effective lateral stress results from Strong Pit.
for the soft normally consolidated soils discussed earlier. It would appear that the level of post-penetration effective stress in fine grained materials may be related not only to soil type but also to the stress history of the material.

5.3 Measured Pressures in Coarse Grained Soils

The results of penetration tests in granular soils at two of the detailed research sites are considered in this section. Penetration of self-boring or full-displacement probes in granular soils can usually be considered as a drained process provided the proportion of fines in the material is not high enough to impede drainage. For the two research sites considered here, namely McDonald Farm and Laing Bridge South, penetration can be considered as a drained event with little or no excess pore pressure being generated (Le Clair, 1988; Gillespie 1990). However, a brief review of measured pore pressures will be given for the sake of completeness, since any evaluation in terms of effective stress requires both the total stress and pore pressures to be known.

5.3.1 Penetration Pore Pressures in Sands

McDonald Farm

The penetration pore pressures during LS-CPTU for both the UBC and UCB (Berkeley) cones are shown in Fig. 5.16. In Fig. 5.16(a) the pore pressures measured at four locations on the cone are presented; data for the location are presented even though they are very variable and subject to filter compressibility effects especially in the dense sand layers. The main body of sand is present between 4 m and 10 m, being overlain by a soft silty fill
Figure 5.16 Penetration pore pressures in granular soils at McDonald Farm using different lateral stress cones.
and underlain by a transition zone before entering the soft clay silt presented earlier at about 14.5 m. The presence of silty layers within the sand and the fines content of the sand itself is evidenced by the slightly positive \( u_2 \) pore pressures although this difference may be the result of differing ground water conditions at the time of testing. However, by the time the \( u_3 \) pore pressure is recorded, the pore pressure is hydrostatic and remains so for the \( u_{LS} \) location. Between \( u_3 \) and \( u_{LS} \) the pore pressure is remarkably constant. This is consistent with previous measurements in clay although, of course, the magnitude of the two pore pressures in clay and sand is very different. In terms of hydrostatic, no excess pore pressure is generated at or above the \( u_3 \) location. The results in Fig. 5.16(b) obtained using the UCB (Berkeley) LS cone suggest pore pressures negative of hydrostatic throughout much of the sand. The two soundings with the two different LS cones are about 2 m apart so some variation in the upper 5 m may be expected since the fill is very heterogeneous. Below 5 m the sand is acceptably homogeneous. From earlier results (Fig. 5.1), it is apparent that in the clay underlying clay silt the lower pore pressure of the Berkeley cone corresponds well with the \( u_2 \) measurements of the UBC cone and that the upper value agrees with the \( u_3 \) measurement. The initial response on entering the clay silt (Fig. 5.1) suggests that all the pore pressure measuring systems are well saturated. The difference between \( u_3 \) and \( u_{lower} \) in the sand may result from the difference in location of the two sensors and also partly from the height of the filter at each location. Below about 9 m the Berkeley cone data are consistent with the UBC data, i.e. \( u_3 = u_{lower} \) and \( u_2 = u_{upper} \). Furthermore, the UBC LS cone data are believed to be correct since good agreement occurs with results obtained using piezocones over the last 10 years at this well established site.
As suggested earlier, the unloading of the soil as it passes the cone tip causes large normal stress reductions and associated pore pressure changes. In stiff, low permeability clays, the large stress reductions can give rise to negative excess pore pressures. In coarser materials, the degree of pore pressure change during unloading will be a function of the interplay between rate of generation and rate of dissipation of excess pore pressure. The slightly positive excess (w.r.t. hydrostatic) at the \( u_2 \) location may be a function of the gradient during unloading since the maximum unloading occurs just behind this location. It is likely, however, that since the \( u_2 \) data is from a sounding conducted in October and the rest of the data taken during July, a higher groundwater condition existed - the difference between \( u_2 \) and \( u_3 \) in Fig. 5.16(a) can be accounted for by a 1 m to 1.5 m difference in depth to the water table.

Hence the \( u_2 \) and \( u_3 \) pore pressures are hydrostatic. Whatever the unloading and/or shear generated stress changes that may have occurred, the associated pore pressure changes have been dissipated before they could be detected by the cone. This is not so for the UCB data. The explanation would appear to lie in the pore pressure measurement system itself. The filter on the Berkeley cone is smaller than that used on the UBC cone and has a lower material permeability. If the transmissivity of the filter system is lower than that of the ground then the measured dynamic pore pressures will depend on the cone characteristics rather than the soil behaviour. In effect the flow of fluid into or out of the filter is restricted and so the negative pressure induced by unloading cannot restore itself to the hydrostatic value existing around the cone. The effect on the \( u_{upper} \) response would be even more pronounced since the size of the porous filter is very small. This is apparent in Fig. 5.16(b). With depth, as the difference between the hydro-
static pressure outside the cone and the lower negative excess in the pore pressure system increases the rate of equilization would increase until correspondence between the two measurements was achieved. This would occur presumably at different depths for the two pairs of sensors due to the different filter areas, the smaller element requiring more time. This is also suggested by Fig. 5.16(b).

Self-boring pressuremeter tests were also performed at this site. The SBPM does not have a pore pressure transducer but rather measures the effective stress by means of a diaphragm transducer fixed on the membrane. The pore pressure is obtained by taking the difference between the effective stress (outside the membrane) and total stress (inside the membrane) measurements. Unfortunately, problems with the transducer were encountered and no effective stress measurements were possible. From the above discussion, however, it is likely that the pore pressures around the lantern were hydrostatic both during insertion of the probe and initial inflation of the membrane to give lift-off. The former is corroborated by the results of self-boring load cell (SBLC) tests.

As would be expected, all $p_i$ pressures measured by the standard DMT would suggest a hydrostatic pore pressure around the blade. Results presented by Tsang (1987) and Campanella and Robertson (1991) suggest that the entire inflation and deflation stages of the dilatometer test in the granular soils at this site are drained with pore pressures remaining hydrostatic throughout.

Laing Bridge South

The pore pressures measured at Laing Bridge South using the UBC LS cone and the SBLC are presented in Fig. 5.17. No Berkeley LS cone data is avail-
Figure 5.17 Penetration pore pressures at Laing Bridge South.
able for this site since the cone malfunctioned during the tests performed at McDonald Farm. It was not possible to repair the cone in order to continue the programme of testing originally planned. The results in Fig. 5.14 comply with the observations made previously for the McDonald Farm site and will not be discussed further. The SBLC data also are in agreement with the SBPM data in the sense that the installation pore pressures are all hydrostatic.

5.3.2 Lateral Stress Measurements in Sands

The development of lateral stress cones was based on the idea from calibration chamber testing that sleeve friction measurements were indicative of variations in the lateral stress acting on the shaft of a penetrometer (this is discussed in detail in Appendix A, Section A.6). The results of limited chamber testing by Huntsman (1985) indicate that the lateral stress in sand measured just behind the tip is lower than that measured some distance up the shaft. This would indicate variable friction sleeve measurements over some initial length of the shaft and that the measurements would be sensitive to design geometry of the equipment. Using the data from the two LS cones used it is possible to compare sleeve friction values at various distances behind the tip. The comparisons for the Laing Bridge South and McDonald Farm sites are shown in Fig. 5.18. At Laing Bridge South, the two $f_s$ values compare very well. At McDonald Farm the three $f_s$ profiles are very similar although increased scatter results from the more heterogeneous conditions at this site. The sleeve friction values would then suggest a reasonably uniform distribution of lateral stress along the shaft. This is contrary to the results from calibration and field testing (Huntsman, 1985; Jefferies et al., 1987). The good agreement in the $f_s$ values is therefore a direct consequence not of the in situ soil state but rather of equipment characteristics.
Figure 5.18 Comparison of friction sleeve measurements in sand at various locations behind the penetrometer tip.
For this reason, methods to evaluate in situ stress conditions based on $f_s$ measurements can only be used for the types of cones for which the correlations were established. Application of this type of correlation to other cones where the friction sleeve may be of another size and located in a different position may give erroneous results.

McDonald Farm

The results of lateral stress cone tests are shown in Fig. 5.19(a) compared with self-boring and seismic cone pressuremeter data and in Fig. 5.19(b) compared with dilatometer data. The UCB LS cone can measure $\sigma_{LS}$ at two locations; at 1D and 7.5D behind the tip. The UBC LS cone measures $\sigma_{LS}$ at 16.6D behind the tip. During the first sounding the UCB cone LS sensor at 1D collapsed during the initial push in sand so no data were obtained at this location. For a subsequent test with the UCB LS cone a 290 mm tip extension was placed on the front of the cone so that the LS sensor previously at 7.5D was now located at 15.6D behind the tip. In this way it would have been able to cross check the LS sensor at 16.6D on the UBC LS cone. The sensor previously at 1D was now at 9D with the extension in place. (The LS sensor at 1D which failed during the first sounding was repaired and thought to be functioning.) Unfortunately during the subsequent sounding the lower LS sensor did not function and after several metres penetration the upper sensor also collapsed. This time it was not possible to repair the UCB LS cone and no further tests were performed.

The $\sigma_{LS}$ measurements from the two LS cones indicated in Fig. 5.19(a) are very different. The UBC data give $\sigma_{LS}$ values close to the total vertical stress whereas for much of the profile the UCB data lie below the hydrostatic pore pressure distribution. Also shown on Fig. 5.19 are the results of SCPM
Figure 5.19 Lateral stress measurements in sand at McDonald Farm.
and SBPM tests which are reasonably consistent with the UBC LS-CPTU data. The SBPM data presented in this figure are considered to be the best available data for this site for determining reference lateral stresses. Several SBLC tests were also performed at this site. In every test, the lateral stress measurements on both sensors suggested that considerable disturbance had occurred during installation, i.e. \( \sigma_h = u_0 \). The disturbance was thought to arise due to the use of a cutting head for the self-boring operation. Considerable difficulty was encountered when trying to penetrate the SBLC probe to depths greater than 5 m. The same problems were also encountered using the SBPM. The SBPM was installed by self-boring but using high pressurized jets of drilling fluid to cut away the in situ soil and transport it to the surface. The use of the jetting technique and specially adapted jetting heads (described in Chapter 3) allowed the SBPM to be installed to depths of 13 m.

The main difficulty of advancing a SBPM in sand is the high skin friction that builds up due to the large surface area of the probe. (For a 100 mm diameter probe, 1 m long with 50 kN/m\(^2\) friction, the pushing force required to advance the probe (by self-boring) is a minimum of 1.5 tonnes.) In dense sand this figure increases dramatically and is supplemented by high loads on the PM cutting shoe. This latter point is evidenced by flaring-out of the PM cutting shoe and has resulted on several occasions in the deeper dense sand layers at both research sites.

The UBC SBPM was installed using standard rods used for CPTU testing. These rods function well when subject to axial loading. When pushing on SBPM in an oversized hole at high axial loads the rods are free to buckle. Lateral deflection of the rods depends on the axial load and support given by the walls of the prebored hole. If little or no support is available the
rods can fail in bending. This occurred for the last SBPM test at McDonald Farm with the probe at 12 m below ground level. Various procedures were attempted to retrieve the probe but without success. A new SBPM probe has since been designed and built at UBC and heavier, thicker-walled BWL rods will be used to push the probe during both the self-boring operation and full-displacement installation. On Figure 5.19(a) the lift-off pressures from the SBPM can be seen to agree remarkably well with the UBC LS-CPTU results. The UCB LS-CPTU on the other hand appears to be on the low side. One reason for this may be the dimensional tolerances of the lateral stress sections on the UCB cone. Measurements with a vernier gave the following diameter values:

| UCB Berkeley LS Cone - Diameter Variations at Lateral Stress Section |
|--------------------------|--------------------------|
| At cone tip               | 1.410"; 1.408"; 1.409"   |
| At lower LS sensor        | 1.369"; 1.368"; 1.379"   |
| At upper LS sensor        | 1.395"; 1.386"; 1.357"   |

For the UBC cone, the vernier gave identical diameter values at the tip and LS section. The variation in diameter of the UCB cone is thought to result from the flexible form of the LS sleeve; three arciform sections are joined and held in place by a pliable polyurethane compound. This would account for the variations in diameter recorded above. It might also be the reason for the low \( \sigma_{LS} \) values obtained in sand at McDonald Farm. The slightly undersized probe would give rise to stress relief and reduced lateral stress acting on the shaft at and close to the location of the change in diameter. This should also apply to the SCPM data since the PM section is 0.4 mm smaller (in diameter) than the remainder of the shaft. While some of the SCPM lift-off
values are very close to \( u_0 \), many of the results would suggest that stress relief has not occurred. In the case of the SCPM, the effect of the smaller-sized PM section is complicated by intrusion of sand behind the lantern giving rise temporarily to an oversized probe. This problem has been discussed by Howie (1991). The soil heterogeneity at McDonald Farm may mask to some extent the comparative response of the probes.

In Figure 5.19(b) the LS cone data are plotted alongside DMT data. In agreement with the earlier suggestion that the DMT \( p_0 \) values are sensitive to the method of measurement, \( p_0 \) (0.05 mm back extrapolated) is consistently higher than \( p_0 \) (first lift-off). The difference in the two appears to be fairly constant with depth.

Laing Bridge South

The results of tests performed with full-displacement probes are presented in Fig. 5.20(a) and generally conform to the comments above for McDonald Farm. Due to greater soil homogeneity at this site, trends in the data are however much clearer. The SCPM results as before are in good agreement with the LS-CPTU profile. In addition, results from the newly built UBC pressure-meter obtained by the 1990 graduate course CIVL 577 are shown on the figure. The SBPM has been converted to a full displacement probe by placing a cone tip in front of the PM section in a similar manner to that described by Hughes and Robertson (1985). The agreement with the LS-CPTU data is encouraging although pore pressure effects may affect the effective stress profile. This is discussed later in this chapter. Again, the \( p_0 \) DMT values are much higher than any of the results from full-displacement cylindrical probes.

After obtaining poor results at McDonald Farm with the SBLC using a rotary cutter and jetting installation technique, the equipment was modified
Figure 5.20 Lateral stress measurements in sand at Laing Bridge South.
to enable installation by jetting only. Based on experience with a self-boring PM this was considered the best technique for minimizing disturbance in sand. The vibrations caused by the cutter motor are transmitted to the soil via the probe and result in disturbance. The SBLC data using jetting are indicated in Fig. 5.20 and compared to the other PM-type probe data. The results are only considered reasonable to a depth of about 6.5 m. Below 6.5 m the high pushing force required to advance the probe results in some deviation from the vertical with anomalously high pressure readings being recorded on one side of the probe. On recovery of the probe, it was apparent that the cutting shoe had made contact with some gravel and had flared out at its edge on one side. This led to increased inclination of the probe.

The scale of the plot in Fig. 5.20(b) also illustrates the unloading the soil experiences during SCPM tests due to the smaller PM section. The increase in FDPM lift-off is greater than that associated with the SCPM data. If no unloading occurred, the two data sets should be very similar.

5.3.3 Initial Effective Lateral Stress in Sand

The calculated effective lateral stress profiles in sand for each of the in situ probes at McDonald Farm are indicated in Fig. 5.21. Figure 5.21(b) presents the same data as Fig. 5.21(a) except that the DMT data has been left off in order to see the detail at the low end measurements. Similarly the calculated profiles for Laing Bridge South are shown in Fig. 5.22. At both sites the following trends are evident:

- The effective pressure profiles \( p_0' \) obtained using the dilatometer are much higher than those obtained using the cylindrical probes. The differ-
Figure 5.21 Effective lateral stress profiles in sand from measurements at McDonald Farm.
Figure 5.22 Effective lateral stress profiles in sand from measurements at Laing Bridge South.
ence between the DMT results and the remainder of the data increases as the relative density of the sand increases.

• In general terms, the results shown in Figs. 5.21(b) and 5.22(b) are in very good agreement. That is, the effective stress profiles obtained from the self-boring and full-displacement probes at both sites show similar relative behaviour. At Laing Bridge South, the stresses measured using the LS cone are slightly higher than those recorded at McDonald Farm.

The measurements obtained during LS cone profiling permit the soil-steel interface friction angle, \( \delta \) to be evaluated. The relationship between the measured sleeve friction \( (f_s) \), effective lateral stress \( (\sigma'_{LS}) \) and \( \delta \) is given by:

\[
f_s = \sigma'_{LS} \tan \delta \quad (5.9)
\]

or

\[
\delta = \tan^{-1}(f_s / \sigma'_{LS}) \quad (5.10)
\]

The profiles of \( \delta \) at both McDonald Farm and Laing Bridge South are shown in Fig. 23. While the range of \( \delta \) values is large the magnitude is considered to be in reasonable agreement with published data. Tomlinson (1981) suggests from his experience with steel piles in sand that a soil-steel friction angle of 20° is appropriate. The data are also in agreement with results suggested from a literature review of measured soil-steel friction coefficients (Potyondy, 1961) together with measurements in ring shear tests performed at UBC (Rinne, 1989). It is also apparent that significant scatter in laboratory friction coefficients also exists.

The scatter in the field data is accentuated by the errors associated with sleeve friction measurements. The range of \( f_s \) measured in the field is very small in relation to the calibrated full-scale and errors of \( \pm 50\% \) may
Figure 5.23 Soil-steel interface friction angles in sand from LS-CPTU.
arise. As stated earlier, the measured lateral stress, $\sigma_{LS}$ has a sensitivity of ±7 kPa. While the data confirm that the values of the parameters being measured during LS-CPTU profiling are realistic, improvements in sensitivity and signal noise reduction are required in order to provide repeatable estimates of parameters such as $\delta$ which are very sensitive to small variation in measured data. As stated previously the LS cone lateral stress module is being modified to attempt to reduce signal noise and improve the calibration characteristics. Data are not presented for the clay sites since both $f_s$ and $\sigma_{LS}$ are very small; hence signal noise is a significant proportion of the measured response.

5.4 Dissipation of Initial Pressures

The data presented so far relate to the initial lateral stresses and pore pressures measured either during penetration (LS-CPTU) or immediately after penetration is halted (DMT, TSC). In some instances, varying degrees of time may pass before the first stress and pore pressure measurements are made (SBPM, SCPM, SBLC). Subsequent to the initial measurements, the excess pressures generated during installation of the measuring device start to decay, the rate of which depends on many factors including the method of installation and soil type. In terms of the lateral stress acting on the probe, little information is available concerning the post-penetration changes that may occur. The dissipation of excess pore pressure, especially for the case of cone penetration testing, has received considerable attention over recent years. These two aspects are considered below.

5.4.1 Dissipation of Excess Pore Pressures

The penetration pore pressure measured at any location on a probe can be divided into two components:
• in situ equilibrium value, $u_0$, which is controlled by the local groundwater regime.

• excess pore pressure generated during probe installation, $\Delta u$, which is a function of both the soil and cone characteristics.

Thus:

$$u_x = u_0 + \Delta u_x \quad (5.11)$$

where the subscript $x$ refers either to the type of probe being used or the location of the pore pressure element on a particular probe. It is the decay with time of the excess porewater pressure that provides information concerning the flow characteristics of a soil. In order to evaluate the dissipation of the generated excess pore pressure, penetration of the probe is halted and the reduction or decay of the excess with time is recorded. The change in pore pressure can be plotted against log time or square-root time depending on the type of analysis to be performed. Either the excess pore pressure:

$$\Delta u = u - u_0 \quad (5.12)$$

or the normalized excess pore pressure:

$$U(t) = \frac{u_t - u_0}{u_i - u_0} = \frac{\Delta u_t}{\Delta u_i} \quad (5.13)$$

where:

$U(t) = \text{normalized excess pore pressure at time } t$
\[ u_t = \text{measured excess pore pressure at time } t \]
\[ u_i = \text{initial excess pore pressure when penetration is halted (} t=0 \text{)} \]

can be plotted against time. \( U(t) \) therefore varies between \( l(t=0) \) and 0 when the excess pore pressure has completely dissipated and \( u_t = u_o \).

Generally, the interpretation of pore pressure dissipation has been directed towards the evaluation of in situ flow characteristics. Theoretical studies of the dissipation of excess pore pressure during cone penetration testing have been presented by many researchers. The methods of Torstensson (1975), Baligh and Levadoux (1986), Gupta and Davidson (1986) and Houlsby and Teh (1988) are considered to provide good approximations to field conditions. In the most comprehensive of these studies, Baligh and Levadoux (1986) concluded the following based on comparison of analytical studies with field measurements:

- that the simple uncoupled solutions provide reasonably accurate predictions of the dissipation process, although even in soft normally consolidated soils the actual pore pressure distribution may differ noticeably from the theoretical solution. Total stress changes may, however, affect pore pressure dissipation especially for locations close to the tip.
- that consolidation is taking place predominantly in the recompression mode for dissipation ratios of less than 50%
- that the initial distribution of excess pore pressures around the penetrometer influences the dissipation process
- that the interpretation is sensitive to estimates of \( u_i \) and \( u_o \) especially during the early stages of dissipation.

These results were obtained for soft normally consolidated to lightly overconsolidated Boston Blue Clay. In moderately to heavily overconsolidated
soils (OCR > 4), the pore pressure gradients around the tip can be very large (Robertson et al., 1986; Sully et al., 1988) and the existing solutions provide a poor estimate of the initial distribution of pore pressure. Difficulties in predicting the initial distribution of excess pore pressure around the probe represent one of the major drawbacks of the theoretical solutions.

During installation of full-displacement probes large excess pressures may arise due to the strains imparted to the soil and it may be necessary to wait for considerable periods of time for the excess pressures to dissipate. For some test methods this may be acceptable and form an integral part of the test procedure (i.e. TSC), whereas for others, not only is it inconvenient but also contradicts the basic philosophy of the test itself, i.e. DMT, LS-CPTU, SBPM.

**LS-CPTU**

The results of LS cone profiling provide measurements of both total lateral stress and pore pressure about 17D behind the tip. The disadvantage of measuring pore pressures behind the tip is that long dissipation times are required to achieve acceptable degrees of pore pressure decay. Furthermore, at locations behind the tip, the pore pressure may initially increase prior to decaying. The increase can result from two conditions:

- from poor saturation of a pore pressure measurement system, and
- due to the high gradient of pore pressure around the tip, drainage from the tip (high pore pressure area) to the zone behind the tip (lower pore pressure area) occurs, the rate of which is determined by the soil permeability. The effect of the flow around the tip on the pore pressures measured on the shaft varies according to the soil stiffness and strength, factors which also determine the pore pressure gradient.
A third possible explanation for this increase is the presence of the maximum penetration pore pressure at some point away from the shaft. This is not suggested by the strain path and cavity expansion approaches to interpretation of penetration pore pressures. However, since neither of the two methods can model correctly the response of stiff overconsolidated clays where unloading effects dominate, this is not considered conclusive. Soil stiffness and strength would control the magnitude of the pore pressure and also the distance of the maximum value from the shaft. Hence for soft clays the maximum would be located on the shaft itself. This concept requires further study. The forms of pore pressure dissipation curves are illustrated in Fig. 5.24 for data obtained at Strong Pit. Only the $u_1$ pore pressure shows initial dissipation. In fact, unloading of the bearing stress causes a sudden decrease in $u_1$. The data in Fig. 5.24 have been normalized and replotted in Fig. 5.25(a). The following points are of interest:

- the $u_1$ value drops to 75% of its penetration value on halting penetration. Thereafter the $u_1$ pore pressure dissipates in a manner similar to the predicted theory
- the $u_2$ value increases for about 20 seconds before decaying
- the $u_3$ value increases for a longer period of time before finally dissipating. The longer initial increase reflects the greater distance of the $u_3$ filter behind the tip.

The normalized curves in Fig. 5.25(a) cannot be used for interpretation as they do not conform with the theoretical dissipation relationship. In an attempt to provide a theoretical normalized distribution, the pore pressure dissipations have been modified such that:
Figure 5.24 Pore pressure dissipation in stiff clay at Strong Pit.
Figure 5.25 Normalized pore pressure plots for Strong Pit data (a) uncorrected and (b) corrected for initial distribution effects.
• $u_i$ is taken as the maximum value, $u_i^{(\text{max})}$ attained during the post-penetration increase.

• the time at which $u_i^{(\text{max})}$ occurs is taken as $t=0$ on the x-axis.

The resulting normalized curves are shown in Fig. 5.25(b). The method for obtaining the new set of curves is considered to be theoretically correct and is applicable for any soil type. Figure 5.25(b) does highlight the problem in interpretation of pore pressure dissipation data at locations behind the tip. The time for 50% dissipation at the $u_i$ location is around 1100 seconds (18 min.) which increases to 2300 seconds for $u_2$ and 10,000 seconds for $u_3$.

It is also possible to evaluate the dissipation data of Fig. 5.24 using a back-extrapolation technique on a square-root time plot. In the root-time plot, the dissipation after the peak caused by redistribution of pore pressure initially depicts a straight line which can be back-extrapolated to $t=0$ to obtain $u_i(t=0)$ for the modified dissipation curve (Fig. 5.26a). The data from Fig. 5.24 are replotted in Fig. 5.26(b) to illustrate this point. The advantage of the root-time plot is that the initial straight line portion can be extrapolated to 50% pore pressure reduction if short dissipation periods are used in the field and measured data are not available (Fig. 5.26c). Alternately, the initial linear slope in the normalized pore pressure - root-time plot can be analyzed to provide estimates of $c_h$ using the theoretical approach suggested by Teh (1987). This approach has been evaluated by Robertson, Sully, Woeller, Lunne and Powell (1991) and preliminary results suggest that it provides $c_h$ values consistent with the log time method.

For the $u_{LS}$ location, the dissipation rates will be longer than for the $u_i$ location. Furthermore, at locations remote from the tip, saturation of
Figure 5.26 Details of the root-time method for evaluating pore pressure dissipation data.
the filter system is a major problem, irrespective of attention to detail during probe preparation. At Lr. 232 St., where the soils are less overconsolidated and soft, unloading of $u_4$ when penetration was halted was not seen to be a problem. However, as for the Strong Pit data, excessively long periods were required for the $u_3$ and $u_L$ pore pressures to stabilize and start to dissipate.

To avoid the long dissipation times associated with the dissipation of excess pore pressures, an attempt was made to use curve fitting techniques to extrapolate the initial short-term decay to the long-term condition. The program used was that presented by Pacific Geoscience Centre and developed initially for interpreting seabed temperature decay data from thermistor measurements. The initial dissipation of excess temperature values over a limited period (about 10 min.) is fitted using a Bessel function approximation and the resulting equation used to evaluate the final in situ ground temperature. Similar governing equations also determine pore pressure dissipation and the technique was modified and applied to in situ dissipation data. Unfortunately the method did not work very well and significant errors in the final predicted equilibrium pore pressures were obtained. This was realized by comparing the curve-fitting results with long duration field dissipation results. The errors arise from similar problems associated with the theoretical dissipation solutions. The Baligh and Levadoux (1986) solution also uses a Bessel function decay curve to predict dissipation of excess pore pressures. These curves have been shown to be very idealized when compared to field measurements even for soft soils (Gillespie, 1990). Baligh and Levadoux (1986) have also shown theoretically that the idealized dissipation curves are somewhat sensitive to errors in $u_4$ and $u_L$, especially at low degrees of dissipation.
The error in evaluating and extrapolating short-term dissipation curves to the long-term condition can be illustrated by comparing $c_h$ values predicted at each level of dissipation. The evaluation is based on the Torstensson predictions of $c_h$ using field data from soft Onsoy clay (Soares et al., 1987). The coefficient of consolidation, $c_h$, at varying degrees of consolidation is normalized with respect to the value obtained at 50% dissipation. The variation of $c_h(U\%)/c_h(50\%)$ is shown in Fig. 5.27 and indicates that short term dissipation data can give rise to anomalous results if extrapolated to predict long-term conditions. The error involved would depend on both soil type and measurement location.

**DMT and TSC**

Pore pressures can be measured with the dilatometer by means of the pressure at membrane closure (Robertson et al., 1988) and for the total stress cells a pore pressure measuring system is incorporated into the blade.

Once total stress cells have been installed in the ground they are usually left for a period of time so that the excess pore pressures generated during installation, $u_{TSC}$, have time to dissipate and return to the in situ equilibrium value. One major problem with the measurement of pore pressures with the TSC is that it is very difficult to ensure saturation of the measurement system. The internal volume of the pore pressure chamber is relatively large and thus subject to appreciable compressibility effects if not completely saturated. Saturation is not aided by the probe design. Details of the spade cell are shown in Fig. 3.10. The porous filter is housed on the pressure cell support plate and is connected to the pneumatic transducer, located 0.43 m behind the filter by means of thick-walled nylon tubing. The tubing and filter are saturated, according to the manufacturer's
Figure 5.27 Variation in $c_h$ values at various stages along predicted dissipation curves

specifications, by loosening coupling A and injecting water into the lines until it exits through the ceramic porous filter. The bleed screw by the filter is loosened to release the initial air volume. On tightening the bleed screw, the injected water then flows through the filter. By this method, good saturation of both filter and tubing could not be achieved.
New filters of polypropylene were machined at UBC for use with the stress cells. These were saturated under vacuum in glycerin prior to placing on the cell. With the pressure cell tip upwards in a vise, de-aired glycerin was injected from the filter location down into the tubing which is connected to the transducer. Loosening the coupling A permitted air escape and penetration of the glycerin to the transducer cavity. Once the tubing was considered to be saturated and air-free, the sides of the filter holder were smeared with glycerin, the filter removed from the glycerin bath and installed. The TSC was then installed in the ground. No vacuum de-airing was used. During installation, the filter is located below the pressure transducer and the column of glycerin can flow under gravity out of the filter. This may have caused some loss of saturation as the blade was pushed initially in an oversized hole, but the effects could not be quantified.

The pore pressures measured with the stress cells were very variable and erratic in their initial dissipation. The variability of the initial pore pressure measurements has been discussed in the earlier sections. An example of the worst case is shown in Fig. 5.28. The equilibrium pore pressure at this depth for this site is between 0 and 10 kPa. As expected, the TSC records a high initial post-penetration pore pressure which decays quickly to about 25 kPa. This value remains essentially constant to day 47 after which it increases to 75 kPa, measured on day 115. Thereafter the pore pressure oscillates but never reaches a value close to the $u_0$ of 10 kPa. Air entry into the pore pressure measurement system is thought to be the reason for the response shown in Fig. 5.28. The effect of the pore pressure variation on the calculated effective horizontal pressure can be seen to be significant (Fig. 5.28).
Figure 5.28 TSC measurements in stiff clay at Strong Pit.

For some of the cells, very good response was obtained (Fig. 5.29).

Extensive dissipation tests using the DMT were not performed during this study. The evaluation or extrapolation of short term data to long term conditions is considered more problematical than is the case for CPTU pore pressures. Dissipation of the DMT pore pressure ($p_2$ reading) occurs at a rate comparable to that at the $u_3$ CPTU location. Reliable curve fitting and extrapolation are not considered possible with presently available equipment. However, the UBC research DMT may provide more consistent data in this respect.
5.4.2 Dissipation of Initial Lateral Stresses

As for the measured pore pressures, the initial measured lateral stresses can be monitored with time to evaluate any post-penetration changes. The monitored changes for the TSC and LS-CPTU data are presented below.
Total Stress Cell

A typical data set from TSC measurements is presented in Fig. 5.29. At this test depth, the measured pore pressures are considered to be reliable. The variation in blade pressure and pore pressure with time are indicated. The effective horizontal stress is obtained by subtracting the net porewater pressure \( u_{TSC} \) from the temperature corrected net blade pressure, \( \sigma_{TSC} \). After a long dissipation period it can be seen that \( u_{TSC} \) is equal to \( u_0 \). Identical type TSC stress dissipation trends have been obtained for the other two sites where TSC's have been installed.

The dissipation of the total stress data has been evaluated using a power function of the form:

\[
\sigma_{TSC}(t) = \alpha_1 t^{-\beta}
\]

where

- \( \sigma_{TSC}(t) \) is the time dependent stress measured using the TSC
- \( \alpha_1 \) is the value of \( \sigma_{TSC}(t) \) at \( t = 1 \)
- \( t \) is the time after installation
- \( \beta \) is the power which controls the rate of stress relaxation.

The power relation was found to provide the best representation of the stress relaxation. Each data set was evaluated and \( \alpha_1, \beta \) values obtained for both Strong Pit and Lr. 232 St. The \( \alpha \) and \( \beta \) values are plotted in Fig. 5.30 as a function of depth. The depth dependence of both parameters is remarkably linear. It is also interesting to note that:

- \( \alpha_1 \) is greater for the stiff clay at Strong Pit than for the soft clay at Lr. 232 St. This is intuitively correct since larger pressures will
\( \alpha_i \) = initial TSC stress reading at \( t=1 \)
\( \beta \) = power decay constant in Eq. (5.14)

Figure 5.30 Variation of \( \alpha_i \) and \( \beta \) for TSC results at clay sites
develop due to full-displacement penetration in stiff clays rather than in soft clays.

- $\beta$ for Strong Pit is more largely negative than for Lr. 232 St. which implies a larger post-installation reduction in $\sigma_{TSC}$ for stiff clay and may indicate also a larger degree of disturbance. Again this would appear correct since due to the larger $\alpha_i$ values in stiff clay, the $\beta$ value will depend to a large extent on the dissipation of the excess pore pressure. Some redistribution or relaxation of stress will also occur. In soft clay, the effect of stress relaxation may be such that the final corrected lateral stress obtained from the TSC may well be very close to the in situ horizontal stress. In stiff clay some amplification of $\sigma_h$ will certainly remain.

Lateral Stress Cone

Similar to the results obtained with the total stress cells, it was also possible to interpret the lateral stress cone dissipation by means of equation (5.14). A typical dissipation with fitted power decay curve is illustrated in Fig. 5.31. As for the TSC data $\alpha_i$ was found to vary with depth whereas $\beta$ was more or less constant at both Strong Pit and Lr. 232 St. Values of $\alpha$ for the LS cone are similar to $\alpha_{TSC}$, whereas $\beta_{LS}$ is much lower than $\beta_{TSC}$. However at Strong Pit $\alpha_{LS}$ is approximately twice $\alpha_{TSC}$ and $\beta_{LS} < \beta_{TSC}$. The $\alpha$ and $\beta$ values obtained with the LS cone are in agreement with the relative magnitudes obtained with the TSC at both the sites.

The difference between the $\beta$ values for the TSC and LS-CPTU data results primarily from the degree of dissipation of excess pore pressures. For the TSC results, the excess pore pressure has completely dissipated whereas for the LS-CPTU data very little pore pressure change has occurred. Based on
Strong Pit
LS cone dissipation
Depth : 3.05m
\( \sigma_{LS} \) (initial) = 543 kPa
\( \sigma_{LS} \) (final) = 415 kPa

Figure 5.31 LS cone initial stress dissipation at stiff clay site.

this, the difference in the \( \beta \) value may provide an alternative form of obtaining the fully dissipated \( \sigma_{LS} \) value from short-term LS-CPTU measurements. However, this will only give reliable results if \( \beta \) is independent of disturbance caused during installation or if the overall disturbance of the two probes is similar. This seems to be suggested by comparison of the measured pore pressures and lateral stresses presented earlier in this chapter, at least for the stiff OC clay.

For data obtained in sand, no noticeable dissipation of lateral stresses was observed.
5.5 Lateral Stress Oedometer Results

The equipment and test procedures followed with the LS-oedometer have been described in Chapter 3. In all, seven samples were tested from three sites. Two undisturbed block samples were tested from both Strong Pit and Lr. 232 St. while three tube samples were tested from 200th St. The data were interpreted in terms of the $K_0$–OCR relationship given by:

$$ (K_0)_{OC} = \left( (K_0)_{NC} (OCR) \right)^m $$

(5.15)

The results of the tests performed and interpreted according to Eq. (5.15) are presented in Table 5.1.

It was apparent during the testing that a temporary shift in the lateral stress zero reading was occurring and this was considered to result due to

<table>
<thead>
<tr>
<th>Site</th>
<th>Sample Depth</th>
<th>$(K_0)_{NC}$</th>
<th>$m$</th>
<th>Loading Phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lr. 232 St.</td>
<td>2.0</td>
<td>0.560</td>
<td>0.453</td>
<td>First unload</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.557</td>
<td>0.441</td>
<td>First unload</td>
</tr>
<tr>
<td>Strong Pit</td>
<td>1.5</td>
<td>0.579</td>
<td>0.598</td>
<td>Initial reload</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.528</td>
<td>0.364</td>
<td>First unload</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.523</td>
<td>0.397</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>0.600</td>
<td>0.559</td>
<td>Initial reload</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.532</td>
<td>0.358</td>
<td>First unload</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.520</td>
<td>0.397</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>2.19</td>
<td>0.470</td>
<td>0.370</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>2.23</td>
<td>0.570</td>
<td>0.364</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>7.25</td>
<td>0.550</td>
<td>0.390</td>
<td>First reload</td>
</tr>
<tr>
<td>200th St.</td>
<td>2.19</td>
<td>0.47</td>
<td>0.370</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>2.23</td>
<td>0.57</td>
<td>0.364</td>
<td>First reload</td>
</tr>
<tr>
<td></td>
<td>7.25</td>
<td>0.55</td>
<td>0.390</td>
<td>First reload</td>
</tr>
</tbody>
</table>
the axial (vertical) friction load on the cell caused by sample consolidation during the load increments. This is described in Chapter 3 and herein termed the residual drift. However due to the test set-up it was impossible to determine the effect of the residual shift at any particular loading step. Only at the end of the test, with the sample completely unloaded could this residual be measured. After the sample was removed from the consolidation ring a second zero reading was taken. The difference between this final baseline and the baseline recorded prior to testing is denoted as the baseline drift during the test duration. The baseline drift was considered to occur proportionately during the test duration so that the overall drift could be distributed throughout the daily readings.

The residual drift which was removed by extracting the sample from the ring is likely to vary throughout the duration of the test and to be dependent on such factors as the level of effective stress in the sample and the direction of loading. As a first estimate, the residual drift correction was applied proportionately to the daily readings. The effect of these corrections are shown on Fig. 5.32.

The exponent of the uncorrected data is \( m = 0.52 \); the best fit line is affected notably by the cumulative error in the baseline at high OCR which occurs at the latest unloading stage in the test. Correcting for both baseline drift and residual drift gives \( m = 0.22 \) whereas only correcting for baseline drift gives \( m = 0.37 \). For all three cases, the \( (K_o)_{NC} \) value only changes vary slightly (0.45 to 0.47). The double correction for baseline and residual drift would appear to overcompensate resulting in a \( K_o \)-OCR relationship much lower than expected. The correction is only applied to the lateral stress and thus it does not consider any reduction in vertical load resulting from increased wall friction. This would thus underestimate the true \( K_o \).
value at each loading stage. However it was not possible to evaluate the suspected vertical load reduction. It was decided to assume that the wall friction generated by the soil on the oedometer ring affected the vertical and horizontal measurements proportionally so that the net effect on $K_0$ was zero (or negligible). In this way, it was only necessary to correct the LS-oedometer data for baseline drift. The results in Table 5.1 have been corrected for baseline drift.
The results from samples recovered at Strong Pit are presented in Fig. 5.33. Due to the stiff nature of the soil which facilitates good sample preparation it was possible to obtain reasonable data during the initial reload of the sample. The data obtained from the first unload cycle and subsequent reload are also shown. Relatively small differences in the first-unload and first-reload $K_0$ value were obtained.

The initial reload data from Lr. 232 St. and 200th St. samples gave very low $K_0$ values which are thought to have resulted from sample disturbance during sample preparation. At Lr. 232 St. the disturbance resulted from the high sensitivity of the soil whereas at 200th St. sample preparation was made difficult by the presence of a large percentage of non-plastic fines and sand. Only initial unload data are presented from these sites (Fig. 5.34).

The relationships developed between $K_0$ and OCR can be used in the field situation to predict $K_0$ with depth if the OCR is known. The application of this technique depends on how representative the laboratory values of $K_0$ are of the field condition, or to what extent $K_0$ changes during aging. This has been discussed previously. Data presented by Jamiolkowski et al. (1985) suggest that $K_0\text{ (lab)}$ increases with time but that the rate of change ($\Delta K_0/\Delta \log t$) is very small ($0.007 \pm 0.002$). Due to the problem of baseline drift and its large effect on the logt-$K_0$ variation, it was not considered appropriate to perform any long-term tests to evaluate the effects of aging on laboratory determined $K_0$ values.

5.6 Stress Dependent Parameters

As discussed in the previous chapters and in Appendix A many soil parameters measured during in situ testing are dependent on the level of effective stress existing prior to the installation of the testing device.
Figure 5.33 LS-oedometer results from test 2 (block sample) - Strong Pit.
(K_0)_{oc} = (K_0)_{NC} (OCR)^m
= 0.56 (OCR)^{0.453}

Data from first unload cycle

Figure 5.34 LS-oedometer results from test 1 (block sample) - Lr. 232 St.
The variation of cone resistance, $q_T$, has been shown to be indicative of stress history variations for the two clay sites studied. In calibration chamber tests on sands, $q_T$ has been directly correlated to the lateral effective boundary stress applied to the sample. The undrained shear strength ratio, $S_u/\sigma_v'$, in clay is another example of a stress history dependent normalized parameter. Profiles of $S_u/\sigma_v'$ for each of the sites have been presented in Chapter 4 and have been shown to be related to the determined OCR variations of each site. Also presented are profiles of the downhole shear wave velocity, $V_s$, determined during seismic cone penetration tests. The variation in $V_s$ would also suggest some stress dependency. However, to evaluate the dependence of $V_s$ on changes in both vertical and horizontal stress, crosshole shear wave velocities are required (or shear wave velocities with differing directions of wave travel and particle movement). A test set-up was designed to allow crosshole shear waves of differing polarization to be generated. The test set-up and procedure are discussed in Chapter 3. The results of the in situ tests are presented below.

5.6.1 Shear Wave Velocity Measurements

Crosshole shear wave velocity measurements were performed at three of the research sites: Lr. 232 St. and 200th St. where an overconsolidated crust becomes normally consolidated at depth. The change to an NC profile is fairly smooth at Lr. 232 St. whereas it is very abrupt at 200th St. (It was hoped that the pronounced stress history variation at 200th St. would prove ideal for mapping using crosshole velocity measurements.) The downhole velocity changes would seem to confirm this expectation (Fig. 4.20). The third site tested was Laing Bridge South, the expectation being that the non-destructive crosshole measurements would be a useful technique for evaluating $K_0$ conditions in sand.
The stress dependency of downhole and crosshole shear wave velocities is discussed in Appendix A, Section A.16.

Lr 232 St.

The measured downhole and crosshole shear wave velocities at Lr. 232 St. are presented in Fig. 5.35. Each data point corresponds to the average of four velocity determinations. The two downhole profiles were performed at the locations of the source cone and first receiver cone used for the crosshole test set-up. A second receiver was used but problems were encountered with data capture and so no results were obtained with receiver R2. As a consequence of this, the crosshole shear wave velocities have been calculated based on the first shear wave arrival (at receiver R1). Campanella, Baziw and Sully (1989) have shown that for the fairly homogeneous conditions at this site very good results can be obtained from this visual technique. Downhole velocities were obtained by the cross-over technique and the cross-correlation method (Campanella, Baziw and Sully, 1989; Campanella and Stewart, 1990). Some scatter exists in the \( V_{S_{DH}} \) values but it is confined to the variable upper 3 m of the profile. Below 4 m the shear wave velocities obtained by both approaches are in good agreement.

The crosshole velocities in Fig. 5.35 have been obtained from two types of source signal. A hit in the up or down direction produces an HV shear wave (horizontal travel, vertical particle movement). The left and right torque hits produce HH shear waves (horizontal travel, horizontal particle movement). The downhole shear waves are VH waves according to this convention. It would seem from Fig. 5.35 that the shear wave velocities obtained from both types of crosshole shear wave are identical.
Velocities determined from cross-over on wave traces

Source location
Receiver 1 location
Up/down hits
Clock/anticlockwise hits

Figure 5.35 Downhole and crosshole shear wave velocity profiles at Lr. 232 St.
Both the downhole and crosshole shear wave velocities indicate a degree of stress level dependence as the determined profiles in the NC portion (d > 5m) showed a linear increase with depth. Furthermore, the high $V_s$ at the surface which reduces with depth to a minimum of about 4 m before increasing linearly with depth gives a similar shaped profile to that obtained with $S_u$ - an accepted indicator of stress history.

**200th St.**

The downhole and crosshole shear wave velocities determined at this site are shown in Fig. 5.36. As mentioned previously the soil at 200th St. is heavily to moderately overconsolidated to a depth of about 5 m. From 5 m the clay silt is slightly overconsolidated (OCR = 2). At depths of about 10 m, the clay is interbedded with dense sand. The shear wave velocities in Fig. 5.36 would seem to individually reflect the stress history variation associated with the above description. The downhole $V_s$ is higher than the crosshole values which again are essentially identical. Two receivers were used at this site so crosshole shear wave velocity determination was possible by both the cross-correlation and the crossover techniques.

**Laing Bridge South**

The data from this site are presented in Fig. 5.37 for both downhole and crosshole shear wave velocities. Much higher variability is associated with these measurements in sand than with those reported earlier for clay. This is especially true for the crosshole measurements. This is unfortunate since it reduces the applicability of shear wave velocity measurements to lateral stress determination, if at all possible, since a statistically acceptable number of tests need to be performed to provide true average estimates of the crosshole velocities.
Figure 5.36 Downhole and crosshole shear wave velocity profiles for 200th St.
Figure 5.37 Downhole and crosshole shear wave velocity profile at Laing Bridge South.
5.6.2 Comparison of Shear Wave Velocity Ratios

As discussed in Appendix A, the ratio of the downhole to crosshole shear wave velocity is thought to be dependent on the lateral stress coefficient (Lee and Stokoe, 1985; Yan and Byrne, 1990). The dependence is a function of the stress parameter used for normalizing the individual velocity measurements. For the three possible stress indices the ratios obtained are:

\[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_A}{C_I} \quad \text{(Mean normal stress method)} \quad (5.16)
\]

\[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_A}{C_I} \left(1 + \frac{K_0}{2K_0}\right)^{nt} \quad \text{(Average stress method)} \quad (5.17)
\]

\[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_A}{C_I} \cdot (K_0)^{-na} \quad \text{(Individual stress method)} \quad (5.18)
\]

where the subscripts A and I denote measurement in the anisotropic and isotropic stress planes, respectively. The \((V_s)_A\) velocity defines both the downhole VH shear wave and the crosshole HV shear wave whereas the \((V_s)_I\) velocity explicitly defines the crosshole HH shear wave. To differentiate the two \((V_s)_A\) velocities, the superscripts DH and XH will be used to denote the downhole and crosshole situations, respectively. For the VH and HV shear waves, if the dependence on stress is the same in the two controlling directions, then the two velocities should be the same, i.e.

\[
(V_s)^{DH}_A = (V_s)^{XH}_A \quad (5.19)
\]

This does not appear to be the case for any of the sites. If anything, it is this ratio which appears to provide the best indication of stress history (Figs. 5.38 and 5.39). However, for Lr. 232 St. and 200th St., it is clearly apparent that,
Figure 5.38 Ratio of crosshole and downhole shear wave velocities from field measurements at Lr. 232 St. and 200th St.

\( (V_S)_A^{DH} \) = VH downhole shear wave velocity

\( (V_S)_A^{XH} \) = HH crosshole shear wave velocity
Figure 5.39 Crosshole shear wave velocity ratios from field measurements.
The data at Laing Bridge South indicate a definite linear increase with depth of the \((V_s)_A\) downhole and crosshole velocity ratio. The field data obtained are presented in Fig. 5.39. Since Eq. (5.20) holds, Fig. 5.38 can be used to evaluate the stress dependency of the downhole and crosshole velocity ratios expressed in Eqs. (5.17) and (5.18). The theoretical velocity ratios from Eqs. (5.17) and (5.18) have been plotted as a function of \(K_0\) in Fig. 5.40 for different values of the exponent. Regarding the two plots the following comments can be made:

- the average stress method in Fig. 5.40(a) appears to be a reasonable basis for using the velocity ratio concept to determine \(K_0\) when \(n_t > 0.1\). Data in the literature would suggest an average \(n_t\) value of 0.25.
- the individual stress method in Fig. 5.40(b) is illconditioned for determining \(K_0\) irrespective of the \(n_a\) value. Small changes in the velocity ratio give large changes in predicted \(K_0\), i.e. a \(\pm 10\%\) variation in \((V_s)_A/(V_s)_I = 1.0\) gives a three-fold variation in \(K_0\).

With this in mind and considering the natural scatter in field data, the average stress method would appear best suited to determining \(K_0\) from velocity ratios.

The velocity ratio profiles in Fig. 5.39 suggest that the \(C_A/C_I\) ratio of 1 used for the plots is also incorrect. \(C_A/C_I\) of two appears to be more realistic. This aside there still appears to be some additional effect on the velocity ratio. Although difficult to separate the two effects it may be that the \((V_s)_A/(V_s)_I\) profile more correctly reflects both inherent and
Figure 5.40 Theoretical dependence of $\left( \frac{V_{s}}{V_{s}} \right)_A / \left( \frac{V_{s}}{V_{s}} \right)_I$ on $K_0$. 
induced anisotropy in the profile. Field measurements indicate that the velocity ratio (based on the average stress method) is insensitive to stress but dominated by anisotropic stiffness. Consequently the use of the mean normal stress definition for evaluating the stress dependency of $V_s$ would be more appropriate rather than a velocity ratio approach. In this case estimates of the velocity constants $C_A$ or $C_I$ are required.

5.7 Discussion and Conclusions

5.7.1 Equipment and Testing Methods

The preceding sections have presented and discussed the results of measurements made using various types of in situ testing device capable of measuring the lateral stress acting on an instrumented section. Representative stress and pore pressure profiles have been compared for each of the geotechnical research sites. In relation to the equipment design and performance, the following comments are made:

- The UBC lateral stress cone gave repeatable consistent data in both sand and clay. In order to guarantee saturation of the pore pressure transducers, it was necessary to apply a vacuum once the cone had been assembled. The method of measurement allows quick determination of all standard CPTU parameters with additional information obtained from the lateral stress module. Baseline stability on all channels was good and the laboratory determined temperature corrections were consistent with changes recorded in the field.

A second lateral stress module has been made but using a 0.5 mm wall thickness on the strain-gauged area as opposed to the 1 mm thickness used during this study. This should increase the sensitivity of the present
system and allow transducer amplification to be reduced. Tests performed at Lower Mainland research sites indicate maximum stress measurements of around 800 kPa which may permit the present LS gauge capacity of 1440 kPa to be reduced further so that sensitivity is increased.

For good performance of the LS cone it is necessary to ensure that the LS sleeve is free to move and that it does not become blocked by soil entering behind the quad ring seals. Near surface (0-2 m) data may be unreliable, especially if stones are present. This is, however, a common problem with all in situ probes.

- The total stress cell (TSC) approach to measuring horizontal stress was implemented with varying degrees of success. At Strong Pit where the stiff clayey silt contains stones (up to cobble and even boulder size), three of the nine blades installed were damaged. The disadvantage of this type of thin blade is that they are very fragile as well as expensive. In the soft clay at Lr. 232 St. no problems were encountered during blade installation and all blades operated well.

The measured stresses from the blades were consistent and repeatable. However, due to the location and set-up of the pore pressure measuring system it was difficult to ensure complete saturation. This is reflected in many of the pore pressure measurements which vary considerably throughout the profile. Similarly, the final dissipated pore pressures do not generally agree with expected values based on knowledge of the equilibrium pore pressure at the site.

The spade cells were calibrated in the laboratory prior to installation and again after removal from the site. Small variations in the baseline readings occurred during the period when the blades were installed.
and these shifts were corrected for. Also, when the blades were removed from Strong Pit and recalibrated, the temperature calibration factors had changed. The baseline and calibration changes are summarized in Table 5.2. This is thought to have occurred as a result of wear on the blade during installation in the stiff stony clay silt. At Lr. 232 St. no calibration changes were measured as a result of installation and removal in the soft clay silt.

Table 5.2
Baseline and Calibration Changes for Total Stress Cells After Installation and Removal at Strong Pit

<table>
<thead>
<tr>
<th>Spade Cell No.</th>
<th>Before Installation¹</th>
<th>After Removal¹</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Baseline Pressure (kPa)</td>
<td>Temperature Factor² B_T (kPa/°C)</td>
</tr>
<tr>
<td>1537</td>
<td>156</td>
<td>1.35</td>
</tr>
<tr>
<td>1538</td>
<td>179</td>
<td>0.58</td>
</tr>
<tr>
<td>1541</td>
<td>135</td>
<td>0.48</td>
</tr>
</tbody>
</table>

¹Data for each TSC referenced to same baseline temperature
²B_T for change in baseline pressure on cooling

• The DMT is a simple and effective piece of equipment. Using the new UBC data read-out unit, errors in data capture have been reduced and production increased. However, for the DMT to be used as a research tool it requires the installation of a pore pressure transducer in the vicinity of the expandable membrane. Ideally, so that both the lift-off and limit pressures can be correlated to pore pressure measurements, the pore pressure transducer should be located on the membrane itself. In some soil types, i.e. sensitive clay silts, whether the p₀ value corresponds to a true lift-off or a back-extrapolation to lift-off, may significantly affect results obtained from the blade.
The lateral stress cone developed at Berkeley and used here was found to give data in good agreement with that obtained using the UBC LS cone only in clay. In sand the UCB LS cone significantly underestimated the lateral stress and this is thought to have occurred as a result of dimensional tolerances at the LS sections. Due to the construction of the LS sensor, differences of up to 0.05" (1.27 mm) were measured between the cone diameter at the tip and that of the location of the lateral stress sleeves. The slightly undersized lateral stress sleeves would permit stress relief to occur, resulting in lower than expected stresses to be recorded. The under read would also increase for soils of higher stiffness.

The lateral stress sensors were fairly easily damaged in sand and the low stresses measured may also have resulted from crosstalk effects on the LS sensor resulting from axial friction loading.

The self-boring pressuremeter appears to function reasonably well in the sand. However, the effective stress sensor on the PM membrane was problematical and did not provide reasonable data. With the use of rods with larger wall thicknesses the problem of rod buckling should be avoided. The new six-arm PM developed at UBC could be very useful for determining whether anisotropic lateral stress variations exist or whether they result purely as a consequence of probe characteristics. The installation by jetting appears to be the most favorable method for self-boring in sand and non-plastic silt.

The self-boring load cell tests performed using a rotary cutter were not very successful in sand. Lateral pressures equal to the equilibrium pore pressure (i.e., c' = 0) were recorded and thought to have resulted on account of the disturbance due to the vibrations caused by the rotary head
motor and shaft. The SBLC was modified to enable self-boring by jetting alone which improved the quality of the installation.

- Pore pressure response of in situ probes can be affected by the mechanical design of the system. Careful calibrations are required to evaluate these possible effects.

- Comparison of pore pressure measurements in stiff clays using both ceramic and polypropylene filter materials suggests that filter squeeze effects are not present at the UBC research sites where stiff OC clay silts are present. This is important for interpretation of measured data.

- Both stress and pore pressure measurements are dependent on the location of the sensor with respect to the penetrometer tip. Large gradients exist around the cone tip and may give rise to anomalous data. Locations remote from the cone tip are preferable if stress measurements are to be made.

- The lateral stress oedometer tests appear to provide $K_0$-OCR relationships which are generally consistent with published data. It would appear from the results obtained that the effects of wall friction on the measured horizontal load may be similar to the vertical load reduction that occurs with the net effect that $K_0$ is not affected, at least for the test set-up used during this study. An attempt to minimize ring friction was made by applying a thin film of silicone grease to the inner walls of the ring. In general, the $K_0$-OCR relation was developed for the first unloading although in some cases reload data are available. The change of $K_0$ as a function of OCR obtained for Strong Pit is lower than expected although there seems to be no explanation for the discrepancy.
5.7.2 Soil Response to Probe Installation

The majority of the discussion in this section refers to the installation of full-displacement probes. Where relevant, the measurements related to self-boring probes will also be commented upon.

The following comments can be made based on the data presented in the earlier sections of this chapter:

- during full-displacement penetration in NC fine grained soils, the measured total stresses are dominated by the pore pressure response. Hence the total stress increment is reflected closely by the excess pore pressure. In sensitive soils, the induced excess pore pressure may be larger than the total stress increment. In OC soils the pore pressures are not so dominant as in NC soils and higher levels of effective stress result.
- in free draining coarse grained materials, excess pore pressures are negligible or zero and the total stress change is mirrored by the effective stress change.
- the geometry of the penetrometer is an important factor determining the stress increase induced during probe penetration.
- a gradient of pore pressure exists around a penetrating cone in fine grained material. The gradient is related to both the strength and stiffness of the soil. Large increases in both mean normal stress and shear stress occur beneath the tip of a penetrometer, although the normal stress increment predominates. As the soil passes the cone shoulder the soil is unloaded and the pore pressures are reduced in proportion to the operative stiffness. After the unloading the remaining excess pore pressure is related primarily to the level of shear stress although a component of the mean normal stress is still present.
as the stiffness and strength of the soil being penetrated increases, so does the pore pressure gradient around the tip. In very stiff (high OCR) soils, the unloading may be sufficient to give rise to negative pore pressures at the location.

from the measurements at the three clay sites of varying stress history, it would appear that the induced pore pressures are sensitive to the in situ pre-penetration state of the soil.

for both soft and stiff clay the maximum unloading occurs at a finite distance behind the tip. Moving further away from the tip, the gradient is small and the data presented here show that the measured stress or pore pressure becomes essentially constant for L/D > 4.

a gradient of stress is also evident in fine grained material. While much of the gradient is due to pore pressure effects, it appears that the effective stress is also affected.

similar gradients of stress can be inferred in sand. The result of the above can be useful for designing cones for tests in calibration chambers. For a LS sleeve located at L/D = 4, no effect of the tip unloading will be experienced. For a 10 cm cone, this gives an instrumented sleeve 14.4 cm behind the tip. Previous cones have used distances of 27.0 cm or more and due to limited chamber size, laboratory calibration has not been possible. The use of a L/D ratio of 4 would permit evaluation of LS cone measurements away from the high stress gradients close to the tip.

the data obtained suggest that the type of measurement and measurement technique do not give rise to differences in the measured lateral stress, i.e. LS-CPTU stresses measured during penetration agree with SCPM lift-off stresses measured after halting penetration. This is true in clay provided the lift-off pressure is recorded immediately after halting penetration.
If dissipation of excess pore pressure is permitted, differences in the measured stresses will obviously result.

- whether the stress measurement is obtained by active membrane expansion or passive cell pressure increase does not seem to affect the final measurements in soft materials. In stiff materials, dimensional tolerances are very important and the probe geometry and method of measurement may modify the in situ lateral stress.

- the use of the sleeve friction measured during CPT to index the horizontal stress condition can be misleading due to its location in a zone of rapidly varying stress. The recorded fs value is to some extent a function of the length of the sleeve and not necessarily indicative of variations in lateral stress. The correlations determined between fs and q'_h would be equipment sensitive and not practical for general use.

- the cone resistance, q_T, on the other hand appears to be a good indicator of lateral stress conditions.

- comparison of trends in lateral stress data measured on the shaft of differing penetrometers agrees well with variations in q_T. This emphasizes that even at locations behind the tip of a penetrometer, full displacement penetration testing has not obliterated the stress history imprint of the soil.

- the measured values of pore pressure from the LS-CPTU (u_{LS}) are generally lower than the TSC values (u_{TSC}). This is surprising since the stress measurements would indicate the opposite. This result is considered to result from the poor response of the TSC pore pressure transducer.

- for all total stress measurements, a pore pressure transducer should be located as close as possible to the stress sensitive area. Due to the pore pressure effects, the interpretation of stress variations based solely on
total stress data may be misleading under certain circumstances, i.e. near surface partially saturated crust.

- the disturbance induced by the plate penetrometers appears to be a function of the plate thickness. Stresses measured with the TSC ($t = 6$ mm) are lower than those obtained with the DMT ($t=13.7$ mm) for all soils tested.
- relative to the LS cone, TSC stresses are lower in soft soil, indicating a lesser amount of disturbance. In medium stiff soil, $\sigma_{TSC}$ is equal to $\sigma_{LS}$ indicating an increased level of disturbance brought about by the increase in soil rigidity.
- the increased disturbance by TSC insertion is thought to originate in the 3D nature of the blade. At the edges of the blade, stress concentration zones are set-up. As soil stiffness increases these zones grow such that at some point they begin to overlap with the stress sensitive area giving rise to a higher average measured stress.
- the total stresses measured with the DMT are much higher than those obtained with the LS cone. This is contrary to general belief and is considered to result due to the finite width of the blade. Also because the transition point from the tip to the shaft is smoother for the DMT than for the cone, less stress relief occurs.
- DMT pressures also give a good indication of stress history variations and provide qualitative information regarding horizontal stress anisotropy.
- for the standard DMT, the $p_0$ value is obtained from back-extrapolation using the gradient between 0.05 mm and 1.1 mm membrane movement. A true lift-off pressure is recorded using the UBC research DMT. In sensitive soils, the two types of measurements give rise to very different $p_0$ values.
- as for the cylindrical probes, the response of NC soils to the penetration of flat penetrometers is dominated by pore pressures. Also, pore pressure
effects reduce as the overconsolidation ratio increases.

- following on from the above: \( \Delta u / \sigma' = f \) (soil type, stress history, penetrometer type).

- the DMT lift-off and 1.1 mm expansion pressures are not limit pressures and the difference between \( p_0 \) and \( p_1 \) is less than between \( p_{ho} \) and \( p_L \) from the pressuremeter, i.e., \( (p_1 - p_0)_{DMT} < (p_L - p_{ho})_{PM} \). This difference may be related to the differing stress paths followed during the probe insertion and measurement or be an indication of the non-uniform stresses that arise during the expansion of a PM of finite length.

- comparing the LS-CPTU and DMT in sand, it appears that as the relative density of the sand increase, the difference between the two measured stresses also increases, i.e. at any depth (\( \sigma'_v \) constant) \( (p_0)_{DMT} - (p_0)_{LS} = f(D_r) \). This again implies higher degrees of disturbance with the DMT as the sand stiffness increases. DMT results are known to be influenced by \( D_r \) whereas the LS cone data are not.

- results obtained with the self-boring probes are very variable. The scatter in the data is due to disturbance effects. In some cases, in both sand and clay, lift-off pressures very similar to the DMT 1.1 mm expansion pressures (\( p_1 \)) were obtained. Similar comments also apply to the SBLC data.

- the total stresses measured by the SBPM, SCPM and LS cone are in very good agreement. In sand the pore pressures for all three probes are very similar, thus giving rise to identical effective stresses. This is not to be expected unless either the SBPM insertion is poor, or the stress relief associated with the full-displacement probes fortuitously reduces the lateral stress to its original condition.
by subtracting the measured pore pressure from the lateral stress, initial effective stress profiles (at the time of installation) can be determined. In soft clay, $\sigma_{LS}'$ and $p_0'$ give values very close to the expected $K_0$ effective lateral stress. In sensitive clay, the excess pore pressures are very large and $\sigma'$ may be close to zero, whereas in stiff clay the effective lateral stresses are higher than expected from the $(K_0)_{OC}$ condition.

- the trends in effective stresses at each of the sites are consistent with the evaluated stress history (based on laboratory consolidation tests).
- after halting penetration, the initial excess pressures begin to dissipate. In sands, the change is negligible, but in clay substantial changes due to both pore pressure dissipation and stress redistribution may occur. It may not be feasible to wait the long periods required for complete dissipation to occur in order to interpret the data to provide estimates of the in situ lateral stress condition.
- the accuracy of the available theoretical solutions does not permit extrapolation of short term dissipation data to the long-term condition. The use of extrapolation methods are complicated by many factors, including soil heterogeneity.
- often dissipation analyses are complicated by an initial rise in the measured pore pressure before decay begins. A method of normalization has been suggested to enable these types of dissipation curves to be analyzed.
- at the $u_1$ location, the pore pressure may suffer from large reductions when penetration is halted and the tip is unloaded. The effect becomes more pronounced as the stiffness of the soil increases.
- at the $u_2$ and $u_3$ locations, very long dissipation periods may be required to obtain 50% reduction of the excess pore pressure. Using Bessel function curve fitting techniques it was not possible to reliably model either the
total stress or pore pressure reduction with time.

- it was possible to model the TSC stress dissipation using a power law correlation: $\sigma_{TSC}(t) = \alpha_1 t^{-\beta}$.

- the $\alpha$ values for Strong Pit are larger than for Lr. 232 St. Also the $\beta$ values are more largely negative. This is consistent with the soil types at each of the locations.

- both $\alpha$ and $\beta$ were found to vary linearly with depth at both sites. It may be possible to predict $\sigma_{TSC}^{(\text{final})}$ at any site based on two or three installed blades. To obtain the final effective lateral stress, the equilibrium pore pressure needs to be subtracted from $\sigma_{TSC}^{(\text{final})}$.

- the LS cone data were treated in the same way. In this case the $\beta$ values only corresponded to short term stress relaxation (no pore pressure dissipation). Values of $\beta$ were essentially constant throughout the profile at both sites.

- a downhole and crosshole seismic cone testing set-up was developed with the ability to generate two types of polarized crosshole shear wave.

- individual downhole and crosshole shear wave velocities indicate some degree of stress dependence. However it appears that the velocity ratio may be more indicative of structural anisotropy rather than stress level. This suggests that the mean normal stress may be the appropriate parameter for normalizing $V_s$ data.

- also the theoretical variation of the velocity ratio obtained by the individual stress method suggests that the correlation is illconditioned for predicting $K_0$. 
CHAPTER 6

6. EVALUATION OF FIELD MEASUREMENTS

6.1 Introduction

In Chapter 5 the results of the field measurements performed at several research sites were presented in terms of both initial pore pressures and lateral stresses. The resulting effective stresses during or immediately after halting penetration were considered as well as their subsequent dissipation.

The objective of this thesis is essentially to demonstrate the dependence of full-displacement testing parameters on the pre-penetration in situ stress state. Reference profiles of lateral stress must therefore be determined for comparative purposes in accordance with accepted procedures. This is discussed in the following section. The subsequent sections deal with the interpretation of the field data.

At all the sites, the ground surface was sufficiently level so as to provide essentially 1D conditions. Hence, the measured reference stress profiles are considered to reflect $K_0$ conditions as the horizontal and vertical stresses are assumed to be in principal directions.

To distinguish between $K_0$ profiles obtained from different probes, the equipment subscript is added, i.e. $(K_0)_{SBPM}$ is the $K_0$ value obtained from the results of self-boring pressuremeter tests. Obviously, depending on the quality of the probe installation and test technique, the $(K_0)_{SBPM}$ may or may not reflect the true $K_0$ condition.

In some cases, where the full displacement probe measurements are used, the $K$ so determined is different from $K_0$ so that the subscript zero is left
off and $K_{DMT}$ is the $K$ value determined from evaluation of the basic DMT data.

6.2 Reference Stress Conditions at Research Sites

The reference stress conditions have been determined based on the results of in situ and laboratory test methods usually considered applicable to the soil types under study. For example, the self-boring pressuremeter is widely recognized as being the best available device for obtaining $q_h$ in soft clays and silts and loose to medium dense sand. The performance of the SBPM in stiff clays and dense sand is very variable and the data are often unreliable. The self-boring load cell has also been used in soft to stiff clay with varying degrees of success. In clay, the use of push-in total stress cells has received considerable attention recently and it is now possible to use the results of TSC and SBPM correlations to correct the TSC data for overread if necessary. In addition, $K_0$ values based on laboratory correlations and evaluated from the lateral stress oedometer tests are also presented on the reference profiles. $K_{COR}$ designates $K_0$ from laboratory-based correlations (i.e. PI, LL, etc.) and $K_{LAB}$ refers to the values determined from the $K_0$-OCR oedometer relationships.

6.2.1 McDonald Farm

The reference lateral stress profile has been determined from lift-off pressures measured during SBPM tests. Data in the sand were obtained by the writer whereas the reference data in the underlying silt is taken from Konrad et al. (1985). Using the peak friction angles for the sand reported in Chapter 4, an average value of $K_{COR}$ is obtained using the well-known Jaky approximation. In the clay silt, $K_{COR}$ is derived based on the Brooker and
Ireland (1965) and Mayne and Kulhawy (1982) correlation between PI and $K_0$ for the condition $OCR=1$. The $K_0$ values from the SBPM tests are presented in Fig. 6.1. For the SBPM, the horizontal effective stresses were determined by subtracting the hydrostatic (equilibrium) pore pressure from the measured total lift-off pressures. The effective stress transducer on the membrane malfunctioned during both of the tests so no direct measurements were possible. Based on the CPTU data it appears reasonable to assume (at least during initial membrane lift-off) that no excess pore pressures were present due to the self-boring process.

In the sand and clay silt, the laboratory-based correlations give values generally lower than the field tests (Fig. 6.1). However, due to the disturbance caused during installation - even if only as a result of generated skin friction between the soil and probe - the SBPM lift-off stresses could be expected to be slightly higher than the pre-existing in situ condition provided the shearing did not produce any structural collapse in the soil. In the profile, the average values of $K$ are:

$$\begin{align*}
\text{Sand: } (K_0)_{\text{SBPM}} &= 0.52 \\
K_{\text{COR}} &= 0.35 \\
\text{Clay Silt: } (K_0)_{\text{SBPM}} &= 0.56 \\
K_{\text{COR}} &= 0.48
\end{align*}$$

The SBPM data presented in Fig. 6.1 for the sand are considered to be of good quality and were obtained after considerable time was spent trying to improve the techniques for installation of probe by self-boring. The writer is grateful to Dr. J.M.O. Hughes for his assistance during this period.

$(K_0)_{\text{SBPM}}$ is given by:
Figure 6.1 $K_0$ variation at McDonald Farm from SBPM and laboratory-based correlations.
\[(K_0)_{SBPM} = \frac{\sigma_{SBPM} - u_o}{\sigma'_{v}} \hspace{1cm} (6.1)\]

where \(\sigma_{SBPM}\) is the total lift-off pressure. For the sand:

\[K_{COR} = 1 - \sin(\phi'_{CPT}) \hspace{1cm} (6.2)\]

and for the clay silt:

\[K_{COR} = 0.44 + 0.42 \left( \frac{P1}{100} \right) \hspace{1cm} (6.3)\]

### 6.2.2 Laing Bridge South

Due to the fact that the self-boring pressuremeter was lost during the last test performed at McDonald Farm, it was not possible to obtain this as reference data for the site. However, self-boring load cell tests were performed to a depth of 8 m. No penetration below 8 m was possible due to the high friction loads generated when pushing through dense sand. Furthermore, occasional gravel within the sand made self-boring difficult and resulted in damage to the SBLC cutting shoe. The \((K_0)_{SBLC}\) values are presented in Fig. 6.2 along with correlation results obtained using the peak friction angle (from CPT) and the Jaky expression. Although the data are limited the results are considered to be reasonable (±50% variation) as confirmed by the available DMT results (discussed later in Section 6.5).

As for the McDonald Farm data, the laboratory correlation provides a lower bound estimate of \((K_0)_{SBLC}\). The SBLC values plotted are average values obtained from both the C and D lateral stress sensors. The effective lateral stress at each sensor is obtained by subtracting the two measured pore pressures (A and B sensors) as discussed in Chapter 5, and
$K_0 = 1 - \sin(\varphi_{CPT})$

Figure 6.2 $K_0$ profiles for Laing Bridge South based on SBLC results and laboratory-based correlations.

$$\sigma_{SBLC}' = \frac{\sigma_{SBLC} - u_{SBLC}}{\sigma_V}$$  \hspace{1cm} (6.4)

where $\sigma_{SBLC}$ is the zero reading on the C or D stress sensor and $u_{SBLC}$ is the pore pressure measured by the A or B sensor.
6.2.3 Strong Pit

The reference lateral stress profile at Strong Pit was determined using the push-in spade cells (TSC) described in Chapter 3. The spade cells were installed in the ground and left for a period of about 3 months during which time readings of both the cell and pore pressure were taken. A typical curve showing the basic raw data and effects of the corrections applied for baseline reading and temperature adjustments is presented in Fig. 6.3. The effect of the equilibrium pore pressure approximating to zero is indicated by

![Figure 6.3 Measured and corrected pressures for TSC 1538 at Strong Pit.](image-url)
the calculated total and effective lateral stresses becoming equal as the excess pore pressure generated during installation dissipates.

To obtain an estimate of the true in situ horizontal stress, the measured blade pressures have to be corrected to account for over-read. The over-read results from the full displacement method of installation and has been shown to be a function of the undrained shear strength of the soil, $S_u$ (Tedd and Charles, 1983). The empirical correction method suggested is:

$$ (\sigma_{TSC})^* = \sigma_{TSC} - 0.5 S_u $$ \hspace{1cm} (6.5)

where $(\sigma_{TSC})^*$ is the corrected TSC lateral stress, assumed to represent the pre-penetration horizontal stress and $\sigma_{TSC}$ is the net total lateral stress measured by the blade and adjusted for temperature effects. Then,

$$ K_{TSC} = \frac{(\sigma_{TSC})^* - u_o}{\sigma_v^*} $$ \hspace{1cm} (6.6)

and it is assumed that $K_{TSC} = K_0$.

Tedd and Charles (1983) use a correction technique based on $S_u$ since it is very difficult to define an appropriate modulus for the problem. Using the approach of Finn (1963), they suggest that the elastic stress increment caused by blade insertion can be evaluated as:

$$ \Delta \sigma = 0.41 \frac{d}{b} \cdot \frac{E}{(1-v^2)} $$ \hspace{1cm} (6.7)

For the Solinst cells, $d/b = 0.06$ and so,

$$ \Delta \sigma = 0.025 \frac{E}{1-v^2} $$ \hspace{1cm} (6.8)
To analyze the residual stress increase at the end of complete dissipation is difficult since the soil close to the blade undergoes a complex series of stress paths caused by:

- initial undrained loading and unloading as blade is installed,
- subsequent dissipation of pore pressures as the soil reconsolidates,
- finally a period of relaxation during which some degree of stress redistribution may occur.

For the initial undrained phase, during installation, Eq. (6.8) can be rewritten as:

\[ \Delta \sigma = 0.1G \]  

since \( v = 0.5 \) and \( E_u = 3G \). The problem of which value of \( G \) to use remains. \( G_{\text{max}} \) determined from the seismic cone penetration test would be a convenient choice so this was investigated using the data base originally employed to evaluate Eq. (6.5). From this it appeared that the overread was equivalent to 0.1% of \( G_{\text{max}} \) (seismic downhole test). For this reduction to have occurred, the soil does not necessarily behave linear elastically. However, Eq. (6.9) may provide reasonable estimates of the initial stress increase during installation of the cells.

The profiles of horizontal stress for the uncorrected and corrected TSC data are presented in Fig. 6.4. The variations in equilibrium pore pressure and vertical stress are also shown. The measured values of \( S_u \) and \( c'_{\text{vm}} \) (from oedometer tests) are also indicated as these parameters serve to illustrate the variation in stress history at the site. Measured equilibrium pore pressures vary between 0 and 10 kPa.
Figure 6.4 Evaluated stress profiles for Strong Pit based on field and laboratory measurements (Sully and Campanella, 1989).

The resulting $K_0$ profiles from corrected and uncorrected data are shown in Fig. 6.5. The variations in OCR and $S_u/\sigma_v'$ are superimposed on the figure. Using the OCR values, $K_0$ values have also been predicted using the results of the lateral stress oedometer tests. The primary dependence of both $K_0$ and $S_u/\sigma_v'$ from field measurements on OCR is evident from the data.

The $K_0$ values from corrected TSC data and LS oedometer correlations are shown in Fig. 6.6. The results show that the corrected value from TSC gives
Corrected TSC pressures obtained from 
Tedd & Charles (1983) empirical method

Figure 6.5 Profile of stress history related parameters for the clayey silt 
at Strong Pit (Sully and Campanella, 1989).

a maximum $K_0$ of nearly two at 2 m depth where OCR is about 10, reducing to a 
$K_0$ value of about 1.1 at a depth of 9 m where OCR is about 2.7. The varia-
tion in $K_0$ at each depth results from the range in pore pressure Eq. (6.6). 
The effective horizontal stress is calculated by assuming two conditions for
Corrected TSC pressures obtained from 
Tedd & Charles (1983) empirical method

Figure 6.6 $K_0$ variation at Strong Pit from TSC and LS oedometer results.

$\kappa_0$, i.e., 0 or 10 kPa as suggested by the field measurements in Fig. 6.4.

The $K_0$ values estimated from the LS oedometer relationship:

$$K_{LAB} = (K_0)_{OC} = 0.530 (OCR)^{0.360}$$

(6.10)

which was derived for the first cycle of unloading are consistently lower than the $K_0$ from TSC. The TSC data can be fitted according to:
\[(K_0)_{TSC} = 0.55 \text{ (OCR)}^{0.610}\] (6.11)

It is interesting to note that the constants in Eq. (6.11) agree well with the relationship derived from LS oedometer tests during initial reloading of the sample. This may indicate that the reduction applied to the TSC data is too small with the result that the in situ horizontal stresses are overpredicted.

The stress history at the site has been evaluated from the results of both laboratory (LS oedometer, oedometer) and field (vane) tests and is considered to be reliable. All the stress history data are verified by the calculated height of overburden removed during gravel extraction at the site. Hence it appears that the overconsolidated state of the stony silty clay can be explained solely on the basis of unloading due to overburden removal. This implies that the form of Eq. (6.11) is primarily dependent on errors in the evaluation of \((K_0)_{TSC}\). Alternatively, the LS oedometer correlation may simply underpredict \(K_0\) in the field, a fact reported for many studies of this kind. As is the case for many studies into the measurement of lateral stress some ambiguity arises as to the correct values applicable to the site. Experience in the U.K. with stress cells in stiff clays would, however, suggest that the final pressures, corrected according to the Tedd and Charles (1983) procedure, provide reasonable estimates of in situ \(\sigma_h'\). It should be noted that the adjustment to account for over-read is large in relation to the measured total pressure and may induce unacceptably large errors due to the nature of the correction applied. Furthermore, in stiff clay the accuracy of measured undrained shear strengths may be questionable. At this site, the ratio of measured total horizontal pressure to corrected pressure varies from 1.5 to 2.0.
6.2.4 Lr. 232 St.

Horizontal stresses at this site have been determined using the SBLC and TSC equipment. Profiles of the measured total and effective lateral stresses have been presented in Chapter 5. Both incremental and lateral stress oedometer tests have been performed on recovered block and tube samples to evaluate OCR and the $K_o$-OCR relationship.

The results of the laboratory tests are presented in Fig. 6.7 and compared with OCR values calculated from the undrained strength ratio, $S_u / \sigma_v'$.

![Diagram](image_url)

Figure 6.7 Variation of OCR with depth at Lr. 232 St. (Campanella, Sully and Robertson, 1989).
The scatter in OCR above 4 m is considered to result from the presence of open root holes in the soil which caused problems of sample preparation.

After installation, the spade cell measurements showed identical trends to those observed at Strong Pit. An initial period of rapid decrease of total horizontal stress is noted during the first few days after installation. Thereafter, the decrease becomes very small until an essentially constant value is obtained. Pore pressure changes are occurring at similar rates. It is interesting to note that after the initial rapid decrease, the effective horizontal stress changes very little during the subsequent monitoring period. Typical data for three of the blades installed are shown in Fig. 6.8.

From the TSC measurements, the resulting total lateral stress and pore pressure profiles are indicated in Fig. 6.9. The variation of \( \sigma_v, u_o \) and \( \sigma_{vm} \) are also shown. The final pore pressures obtained from the pressure cells agree well with the equilibrium pore pressure calculated with the water table at 1.0 m below ground level although some scatter is present. The lateral pressures above 5 m are close to \( \sigma_v \) and become progressively lower with depth.

The calculated \((K_o)^\text{TSC}\) values are show in Fig. 6.10(a) and compared with the results of the LS oedometer tests which yielded the following \( K_o \)-OCR relationship:

\[
K_{\text{LAB}} = (K_o)_{OC} = 0.56 \,(\text{OCR})^{0.45}
\]  

(6.12)

This determined relationship agrees well with the empirical correlation presented by Brooker and Ireland (1965) based on soil plasticity. The good agreement between the laboratory and field data is encouraging.
Figure 6.8 Time dependence of $\sigma_{TSC}$ at Lr. 232 St.

$K_{LAB}$ has also been determined from the results of oedometer tests on both horizontally and vertically-cut samples. The horizontally-cut sample provide estimates of $\sigma'_{vm}$ and hence OCR whereas the vertically-cut samples provide $\sigma'_{hm}$ and $K_{LAB}$. The data in Fig. 6.10(b) have been obtained using samples cut from undisturbed block and tube samples. The definition of $K_{LAB}$ can be expressed in two possible forms based on the laboratory data, i.e.
Figure 6.9 Measured stress profiles at Lr. 232 St.

\[ K_1 = \frac{\sigma'_{hm}}{\sigma'_v} \]  \hspace{1cm} (6.13)

or

\[ K_2 = \frac{\sigma'_{hm}}{\sigma'_{vm}} \]  \hspace{1cm} (6.14)

\( K_2 \) is the smaller of the two and for conditions of ideal sampling should be conceptually equal to \((K_0)_{NC}\), since both \(\sigma'_{hm}\) and \(\sigma'_{vm}\) occurred at some normally consolidated prior state. Hence, \( K_2 \) cannot be considered as the \( K \) represent-
Figure 6.10 $K_0$ profiles from laboratory and field measurements - Lr. 232 St.
ative of present conditions. It may also provide a check on the quality of the oedometer data. $K_1$ on the other hand corresponds to the maximum possible value of $K$. Since during the unloading, $\sigma'_h$ has reduced from $\sigma'_{hm}$ to some other arbitrary value, then $K_1$ will theoretically overpredict the true present value, $K_0$. The values on Fig. 6.10(b) correspond to $K_2$ and can be seen to be scattered around the NC $K_0$ value of 0.56. $K_1$ values, all of which are larger than 2 in the surficial crust, are not shown on the figure. They obviously represent a maximum fictitious $K$ value which cannot exist under the present stress conditions. It would appear from this, that apart from conceptual problems in the use of oriented samples in oedometers, i.e. reorientation of loading direction, the theoretical basis for performing this type of test is incorrect. The good agreement of $K_1$ with measured $K_0$ conditions obtained from the one result at Strong Pit may be fortuitous. At Lr. 232 St. the $K_1$ data are anomalous and meaningless. $K_2$ should be equal to $(K_0)_{NC}$ as is the case at this site.

The SBLC data on Fig. 6.10(b) suggest $K_{SBLC}$ values lower than $K_{LAB}$. The SBLC data have been reduced using the initial stresses measured in the field, i.e. Eq. (6.4). While the trend of the data appears to be compatible with the results in Fig. 6.10(a), the $K_{SBLC}$ are on the low side. This is thought to arise due to the high sensitivity of the clay silt. As was demonstrated in Chapter 5, after initial penetration the effective stresses are very low due to induced pore pressures larger than the total stress increase. Only by allowing dissipation of the excess pore pressures could higher $K_0$ values be obtained. For the full-displacement probes, it was concluded that the initial effective horizontal stresses could only be reasonably estimated by means of a total stress analysis, i.e., evaluating the measured total lateral stress or induced excess pore pressures. This may also be true for the SBLC but the problem is complicated in that the SBLC installation cannot be
modelled as a cavity expansion process since it is assumed to enter without causing disturbance. What disturbance that is caused, is probably less than that associated with full-displacement probes. In non-sensitive soils the SBLC may perform well; in sensitive soils it appears that long relaxation periods are necessary for the effects of disturbance to be diminished.

6.2.5 200th St.

The variation in OCR at this site has been determined from 1D consolidation tests, and is shown in Fig. 6.11. The relationship obtained from LS oedometer tests is also indicated \( (K_{LAB}) \) as are the results from the TSC measurements. Initially, four spade cells were installed at the site but two of them were damaged during installation. The \( K_{TSC} \) values in Fig. 6.11 have been corrected to account for overread according to Eq. (6.5). Good agreement between measured and interpreted stress history parameters is evident from Fig. 6.11.

6.2.6 Conclusions

The reference stress profiles presented above have been obtained from two types of tests, namely:

- self-boring probe data in sand at McDonald Farm and Laing Bridge South. Results of SBPM tests are also available for the clay silt at McDonald Farm.
- TSC data in the fine grained soils at Lr. 232 St., Strong Pit and 200th St. SBLC data are also available at Lr. 232 St.

In the case of the self-boring pressuremeter data, the \( K_{SBPM} \) determined from lift-off pressures is assumed to be equivalent to \( K_{s} \) which implies no
disturbance during probe installation. At McDonald Farm the $K_o$ values are consistent with the expected stress history. This is also true for Laing Bridge South but much more scatter is present in the data but this appears to be equipment related rather than representative of in situ variations. For the case of the TSC data, it is assumed that the adjusted lateral pressures
correctly reflect in situ $q_h$ variations, i.e. the disturbance induced during installation is accounted for by reducing the measured stress in proportion to the undrained shear strength. Hence $K_{TSC}$ is assumed to equal to $K_0$.

The so-defined reference stress profiles will be used in subsequent sections for determining parameters indicative of stress history variations and for interpreting the stresses and pore pressures recorded during the installation of full displacement probes.

6.3 Stress History Parameters from CPTU

6.3.1 Introduction

In the ideas presented in Chapters 1 and 2, and also in the discussion of the field data presented in Chapter 5, it has been argued and subsequently demonstrated that certain large strain parameters are indicative of the variation in stress history within a particular soil deposit. A commonly used example of this is the correlation between the undrained strength ratio in clay and OCR. Data presented in Section 6.2 illustrate how reliable OCR estimates based on $S_u/\sigma'_v$ can be when compared to the results of laboratory consolidation tests. Similar stress history parameters can be determined from other in situ test data.

The results of piezocone (CPTU) soundings have been chosen as the basis for determining index parameters related to stress history since:

- data from CPTU soundings are very repeatable and not subject to operator bias, provided the equipment calibration and test set-up are performed in accordance with standard procedures.
- the equipment provides a number of soil parameters which can be combined in order to optimize the sensitivity to variations in soil characteristics.
• the electronic data capture permits definition of small scale variations within a soil profile which may affect the parameter(s) being evaluated.
• piezocone testing is commonly used worldwide and published data are available for many well-documented research sites. This allows for comparison of data trends obtained at local research sites with those obtained in diverse soil types at other locations.

An attempt was made to define stress history indicators in both sand and clay. More specifically, in the proceeding sections, the in situ stress state is considered in terms of OCR and $K_0$ only. While it is recognized that other parameters may be indicators of the soil state, this section is limited to the evaluation of stress state based on CPTU data. In sand the cone resistance has been correlated to $K_0$, while in clay the correlations make use of the penetration pore pressures. These parameters are considered to be the optimum ones in each particular soil type in terms of sensitivity to changes in stress history. The advantage of using these types of index parameters is that they permit rapid estimates of variations in stress state to be obtained from relatively uncomplicated test procedures.

6.3.2 $K_0$ from CPT in Sand

Calibration chamber testing in sand has demonstrated the effect of certain variables on the measured CPT parameters, i.e. $q_c$ and $f_s$. Results presented by Baldi et al. (1982, 1986a, 1986b) concentrate primarily on the effects of relative density ($D_r$) and OCR and do not distinguish between the effects of changes of stress state, i.e. $\sigma_v'\text{ and } \sigma_h'$. Houlsby and Hitchman (1988) performed 19 CPT chamber tests with up to 7 different stress conditions. $K_0$ values ranged from 0.5 to 2.0. The results indicate very
Figure 6.12 Effect of stress state on $q_c$ measured in calibration chamber tests (after Houlsby and Hitchman, 1988).

strongly the unique dependence of $q_c$ on $D_r$ and $\sigma'_h$ (Fig. 6.12a) whereas no such correlation is evident with $\sigma'_v$ (Fig. 6.12b).

Houlsby and Hitchman suggest a relationship of the form:

$$\frac{q_c}{P_a} = A \left(\frac{\sigma'_h}{P_a}\right)^{0.6} \quad (6.15)$$

where $P_a$ is the atmospheric pressure and $A$ is a factor which depends on $D_r$ (or $\phi$). The relationship can also be written as:

$$q_c = N_h \sigma'_h \quad (6.16)$$

where $N_h$ represents changes in density (or friction angle) and soil stiffness. $N_h$ was shown to be a function of $\phi'_t$ (from triaxial compression tests):
Sully and Campanella (1989) re-evaluated the published data of Houlsby and Hitchman (1988) to determine a relationship for $A$ with $D_r$. The resulting linear equation is of the form (Fig. 6.13):

$$A = 2.99 \left( D_r \% \right) - 23 \quad (6.18)$$

In the field, $D_r$ can be determined at any depth using the relationship between

by Baldi et al. (1986):

$$D_r = \frac{1}{C_2} \ln \left[ \frac{q_c}{C_0 \left( \sigma'_v \right) C_1} \right] \quad (6.19)$$

where:

- $C_0 = 172$
- $C_1 = 0.51$
- $C_2 = 2.73$

and $q_c$ and $\sigma'_v$ are in kPa.

In Fig. 6.13, some of the data points for dense sand are not in good agreement with the linear extrapolation. This is due to the effects of the chamber boundaries on the measured $q_c$ since the data have not been corrected for chamber size effects. The linear correlation has been weighted to consider the data at lower densities since at these values chamber size effects do not greatly influence test results.
Figure 6.13 Correlation between $A$ and $D_r$ based on calibration chamber test data.

Equation (6.15) can be re-written in units of bar (1 bar = 100 kPa) to eliminate $P_a$. $K_0$ can be evaluated from field data by $(q'_h = K_0 q'_v)$:

$$K_0 = \frac{1}{\sigma'_v} \cdot \left(\frac{q'_c}{A}\right)^{1.67}$$

(6.19)

The method has been applied at the two sand sites used during this research and the predicted $K_{CPT}$ values are compared to SBPM and laboratory values in Fig. 6.14. The agreement between $(K_0)_{SBPM}$ and $K_{CPT}$ at McDonald Farm is excellent. At Laing Bridge South, $K_{CPT}$ falls between the range of $K_{SBLC}$
Figure 6.14 Predicted $K_{CPT}$ values for McDonald Farm and Laing Bridge South.
values. The suggested method would appear to provide lateral stress coefficients consistent with those of the reference profile.

6.3.3 OCR from CPTU in Clay

From results of penetration tests in Boston Blue Clay, Baligh et al. (1980) suggested that the pore pressure measured during undrained penetration in clays may be a function of its stress history. Subsequently, various pore pressure parameters have been proposed as indicators of stress history. The various correlations equate OCR with one or more of the parameters measured during a piezocone sounding, namely the total pore pressure, u, the excess pore pressure, Δu, and the corrected tip resistance, qt. In some cases, σv or σ'v are incorporated. Campanella and Robertson (1988) review the different parameters that have been suggested for delineating OCR variations in fine grained soils.

Wroth (1984) pointed out that only the shear induced pore pressure resulting from cone penetration correctly reflects the soil stress history. Based on Critical State Soil Mechanics, Wroth (1984) stated that any pore pressure parameter to be used for OCR correlation should relate a change in pore pressure (Δu) to changes in octahedral and shear stresses around a penetrating cone. Due to problems in evaluating stress changes close to a cone, the most promising pore pressure parameter available was considered to be Bq, as defined by:

\[ B_q = \frac{\Delta u}{q_t - \sigma_v} \]  

(6.20)

However, this does not allow the octahedral and shear related pore pressures to be separated, since the measuring locations are generally in areas where
the soil undergoes complex stress changes. Studies performed by various researchers have shown that no universal correlation exists between $B_q$ and OCR (Battaglio et al., 1986; Campanella, et al., 1986; Jamiolkowski et al., 1985; Lunne et al., 1985). Jamiolkowski et al. (1985) suggest that while $B_q$ should reflect changes in OCR, further validation of the parameter is necessary in relation to the stress field around a cone.

The foregoing comments are confirmed by the results of the data presented in Chapter 5. The data have also shown that the measured pore pressures are dependent on both soil characteristics and probe geometry. It is worthwhile noting that all of the proposed pore pressure parameters use only one value of measured pore pressure, usually that corresponding to a filter located behind the tip. For that reason it is difficult to evaluate the shear induced pore pressures which may provide the basis for an OCR correlation. Robertson et al. (1986) recently presented field data on the distribution of normalized pore pressures around a penetrating cone for various cohesive deposits of varying OCR (Fig. 2.9). The total dynamic pore pressure, $u$, is normalized with respect to the hydrostatic pore pressure, $u_0$, at the depth of interest. The following comments regarding the figure can be made:

- the pore pressure measured on the tip or face of the cone is always higher than that measured behind the cone.
- as the overconsolidation ratio increases, the pore pressure measured at the tip or along the face of the cone is positive and increases. However, behind the tip the pore pressure may become negative depending on the level of overconsolidation.

Based on the above observations, it would appear that the ratio of, or the difference between, the normalized pore pressures measured on the face,
(u/u_o)_1, and at the base of the cone, (u/u_o)_2, may provide a basis for evaluating the stress history of a clay. This relationship can be explained by considering the soil behaviour at each of the measuring locations. On the tip, the high normal stresses appear to dominate the pore pressure response, whereas behind the tip the normal stresses are released and the shear induced pore pressures appear to be dominant. As the overconsolidation ratio of the soil increases the difference between the pore pressures increases.

Sully, Campanella and Robertson (1988b) evaluated various ratios which incorporated the u_1 and u_2 pore pressures. Considerable scatter existed when the ratios were correlated with OCR. Use of a pore pressure ratio relating two values, one of which is usually considerably larger than the other, gives rise to a large degree of uncertainty as errors in measurement become very important. In addition, the large variation in pore pressure ratio, especially at high OCR, may result from not only from soil characteristics but also from cone geometry and tolerances.

A pore pressure difference parameter was found to be more appropriate as a stress history indicator. The pore pressure difference parameter, PPD, was defined as:

\[ \text{PPD} = \frac{u_1 - u_2}{u_o} = \frac{\Delta u_1 - \Delta u_2}{u_o} \]  

Published data from North and South American clays were reviewed and the following relationship proposed (Sully, Campanella and Robertson, 1988a):

\[ \text{OCR} = 0.66 + 1.43 \text{(PPD)} \quad (r = 0.98) \]  

where r is the coefficient of correlation.
At that time further validation of the idea was not possible due to a lack of data. Subsequently, data from several European clays were included, and the data base more than doubled. The resulting correlation is shown in Fig. 6.15 and given by Sully, Campanella and Robertson (1988b):

\[
OCR = 0.49 + 1.50 \times (PPD) \quad (r = 0.96) \quad (6.23)
\]

The correlation in Fig. 6.15 is valid up to an OCR of about 10 to 15. The method was applied to CPTU obtained at Lr. 232 St. (not used for initial correlation) and the results are shown in Fig. 6.16. The PPD prediction
appears to slightly overestimate OCR when compared to the laboratory data. This may result from the high excess pore pressures generated during penetration in this sensitive clay.

The correlation given by Eq. (6.23) was also evaluated for clays with much higher overconsolidation ratios. The agreement with Eq. (6.23) was not good (Fig. 6.17) and a general relationship is not possible. However, individual site specific linear correlations are evident. The results at high OCR may be complicated by the fact that cone geometry may contribute appreciably to the measured "soil response", especially at the location

Figure 6.16 Comparison of OCR predicted from PPD with the reference profile at Lr. 232 St.
behind the tip. This is evident from the results presented in Chapter 5. The data in Fig. 6.17 have been obtained from published studies where various types of cone penetrometer have been used. As the soil stiffness increases, then the measured response will be progressively more influenced by variations in dimensional tolerances between the tip, pore pressure filter and friction sleeve. The soil at the base of the tip is undergoing unloading and dramatic changes of strain rate and direction, hence the gradient of stress and pore pressure relative to the measuring location will affect the measured data.

The PPD definition uses the $u_1$ and $u_2$ pore pressures since these are more readily available in the literature. From the conclusions in Chapter 5,
it would appear more reasonable to incorporate a pore pressure value measured further away from the tip singularity since the gradients there are much less and consequently pore pressure measurements may been more reliable. Figure 6.18 presents CPTU from heavily overconsolidated Taranto clay where three pore pressure measurements were used (Battaglio et al., 1986). The more consistent measurements of $u_3$ provide a better relationship for PPD and OCR resulting in much less scatter. However the use of the $u_3$ pore pressure is restricted in practice since few piezocones are available that are able to perform this measurement.

Figure 6.18 Comparison of PPD parameters using $u_1$ and $u_2$ or $u_3$ for heavily overconsolidated Taranto clay (Sully, Campanella and Robertson, 1988a).
6.3.4 **Effect of Lateral Stress on Penetration Pore Pressures**

Based on the preceding section, it is possible to extend the PPD principle to evaluating the effects of lateral stress on penetration pore pressures. The excess pore pressure measured during CPTU at any location on or behind the cone tip can be expressed as a proportion of the total cone resistance, \( q_t \):

\[
\Delta u_1 = f_1(q_t) \quad (6.24)
\]

\[
\Delta u_2 = f_2(q_t) \quad (6.25)
\]

where the constants \( f_1 \) and \( f_2 \) vary according to the characteristics and stress history of the soil. Data to confirm this locally has been presented by Campanella, Sully and Robertson (1988) and in general, for many different soil types by Mayne and Holtz (1988).

It follows then, that the pore pressure gradient around the tip (\( \Delta u_1 - \Delta u_2 = u_2 - u_1 \)) is also some function of \( q_t \). Furthermore, the acceptance that the cone resistance in clay is principally related to the horizontal effective stress, as has been shown to be the case for sands tested in calibration chambers (Baldi et al., 1982; Houlsby and Hitchman, 1988), leads to the conclusion that:

\[
u_1 - u_2 = f_3(q_t) = f_4(\sigma'_h) \quad (6.26)\]

The dependence in Eq. (6.26) between the pore pressure gradient around the tip and the in situ horizontal stress is also demonstrated by the data presented in Chapter 5 for the clay sites.
Dividing the pore pressure gradient defined in Eq. (6.26) by the vertical effective stress gives a hypothetical correlation to the commonly normalized form of the lateral stress condition:

\[
\frac{u_1-u_2}{\sigma'_v} = f_4 \left( \frac{\sigma'_h}{\sigma'_v} \right) = f_4 (K_0) \tag{6.27}
\]

The pore pressure difference normalized in this way is termed PPSV. Very little chamber test data exist for undrained penetration in clays; that which does exist is inadequate to provide sufficient data to evaluate the suggested dependence of PPSV on \( K_0 \). Consequently, it has been necessary to use field data to evaluate the basis of Eq. (6.27).

Initially, lateral stress measurements at two of the research sites in the Lower mainland were correlated with the results of piezocone soundings (Fig. 6.19). However, the results were inconclusive and limited. To amplify the data base, published data from recognized research centres around the world were also incorporated. Table 6.1 lists the soil deposits that have been used and provides information concerning the types of measurements performed at each. The results of the study are shown on Fig. 6.20, which indicate that for a particular site there exists a definite trend between the normalized pore pressure parameter, PPSV, and the lateral stress coefficient, \( K_0 \). Even for the anomalous Brent Cross data a definite trend exists.

Assuming a linear variation between PPSV and \( K_0 \) (as suggested by Fig. 6.20) then:

\[
K_0 = a + b(PPSV) \tag{6.28}
\]
where $a$ and $b$ are constants. The coefficient, $a$, will always be less than the normally consolidated value of $K_0$ since the minimum value of PPSV is between 0.25 and 0.75 depending on the clay characteristics. The second coefficient, $b$, averages 0.11. For many of the sites used to produce Fig. 6.20 this approximation is fairly realistic (for comparison with the data trends of individual sites, the dotted line on Fig. 6.20 has a gradient of 0.11). At other sites, notably St. Alban and Lr. 232 St., both comprising sensitive clay silts, this simplification does not hold; however, the variation between $K_0$ and PPSV for a particular site essentially remains linear. The method employed for the measurement of lateral stress at Brent
### Table 6.1
Details of In Situ Measurements for Evaluation of $K_0$ and PPSV Parameters (Sully and Campanella, 1991)

<table>
<thead>
<tr>
<th>Site Location and Soil Type</th>
<th>Methods Used For Lateral Stress Determination&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Brief Details of Piezocone Measurements&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cowden, U.K. (Glacial clay till)</td>
<td>PBPM, SBPM, TSC</td>
<td>D</td>
<td>Lunne et al. (1986b) Powell et al. (1983)</td>
</tr>
<tr>
<td>Emmerstad, Norway (Quick clay)</td>
<td>FV</td>
<td>D</td>
<td>Aas (1985) Aas et al. (1986)</td>
</tr>
<tr>
<td>Haga, Norway (Lean marine clay)</td>
<td>FV, HF, SBPM</td>
<td>D</td>
<td>Aas et al. (1986) Lunne et al. (1986a)</td>
</tr>
<tr>
<td>Madingley, U.K. (Gault clay)</td>
<td>SBPM, TSC</td>
<td>D</td>
<td>Lunne et al. (1986b) Powell &amp; Uglow (1986)</td>
</tr>
<tr>
<td>Rio de Janeiro, Brazil (Guanabara Bay clay)</td>
<td>No information given</td>
<td>S</td>
<td>Sills et al. (1988) Soares and Dias (1989)</td>
</tr>
<tr>
<td>Saugus, MIT (Boston Blue clay)</td>
<td>HF, SBPM</td>
<td>D</td>
<td>Ladd et al. (1979) Levadoux &amp; Baligh (1980)</td>
</tr>
<tr>
<td>St. Alban, Quebec (Champlain Sea clay)</td>
<td>SBPM, TSC</td>
<td>D</td>
<td>Roy &amp; Chi Thien (1987) Roy et al. (1982a,b) Tavenas et al. (1975)</td>
</tr>
<tr>
<td>Strong Pit, B.C. (Stiff clay silt)</td>
<td>TSC</td>
<td>D</td>
<td>Sully &amp; Campanella (1989)</td>
</tr>
<tr>
<td>McDonald Farm, Vancouver</td>
<td>SBPM</td>
<td>D</td>
<td>Gillespie (1990) Konrad et al. (1985)</td>
</tr>
<tr>
<td>Calib. Chamber, Oxford (Speswhite kaolin)</td>
<td>Transducers on CC boundaries</td>
<td>S</td>
<td>May (1987)</td>
</tr>
</tbody>
</table>

---

<sup>a</sup> FV = Field vane  
HF = Hydraulic fracturing  
LAB = Laboratory data  
PBPM = Prebored pressuremeter  
SBPM = Self-bored pressuremeter  
TSC = Total stress cells (push-in type)

<sup>b</sup> S = Simultaneous measurement of $u_1$ and $u_2$ pore pressures in one sounding  
D = Dual (adjacent) soundings to independently measure $u_1$ and $u_2$
Cross (Lab. and prebored PM) may suggest an explanation for the discrepancy (Table 6.1) since at most of the other sites either self-boring pressuremeter (SBPM) or corrected total stress cell (TSC) results form the reference for $K_0$.

This aside, it is unlikely that a global correlation exists between $K_0$ and PPSV since stress history may result from various mechanisms. The sensitivity of penetration pore pressures to these individual mechanisms has not been extensively studied although the topic has been considered earlier in this thesis.

Evaluation of field data from fourteen sites and one calibration chamber study suggests that an approximately linear relationship exists between PPSV and $K_0$ for any particular site. The scatter in the data precludes the use of a generalized relationship. However, the advantage of this type of approach may be in the evaluation of $K_0$ for moderately to heavily overconsolidated clays; these materials present problems for lateral stress measurement using any of
the methods listed in Table 6.1 (Jamiolkowski et al., 1985). The normally consolidated value of $K_0$ may be estimated from an empirical relationship with PI or $\phi'$ (Brooker and Ireland, 1960; Mayne and Kulhawy, 1982) and the over-consolidated value obtained based on pore pressure measurements during CPTU and the correlations shown in Fig. 6.20.

6.3.5 Conclusions

This section has evaluated several parameters which have been shown to be related to the stress history in both sand and clay deposits. The application of these parameters to the research sites included in this research has shown them to be a useful form of evaluating changes in both $K_0$ and OCR within a soil profile. The parameters defined are all based on large strain in situ measurements and further confirm the results of Chapter 5, where the connection between small strain and large strain had been demonstrated based on field measurements. As discussed in Chapter 1, the horizontal stress is measured under conditions of low or small strain.

6.4 Interpretation of Cone Data by Cavity Expansion Theory

6.4.1 Introduction

The installation of full displacement penetrometers in the ground induces stress and/or pore pressure increases to the surrounding soil as a result of the imposed strains. The nature of the pressure increase will depend on whether the penetration process is drained, undrained, or partially drained. In order to back-figure the initial in situ stress conditions from the full displacement measurements, the pre- and post-penetration stresses should be related:

$$\sigma_{FD} = \sigma'_h + \Delta \sigma + u_0$$  \hspace{1cm} (6.29)
where
\[
\begin{align*}
\sigma_{FD} &= \text{total stress measured by the penetrometer} \\
\sigma'_{h} &= \text{initial in situ effective lateral stress} \\
u_o &= \text{equilibrium pore pressure}
\end{align*}
\]

For drained penetration, \(\Delta \sigma = \Delta \sigma'\) since \(\Delta u = 0\). Under undrained conditions

\[
\Delta \sigma = \Delta \sigma' + \Delta u \quad (6.30)
\]

although it is generally assumed that no change in effective stress occurs. As demonstrated in Chapter 5, the magnitude and variation of the induced pressures are also dependent on both probe geometry and soil characteristics.

Theories to evaluate the measurements obtained during penetration comprise the following:
- bearing capacity approaches
- cavity expansion methods
- finite element techniques
- strain path methods

The basic ideas behind each of these approaches are outlined in Appendix B and will not be discussed further here. Of the listed methods, cavity expansion (CEM) formulations have been widely used to model cone penetration, and their application is discussed below.

6.4.2 Undrained Cavity Expansion in Clay

Cavity expansion methods consider the monotonic increase in stress required to open and maintain a right cylindrical or spherical cavity in soil.
The theory was initially developed for indentation analysis in metals but was subsequently adapted to problems in soils (Hill, 1945; Nadai, 1950; Gibson and Anderson, 1961), whereby the cavity expansion is assumed to represent either:

- the conditions arising due to the full-displacement penetration of a cylindrical pile or penetrometer (Clark and Meyerhof, 1972; Roy et al., 1975)
- the stress path caused by the expansion of the membrane on a self-boring pressuremeter (Baguelin et al., 1973).

Recent developments in the application of CEM to penetration problems include:

- analysis of the contraction (unloading) portion of the pressure-volume strain curve to provide information on soil parameters (Houlsby et al., 1986; Houlsby and Withers, 1988).
- analysis of the cone pressuremeter test where the pressuremeter expansion is modelled as the continued expansion of the cylindrical cavity resulting from cone penetration.

The validity of the cavity expansion process to modelling cone penetration requires some comment.

Early studies of the displacements around piles in clay have shown that the installation process lies between the conditions associated with the expansion of a spherical and a cylindrical cavity (Clark and Meyerhof 1972; Roy et al., 1975). The plane strain constant volume condition of the cavity expansion process in clay has been confirmed by radial displacement measurements both in the field and in the laboratory (Randolph et al., 1978; Cook and Price, 1973). These studies show that once the penetrometer or pile tip has passed a particular level, little additional vertical movement of the soil occurs. Randolph et al. (1979) conclude that if the residual shear stresses on the pile are ignored, then the stress changes adjacent to the pile
(or penetrometer) will be similar to those predicted using cylindrical cavity expansion theory. The plane strain nature of deformations and the applicability of the cylindrical cavity model has been theoretically confirmed for soft clays by the strain path method (Baligh and Levaloux, 1986; Teh and Houlsby, 1988). The stress and strain paths resulting from full displacement penetration are however very different from those associated with cavity expansion and the discrepancy between the two depends on various factors. Norbury and Wheeler (1987) show that for an elastic plastic soil, the deviation between the theoretically more correct strain path solution and the less rigorous cavity expansion solution increases as the tip angle of the penetrometer increases. However as the soil moves further away from the tip, both Baligh (1986) and Teh (1987) show that cavity expansion is a reasonable description of the strain distribution around a cone shaft. Teh's analysis suggests that a minimum distance of 10 diameters (L/D=10) is required for similar strain distributions for the two cases. The results obtained in Chapter 5 would suggest a smaller L/D ratio of 4, after which no further stress variation occurs other than that associated with the dissipation of excess pore pressure, if generated. This difference may result from the simplifying assumptions concerning soil behaviour used in the theoretical studies and general differences resulting from probe geometry and its effects on measured response, i.e. tolerances along the shaft, finite length of pressuremeter section, etc. More importantly the selection of the appropriate soil parameters for use in the theoretical models is often arbitrary. Recommendations exist as to which values of strength and stiffness to apply but agreement between field and theoretical studies varies considerably.

Present understanding considers the penetration of a cone and the expansion of a pressuremeter as a mixture of spherical and cylindrical cavity
expansions. In the case of the penetrometer, the spherical condition refers to
the penetration of the tip which then reverts to the cylindrical case as the
soil moves up the penetrometer shaft. Self-boring pressuremeter expansion is
considered initially as a cylindrical cavity expansion which then progressively
transforms to a spherical condition as the deformation increases and end
effects (finite PM length) become important. From this, it is evident that the
strain path associated with full displacement pressuremeters is even more
complex. Furthermore, since soil behaviour is stress/strain path dependent any
attempt to model the change in stress in this type of in situ test is subject
to some degree of empiricism.

This aside, an attempt is made to use cavity expansion theory to evaluate
the stresses measured during full displacement probe installation. Emphasis is
given to the stresses measured with the lateral stress cone and the full
displacement pressuremeter. The Randolph et al. (1979) and Houlsby and Withers
(1988) theories have been used for this purpose.

Cylindrical Cavity Theory - Randolph et al. (1979)

Randolph et al. (1979) point out that the use of elastic plastic material
properties in cavity expansion theory does not take account of:
- pore pressures generated in shear
- dependence of soil strength on effective stress and stress history

Randolph et al. (1979) use a modified Cam clay soil model incorporated into a
finite element program to model the installation of pile in clay in terms of
the cavity expansion problem. They then derive the following expressions based
on the finite element results:
- the mean total stress increase, \( \Delta p \), during cavity expansion can be estimated
  by:
\[ \Delta p = S_u \ln \left( \frac{G}{S_u} \right) \]  \hspace{1cm} (6.31)

- the maximum pore pressure generated, \( \Delta u \), in a work hardening soil where the mean effective stress changes during shearing are given by:

\[ \Delta u = (p_i' - p_f') + S_u \ln \left( \frac{G}{S_u} \right) \]  \hspace{1cm} (6.32)

where \( p_i' \) and \( p_f' \) are the initial and final mean effective stresses, the difference of which provides an estimate of the pore pressure generated in shear.

- immediately after installation, the principal stresses are:

\[ \sigma_h' = \left[ \sqrt{3}/M + 1 \right] S_u \]  \hspace{1cm} (6.33)

\[ \sigma_v' = p' = \left( \sqrt{3}/M \right) S_u \]  \hspace{1cm} (6.34)

\[ \sigma_\theta' = \left[ \left( \sqrt{3}/M \right) - 1 \right] S_u \]  \hspace{1cm} (6.35)

where \( M \) is defined as:

\[ M = \frac{6 \sin \phi'}{3 - \sin \phi'} \]  \hspace{1cm} (6.36)

Randolph et al. (1979) compare predictions based on the above relationships and obtain good agreement with field data for piles in clay.

In terms of using cavity expansion methods to back-figure the in situ stress state, the method of Randolph et al. (1979) is hindered by the fact that an a priori knowledge of \( \sigma_h \) is required to calculate the initial mean effective stress. However, due to the magnitude of the induced pore pressure, errors in the estimated \( \sigma_h' \) are not so important.
The soil parameters used in Eqs. (6.31) to (6.36) have been obtained in two ways:

- \( G = G_{\text{max}} \) obtained from the seismic CPT has been combined with \( S_u \) obtained from the field vane test
- \( G \) and \( S_u \) have been determined from the Houlsby and Withers (1988) unloading analysis.

The first choice of \( G_{\text{max}} \) and \( S_u \) is based purely on the combination most convenient from an in situ testing viewpoint. \( G_{\text{max}} \) is readily determined from the downhole SCPT while \( S_u \) also provides another field standard.

The parameters from the Houlsby and Withers analysis are obtained graphically from the analysis of PM contraction during cylindrical unloading. The details of the graphical construction are shown in Fig. 6.21. Once \( S_u \) and \( G \) have been determined, the pre-penetration horizontal stress (\( \sigma_h \)) can be obtained from:

\[
P_L = \sigma_h + \frac{(2+m)}{3} S_u [(1+\ln(G/S_u)]
\]

where \( m=1 \) for cylindrical and \( m=2 \) for spherical cavity solutions. Both Houlsby and Withers (1988) and Hers (1989) report \( \sigma_h \) values much higher than expected when determined from Eq. (6.37) using either spherical or cylindrical formulations for full displacement pressuremeter tests in both soft and stiff clays.

Comparison with Field Data at Research Sites

The stresses and pore pressures calculated by the Randolph et al. equations and specified soil parameters were compared with field measurements at the three clay sites, each having different characteristics.
The calculated and measured total penetration lateral stress, pore pressure and effective lateral stress for McDonald Farm are presented in Fig. 6.22. The field data comprises $\sigma_{LS}$ measurements from LS-CPTU, $p_o$ from DMT and $p_L$ from SBPM. In this way it is assumed that the LS cone data represent a limit pressure, which should be the case if the soil around the cone is at failure and the cone penetration process can be modelled as a form of cavity expansion. It must also be borne in mind that at this site the lift-off pressures from the full-displacement pressuremeters agreed well with the LS cone data (Fig. 5.8). While the DMT is not a cylindrical probe and thus not correctly modelled by cylindrical or spherical cavity expansion theory, it is included on the figure.
Stresses and pore pressures measured during full-displacement penetration

Figure 6.22 Measured and predicted pressures in NC clay silt at McDonald Farm.
for comparative purposes. The following comments can be made concerning Fig. 6.22:

• the total lateral stress calculated using the Houlsby and Withers (HW) method slightly underpredicts $\sigma_{LS}$ whereas the $G_{max} - S_u$ (FVT) data greatly overestimates $\sigma_{LS}$ but is a good representation of the PM limit pressure. The dilatometer $p_o$ value lies midway between the limits of the calculated stresses.

• the penetration pore pressures ($u_o + \Delta u$) measured in the field are larger than those predicted by the HW analysis. The upper limit of pore pressure agrees well with the pressures measured at the $u_1$ location (Fig. 5.1). This is consistent with $N_{\Delta u}$ values calculated from data reported by Campanella, Sully and Robertson (1988):

$$\frac{\Delta u_1}{S_u} = N_{\Delta u} = \ln(G/S_u) \quad (6.37)$$

• the HW calculated and measured effective stresses are in good agreement which may indicate that the $S_u$ (HW) is the appropriate strength for this analysis since the effective stress term is independent of the shear modulus. If this is the case, and the theory is correct, it would imply that the HW analysis underestimates the $G$ required for cylindrical cavity interpretation. $N_{\Delta u}$ values for the $u_{LS}$ location suggest a $G/S_u$ of 115.

Hers (1989) reported that $S_u$ (HW) was generally less than $S_u$ (FVT) and attributed the difference to strain path differences during mobilization. It is also likely that the HW value represents some stage between peak and residual and is affected by the variation within the plastic zone caused by PM inflation. In this sense it may provide a good representation of the operative shear strength during cavity expansion.
Comparison of the \( G/S_u \) ratios for both cases suggests that:

\[
\frac{G_{\text{max}}/S_u \text{(FVT)}}{(G/S_u)_{\text{HW}}} = 9.5. \tag{6.38}
\]

which concurs with the data presented by Sully and Campanella (1990) for other sites in the Lower Mainland.

**Lr. 232 St.**

The profiles of measured and calculated penetration lateral stresses and pore pressures are presented in Fig. 6.23. For this site both the pore pressure and total lateral stress profiles are in good agreement with calculated ones based on \( G_{\text{max}} \) and \( S_u \text{(FVT)} \). However, both \( G \) and \( S_u \) parameter combinations overestimate the effective lateral stress after penetration — the calculated values of \( \sigma_h' \) are very similar since at this site since \( S_u \text{(FVT)} \) and \( (S_u)_{\text{HW}} \) are essentially the same (Hers, 1989). When applying this method to sensitive soils, Randolph et al. (1979) use a linear variation of \( S_u \) throughout the plastic region with:

\[
S_u = S_{ur} \text{ at } r = r_o \tag{6.39}
\]

\[
S_u = S_{up} \text{ at } r = R_p \tag{6.40}
\]

where \( S_{up} \) and \( S_{ur} \) are the peak and residual undrained shear strengths, \( r \) is the radius of the cavity, \( r_o \) is the penetrometer radius and \( R_p \) is the radius of the plastic zone. For Lr. 232 St., it is assumed that:
Stresses and pore pressures measured during full-displacement penetration.

Figure 6.23 Measured and calculated pressures in sensitive clay silt at Lr. 232 St.
\[ S_{up} = (S_u)_{FVT} = (S_u)_{HW} \]  

and the sensitivity is given by:

\[ S_t = \frac{S_{up}}{S_{ur}} \]

However it is not possible to evaluate the effective stress reduction that results from the sensitivity of the soil due to the ambiguity in the correct \( G \) value to use. This aside, a value of \( S_t \) between 3 and 5 is required to reproduce the field data if \( S_{ur} \) is used in Eq. (6.32). This compares with \( S_t \) of 14 to 23 for this site as determined from field vane tests. The results at Lr. 232 St. confirm the earlier conclusion that total stress analyses of the cavity expansion condition are more reliable for sensitive clays than effective stress based analyses.

**Strong Pit**

Figure 6.24 presents the same interpretation for the stiff OC clay silt data at Strong Pit. The field measurements of total penetration lateral stress and pore pressure are significantly smaller than the calculated values based on the Randolph et al. (1979) model and the two \( G-S_u \) combinations. No SCPMT data are available at this site so it was not possible to directly evaluate the HW rigidity index \( (G/S_u)_{HW} \). However evaluation of the data from both McDonald Farm and Lr. 232 St. suggests that:

\[ \frac{(G_{\text{max}}/S_u)_{(FVT)}}{(G/S_u)_{HW}} \text{SCPT} \approx 9-10 \]

Based on the above, a value of 9.5 has been estimated for Strong Pit.
Stresses and pore pressures measured during full-displacement penetration

Figure 6.24 Measured and calculated pressures in stiff OC clayey silt at Strong Pit.
As discussed earlier in Chapter 5, as the soil passes the cone tip, the soil is unloaded and normal stress relief occurs. The effect of the unloading appears to be proportional to soil strength and stiffness and is thus more pronounced in stiff OC soils than in soft NC materials. The field data in Chapter 5 illustrate this and it would appear to be confirmed here. Obviously in stiff soils, the poor correlation between measured and calculated total stresses results from no consideration of the strain path followed by the material. In soft soils the unloading is much less and insignificant in terms of the magnitude of the parameters being measured. Stress relief is also a function of the probe characteristics, although the effect of this factor is common to all the sites studied here as the same equipment is being used.

6.4.3 Drained Cavity Expansion in Sand

Cone penetration occurs under drained conditions in the sand deposits at the two research sites described earlier in the thesis (McDonald Farm and Laing Bridge South). The theory of cavity expansion for sand differs from that for clay in that volume change occurs and this must be considered when computing the radius of the plastic zone, \( R_p \) (Vesic, 1972).

The equations presented by Vesic (1972) are given in Appendix B. In simplified form they can be written (for cylindrical cavity expansion in a \( c'-\phi' \) soil) as:

\[
P_{cc} = c'F_c + F_q \sigma'_h
\]  

(6.44)

In sand, \( c'=0 \) and \( F_q = f(I_{rr}) \), so:

\[
P_{cc} = f(\sigma'_h, I_{rr})
\]  

(6.45)
where $P_{cc}$ is the cylindrical cavity limit pressure and $I_{rr}$ is the reduced rigidity index (which depends on the volume change during expansion). $P_{cc}$ is very sensitive to changes in $I_{rr}$, and not very suitable for back-analysis purposes. The closed form solution of Carter et al. (1986) is similar to that presented by Hughes et al. (1977) except that the former considers elastic strains in the plastic region - which are important at the large strains involved in the cavity expansion process. Furthermore, the Carter et al. (1986) method can be applied to the condition of initial cavity radius equal to zero (i.e. full-displacement installation).

The equations of the Carter et al. (1986) method are given in Appendix B. For clarity, the equations have been reduced by computing the constants involved for the cylindrical cavity based on a $\phi_{cv}$ of 33° for both McDonald Farm and Laing Bridge South (Howie, 1991). However, the choice of $G$ is problematical since it is dependent both on stress and strain levels. Sully and Campanella (1989) evaluated DMT data and found that the ratio $E_D/G_{\text{max}}$ is close to unity, at least for the Laing Bridge South and McDonald Farm deposits (Fig. 6.25). $E_D$ is measured after installation of a flat DMT blade and thus represents a large strain measurement within a disturbed soil annulus. In this particular case, it would appear that the opposing effects of stress and strain history cancel. It is convenient then to apply $G_{\text{max}}$ to the cavity expansion problem in sand when interpreting full-displacement probe data where PM data are not available. Where PM unload-reload data are available, several methods exist for evaluating $G$ (Byrne et al., 1990; Belloti et al., 1989) but the selection of the appropriate value for the full-displacement condition is complicated.

Other parameters used in the Carter et al. (1986) model are:

- Poisson's ratio, $\nu$, assumed equal to 0.1
Figure 6.25 $G_{\max} - E_D$ ratios for published data on sand (modified after Sully and Campanella, 1989)

- dilation angle, $\psi$, determined according to Bolton's (1986) stress dilatancy theory which also requires some estimate of $K_0$.

The expression of Carter et al. (1986) for a cylindrical cavity ($c'=0$) then reduces to:

$$\frac{2G_{\max}}{\sigma_h'} = 0.343 \left( \frac{p_L}{\sigma_h'} \right)^{4.4} - 0.293 \left( \frac{p_L}{\sigma_h'} \right)$$  \hspace{1cm} (6.46)$$

where $p_L$ is in terms of the effective stress. Equation (6.46) is ill-conditioned for evaluating $\sigma_h'$ since small errors or variations in $G_{\max}$ and $p_L$.
may give rise to unacceptably large errors in $\sigma'_h$. This, however, appears to be universal when interpreting large strain parameters to estimate small strain quantities. It is also interesting to note that the above equation suggests that $G_{\text{max}}$ (or shear wave velocity) should be related to $K_o$. In using Eq. (6.46) with the LS cone it has been assumed that $\sigma'_{LS}$ is an effective limit pressure and hence $p'_L = \sigma'_{LS}$.

Figure 6.26 presents the cylindrical cavity solution results compared against field measurements. The measured field data fall considerably below the calculated values. At McDonald Farm the ratio of calculated to measured effective lateral stress lies between 6 and 8 with only a minor apparent effect of relative density. This ratio varies between 3 and 5 at Laing Bridge South. Two principal effects are considered to be responsible for the large difference in the calculated and measured values:

- firstly, the use of $G_{\text{max}}$ in Eq. (6.46) may be inappropriate; and
- secondly, the effect of the unloading as the soil passes the cone shoulder considerably reduces the stresses acting on the shaft of the penetrometer. This confirms the earlier suggestion of Hughes and Robertson (1985) that the stress state close to full displacement probes is low due to unloading as the soil passes the tip.

The good agreement between the DMT $p'_s$ and calculated cavity expansion lateral stress would indicate that $G_{\text{max}}$ is an appropriate modulus. The effect of unloading is considered to promote the low stress condition (relative to the calculated limit pressure) measured by the lateral stress cone and hence in stiff soils, where unloading is an important effect of cone geometry, $\sigma'_{LS}$ cannot be interpreted as a limit pressure. This is also confirmed by the results in the stiff clay silt at Strong Pit.
Figure 6.26 Comparison of field measurements of lateral stress with values predicted from cylindrical cavity expansion in sand.
6.5 Empirical Approaches to Evaluate $K_0$

The in situ determination of $K_0$ by full-displacement penetrometers is a relatively new area of research, since most of the earlier work has relied on $K_0$ profiles determined either from laboratory tests on undisturbed samples or from self-boring pressuremeter tests. As experience with self-boring pressuremeters became more widespread it became apparent that, even under ideal conditions of installation, errors existed between results obtained from PM tests and actual in situ conditions. This has been suggested by the use of calibration chamber tests with clean sands (Bellotti et al., 1987). Improvements in the PM technique (both installation and probe expansion) are required to improve reliability of data.

As a consequence of the difficulties associated with self-boring probes, full displacement (FD) probes were developed whereby a particular indicator of lateral stress or strain history could be measured in a modified disturbed soil. The post-penetration stress has been shown earlier to be dependent on the initial pre-penetration stress state. Empirical correlations have been developed for interpreting FD probe data in order to recover the initial state variable $K_0$. Similar correlations also exist for OCR, and many of these type of corrections have been developed as a result of chamber testing (usually with sands) or based on field obtained reference profiles (SBPM, SBLC, TSC, HFT, etc.).

Appendix A lists and critically discusses 17 methods which can be used to provide direct or indirect assessments of the in situ horizontal stress. Nine of these methods have been used at some or all of the UBC research sites. The empirical correlations presently in use for the following full displacement tests are considered and applied to the results of the in situ testing programme:
- Dilatometer Test (DMT)
- Lateral Stress Cone Penetration Test (LS-CPTU)
- Full Displacement Pressuremeter Test (FDPMT)
- Field Vane Test (FVT)
- Shear Wave Velocity Measurements (Downhole and Crosshole)

The empirically correlated K values are compared with the reference profiles discussed in Section 6.2. Methods for estimating lateral stresses from CPTU in sand and clay have been discussed in Section 6.3 and will not be considered here.

In the following, the lateral stress coefficients determined from empirical correlations are denoted $K_{sub}$ where the subscript refers to the test method or interpretation technique. $K_{sub}$ may or may not correspond to $K_o$ according to the "correctness" of the empirical correlation. A discussion of the correlations is presented in Appendix A. Only the relevant equations will be reproduced here.

6.5.1 Dilatometer Correlations for $K$

The correlations developed for the DMT all relate to the horizontal stress index, $K_D$, defined as:

$$K_D = \frac{P_0 - u_0}{\sigma_v'} = \frac{P_0}{\sigma_v'}$$  \hspace{1cm} (6.47)

In Marchetti's (1980) original paper, $K_D$ was related to $K_o$ in the form:

$$K_o = (0.667 K_D)^{0.47} - 0.6$$  \hspace{1cm} (6.48)
which was suggested for both sand and clay. Subsequently, different relationships have been suggested according to soil type.

**DMT Correlations in Sand**

Schmertmann (1983) presented the following relationship based on limited CC tests:

\[
K_o = \frac{40 + 23K_D - 86K_D(1-\sin\phi') + 152(1-\sin\phi') - 717(1-\sin\phi')^2}{192 - 717(1-\sin\phi')} \tag{6.49}
\]

where \(\phi'\) is the triaxial friction angle determined from CPT or DMT data.

Baldi et al. (1986) reviewed all available CC DMT data and derived the following expression which was obtained after being modified to provide a field-calibrated relationship for Po River sand:

\[
K_o = 0.376 + 0.095K_D - 0.0046(q_c/\alpha') \tag{6.50}
\]

The profiles of \(K_{\text{DMT}}\) obtained from the two empirical equations are shown in Fig. 6.27. Also shown on the figure are the SBPM data and \(K_{\text{CPT}}\) from the correlation presented in Section 6.3. The following comments are relevant to both profiles:

- the Schmertmann (1983) method considerably overestimates the reference \(K_o\)
- \(K_{\text{CPT}}\) is in good agreement with the self-boring probe reference profiles
- \(K\) from the Baldi et al. (1986) expression and calculated using \(\phi'_{\text{DMT}}\) are in very good agreement and generally form a lower bound to the reference profile compared to the other DMT methods. The Baldi et al. method agrees well with the reference SBPM data.
Figure 6.27 $K_{DMT}$ profiles in sand.
It would appear that good estimates of $K_v$ in sand can therefore be obtained from empirical correlations using full-displacement probe measurements. Of the correlations available, the Baldi et al. (1986) method (using DMT data) and the Sully and Campanella (1989) method (using CPT data) appear to provide the best match to the reference profiles for the two sand sites considered here.

**DMT Correlations in Clay**

Lacasse and Lunne (1988) and Powell and Uglo (1988) present essentially the same relationship for clays based on field data from soft to stiff clays:

$$K_v = a (K_D)^m$$  \hspace{1cm} (6.51)

where

- $m = 0.44$ (high PI) to 0.64 (low PI) \hspace{1cm} Lacasse and Lunne (1988)
- $a = 0.34$

- $m = 0.55, a = 0.34$ \hspace{1cm} Young clays \hspace{1cm} Powell and Uglo (1988)
- $m = 0.54, a = 0.68$ \hspace{1cm} Aged clays

In Appendix A, the above methods have been reviewed to produce a correlation independent of clay type by considering the undrained strength ratio and PI. The basic form of Eq. (6.51) has constants $a = 0.34$ and $m = 0.55$. The correction has been evaluated to eliminate the differences in $K_v$ obtained from Eq. (6.51) and the field reference values. The following empirical correlation would appear to provide a good fit for the 13 sites where data have been published:

$$K_v = 0.34(K_D)^{0.55} - \left[ \frac{15-\text{PI}}{\text{PI}} \cdot 0.5\left(\frac{S_U}{\sigma_v}\right)_{\text{DMT}} \right]$$  \hspace{1cm} (6.52)
The $K_{DMT}$ profiles obtained in the clay silt at the three research sites are shown in Fig. 6.28.

At McDonald Farm, all $K_{DMT}$ estimates give essentially the same result and agree reasonably well with the SBPM data. At Lr. 232 St. and Strong Pit, the Marchetti (1980) correlation overpredicts $K_o$ by an amount which increases as the OCR (and stiffness) of the soil increases. The Powell and Uglow (1988) correlation (PU) and the correction to it presented here (PUCOR) bracket the reference values of $K_o$. (At Strong Pit the average PI = 15% and so no correction is required.) The LS oedometer data agree well with the PU and PUCOR data at Lr. 232 St. but are much lower at Strong Pit. (In the latter case, this may result from the low value of exponent in the $K_o$-OCR relationship determined in the laboratory due in part to friction cross talk effects and effects of initial stresses in the sample.) The Powell and Uglow (1988) correlation and the corrected relationship determined here would appear to provide a good basis for estimating $K_o$ in fine grained soils across the range of stress histories encountered during this research.

6.5.2 LS Cone Correlations for $K$

**LS-CPTU Correlations in Clay**

In Chapter 5 when the LS cone data were presented, it was suggested that, in non-sensitive NC to lightly OC silts and clays, the $K$ value could be estimated on the basis of effective stresses during penetration, i.e.,

$$K_{LS} = \frac{\sigma_{LS} - u_{LS}}{\sigma_v}$$

(6.53)

In more overconsolidated soils, this procedure overestimates the in situ $K_o$ value since the effective stresses measured by the cone are higher than the pre-penetration stresses. Hence in OC soils, the lateral stress during
Figure 6.28 $K_{DMT}$ profiles in fine grained soils.
penetration should be corrected for over-read in a manner similar to that employed for the TSC. Due to the limited data in OC soils it was not possible to evaluate the magnitude of the correction and the factors affecting it.

An alternative solution in silt and clay is to adopt the empirical technique suggested by Lacasse and Lunne (1983), originally employed with pressuremeter limit pressures, which can be used to evaluate the total stress increase due to probe installation. The total stress analysis is performed for Lr. 232 St. and Strong Pit using the Houlsby and Withers (1988) unloading analysis to determine \( G \) and \( S_u \) from field vane tests. Hence:

\[
K_{LS}^T = \frac{\sigma_{LS} - S_u (1 + \ln I_r) - u_s}{\sigma_v^T}
\]  

for the cylindrical cavity condition, where the superscript \( T \) denotes that \( \sigma_h \) is obtained from a total stress analysis of the stress increase caused by penetration. (The \( K \) value in all cases is the ratio of the effective vertical and horizontal stresses, but \( \sigma_h \) itself may be obtained from an effective or total stress analysis of the full-displacement penetration measurements.)

The profiles of \( K_{LS}^T \) are compared with reference data in Fig. 6.29. The \( K_{DMT} \) profile shown is that obtained from the Marchetti equation which has been shown to considerably overestimate the reference value. The agreement of the \( K \) values derived according to Eq. (6.56) is good at both sites, although at Strong Pit above 4 m the match progressively worsens towards the ground surface. The \( K_{LS}^T \) values estimated on the basis of cylindrical cavity expansion theory, assuming that \( \sigma_{LS} \) is equivalent to a limit pressure, appears to correlate reasonably well with in situ lateral stress measurements with the selected \( G \) and \( S_u \) values. Earlier calculations based on the theory of Randolph et al. (1979) indicated similar results. Hence the higher than expected near-surface
Figure 6.29 $K_{LS}^T$ profiles for Lr. 232 St. and Strong Pit (Sully and Campanella, 1990).
\( K_{LS}^T \) values at Strong Pit result from the theory of not being cable to consider soil unloading at the tip singularity.

**LS-CPTU Correlations in Sand**

The lateral stress coefficient from LS cone profiling in sand is defined in Eq. (6.53). The \( K_{LS} \) profiles for the two sand sites are shown in Fig. 6.30 where it can be seen that in general \( K_{LS} > K_0 \). \( K_{LS} \), in fact, appears to agree well with the Schmertmann (1983) correlation for \( K_{DHT} \) which has been shown earlier to overestimate the reference profile. To evaluate the amplification of lateral stress caused by cone penetration, \( \sigma'_{LS} \) has been normalized by \( \sigma'_h \) where \( \sigma'_h \) has been calculated from \( K_0 \). The data are replotted against \( q_c/\sigma'_v \) which is an indicator of relative density (Fig. 6.31). The amplification factor for the lateral stress cone, \( A_{LS} \), is defined as:

\[
A_{LS} = \frac{\sigma'_{LS}}{\sigma'_h}
\]  

(6.57)

Two effects are apparent from the trends shown in Fig. 6.30:
- the amplification factor, \( A_{LS} \), is sensitive to small changes in the \( q_c/\sigma'_v \) ratio
- the shape of the \( A_{LS} - q_c/\sigma'_v \) relationship appears to be similar for the two sites studied; however, the curves are offset laterally due to some secondary effect

The secondary effect on the data trend may be caused by, inter alia, grain characteristics (shape, size, angularity, etc.). The data highlight the problems of applying numerical techniques to natural sand deposits and how the effects of subtle variations in soil characteristics can significantly modify response under given conditions.
Figure 6.30 $K_{LS}$ factors for sand research sites (Campanella, Sully, Greig and Jolly 1990).
Figure 6.31 Lateral stress amplification factor for sands tested (Campanella, Sully, Greig and Jolly, 1990).
From Fig. 6.30, the general trends are:

\[ K_{LS} = K_{DMT} = 1 - 1.2 (K_0) \] for McDonald Farm

where \( K_0 = 0.55 \), and

\[ K_{LS} = K_{DMT} = 1.5 - 2.5 (K_0) \] for Laing Bridge South

where \( K_0 = 0.5 \). However, the reference SBPM profile for both sites suggests a higher lateral stress coefficient. In fact, if the self-boring probe data are taken as the true reference data, then the \( K_{LS} \) profiles are representative of \( K_0 \). This is evidenced by the good correlation in sand between \( \sigma'_{LS} \) and \( \sigma'_{SBPM} \) (Figs. 5.21 and 5.22).

The resolution on the present LS cone is 7 kPa, which needs to be improved in order to provide answers to the problem of lateral stress measurement. Refined signal processing and a modified lateral stress sleeve (under construction) will improve the resolution to about 1.0 kPa. This will hopefully reduce the data scatter and permit more detailed analysis of in situ test data.

6.5.3 Full Displacement Pressuremeter Correlations

As discussed previously, limit pressures obtained from pressuremeter expansion can be interpreted by means of cavity expansion solutions. If the pressuremeter is installed by self-boring and no significant soil unloading occurs then the cavity expansion should provide a reasonable solution to the problem. Full displacement pressuremeters may require radial strains large enough to extend the plastic zone due to PM inflation out past the plastic
radius set up due to the cone penetration aspect of the installation, if the cavity expansion solutions are to provide reliable interpreted data.

Hers (1989) analyzed the SCPM limit pressure data at three clay sites using the empirical approach of Lacasse and Lunne (1983) described in the previous section. The measured limit pressures were interpreted by both spherical and cylindrical cavity solutions using the Houlsby and Withers (1988) unloading parameters. At all three clay sites (McDonald Farm, and Lr. 232 St. included), the $K_{\text{SCPM}}$ values from the spherical and cylindrical solutions greatly overpredicted the reference $K_o$ value (Hers, 1989). The same interpretation method applied to SBPM limit pressures gave $K_{\text{SBPM}}$ in good agreement with $K_o$. The overprediction is thought to result from the full-displacement mode of installation, the length-diameter ratio of the probe and the maximum cavity strain attainable (i.e. probe characteristics). It is interesting to note that field data indicate that the SCPM lift-off pressure is the same as $\sigma_{\text{LS}}$ from the LS cone. Furthermore, the assumption that $\sigma_{\text{LS}}$ is a limit pressure and the fact that evaluation by means of the Lacasse and Lunne (1983) empirical procedure gave $K_{\text{LS}}$ values in reasonable agreement with the reference $K_o$ at both Lr. 232 St. and Strong Pit (Fig. 6.29), would imply that, as stated earlier, the expansion of the PM on a full displacement (normal) probe cannot be treated as the continued expansion of a cavity (spherical or cylindrical) unless the PM expansion pushes the plastic zone out past the limit previously established when the cone was inserted. It may also suggest that due to the finite L/D ratios of the PM sections in use, the strains set up in the soil do not satisfy the cylindrical plane strain condition, giving rise to a condition intermediate between spherical and cylindrical expansion. Furthermore, drainage during PM expansion may be a factor in more permeable clay soils resulting in larger limit pressures than would occur for the totally undrained case. The
pressuremeter still requires further basic research to examine these factors for all soil types.

Byrne et al. (1990) have presented a numerical technique for analyzing PM unload-reload loops to obtain $G_{\text{max}} (G_{HH})$. By comparing the $G_{HH}$ (SBPM) (corrected for disturbance effects) with $G_{VH}$ from downhole or crosshole seismic velocity measurements, Byrne et al. (1991) present the following relationship:

$$G_{VH} = \left( \frac{G^*}{\alpha} \right) \alpha_D \alpha_A$$  \hspace{1cm} (6.58)

The disturbance factor, $\alpha_D$, is shown to be about 1.4 for both SBPM and FDPM tests. $\alpha_p$ is determined from a nomograph, and

$$\alpha_A = \left( \frac{1+K_o}{2K_o} \right)^{0.5}$$  \hspace{1cm} (6.59)

Hence, if $G^*$ is measured in situ, the test conditions provide $G_{\text{max}} (G_{HH})$ using the Byrne et al. (1990) nomograph. $G_{VH}$ can be obtained from shear wave velocity measurements and so $\alpha_A$ can be evaluated to provide an assessment of $K_o$. Byrne et al. (1991) perform this analysis to show that $\alpha_D = 1.4$, i.e. by assuming $\alpha_A$ (based on site measurements). Alternatively, assuming $\alpha_D = 1.4$ will result in $\alpha_A = 1.17$ and thus provide the correct $K_o$. The calculated value of $K_o$ however is sensitive to the $G_{VH}/G_{HH}$ ratio. Analysis of the Byrne et al. (1991) data shows that $\alpha_A$ could be as high as 1.97 which results in $K_o = 0.35$. This lower bound is in agreement with the lower bound estimates of the reference $K_o$ profile discussed earlier.

The Byrne et al. (1990) procedure was applied to SCPM unload reload moduli obtained at Laing Bridge South. The ratio of $G_{VH}$ to $G_{HH}$ ($G^*/\alpha_p$) varied from 1.17 to 1.24, which is equal to $\alpha_D \alpha_A$. If $\alpha_D$ is 1.4 as reported by Byrne et
al. (1991), then $\alpha_A = 0.83-0.89$, less than the expected value of 1.18 ($K_0 = 0.5-0.6$). The quality of the PM data is considered to be reasonably good (Howie, 1991) and may require $\alpha_D$ smaller than that calculated for McDonald Farm. Assuming $\alpha_D = 1$, then $\alpha_A = 1.17-1.24$ and the resulting $K_0$ varies between 0.48 and 0.57.

The method suggested by Byrne et al. (1990) provides a theoretical basis for evaluating PM and SCPT data to determine $K_0$ but the results are dependent on the factor $\alpha_D$. Further comparisons based on field and laboratory results may provide a way of standardizing the correction for the effects of disturbance associated with different types of PM. Soil type characteristics may also influence $\alpha_D$ in a similar way to that presented for the lateral stress amplification factor, $A_{LS}$.

6.5.4 Indexing $K_0$ by Shear Wave Velocity Measurements

The results of Lee (1985) and Lee and Stokoe (1985) suggest that the elastic shear wave velocity in sand is a function of the effective stress state and that the stress state can be evaluated from crosshole and downhole velocity measurements. From measurements in a large cubical chamber, they suggest that the dependence of the shear wave velocity on stress is best described by the individual stress method:

$$V_s = C \sigma_a^{na} \sigma_a^{nb} \quad (6.60)$$

where the parameters are as defined in Chapter 5 and Appendix A.

Yan and Byrne (1990) performed downhole and crosshole tests in a hydraulic gradient model test set-up and concur with the results of Lee and Stokoe (1985). Both studies suggest the use of the crosshole-downhole velocity ratio
(V_{DH}/V_{XH}) as a means of evaluating $K_o$. In Chapter 5, field data were presented to show that this velocity ratio may be more influenced by structural rather than stress anisotropy. Furthermore the theoretical variation of the velocity ratio against changes in $K_o$ was shown to be illconditioned for determining $K_o$. In situ downhole and crosshole shear wave velocities also presented in Chapter 5 do indicate stress dependent effects and may provide the best route to obtaining information on $K_o$ changes. The drawback to the method is that the shear wave velocity constant in Eq. (6.60) is required in order to back-figure $\sigma''_h$. Consequently, the value of $K_o$ determined in this way will depend on the errors in the velocity constant $C$.

The error in $C$ due to the selection of stress method was evaluated for the LBS sand where a $K_o$ value of 0.55 was assumed for the whole profile. For the individual stress method $n_a = n_b = 0.125$ was used and $n_t = 0.25$ was taken for the average stress method. (These values are considered representative for the soils present.) For both stress definitions the same value of $C$ was obtained:

$$C_{AVG} = 59.23$$

$$S.D. = 6.62$$

Conversely, if a $C = 59.23$ is applied to the field data, then the statistics for $K_o$ for each of the stress methods are as follows:

Individual stress method: $(K_o)_{AV} = 0.751$

$$S.D. = 0.605$$
Average Stress Method: \( (K_0)_{AV} = 0.594 \)
S.D. = 0.678

Clearly, the variations in \( v_s \) are such that a single average velocity constant cannot be used to estimate \( K_0 \). Further development of this approach in sands is required if it is to be of any practical use. It may be more appropriate to use \( G_{\text{max}} \) rather than \( v_s \) in this correlation in a form similar to the pressure-meter approach described earlier. This, however, involves a velocity squared term so that variations in \( v_s \) are accentuated further.

In fine grained materials it may be possible to combine field and laboratory shear wave velocity data to arrive at an estimate of \( K_0 \). This is discussed with reference to the resonant column tests performed on undisturbed samples from Lr. 232 St. reported by Zavoral (1990).

From the results of resonant column tests on isotropically consolidated samples from Lr. 232 St., the following relationship was obtained for \( G_{\text{max}} \) in terms of confining stress \( (\sigma'_3) \) for depths greater than 6 m (Zavoral, 1990):

\[
G_{\text{max}} = 292.1 \left( P'_a \right)^{0.1} (\sigma'_3)^{0.9} 
\]

From field data:

\[
G_{\text{max}} = \rho v_s^2
\]

For stress ratios less than 2.5, the deviatoric component of stress has no effect on \( v_s \) (Hardin and Black, 1968), and the shear wave velocities from isotropic stress conditions can be equated to the same level of mean normal stress, i.e. \( \sigma'_3 \) can be substituted with \( \sigma'_m \) or \( \sigma'_\text{oct} \) (Zavoral, 1990). The
results of $K$ determined in this way are shown in Fig. 6.32. Below 5 m the $K$ values derived from Eqs. (6.61 and 6.62) are in good agreement with reference values ($K_0$). Above 5 m, calculated values become progressively higher as the ground surface is approached and this may result from:
- the effect of OCR which is not considered in Eq. (6.61)
- the variation and uncertainty in $V_s$ in the variable near-surface sediments at Lr. 232 St
- the sensitivity of the method to changes in $\sigma_v'$ for near-surface conditions.

Due to a lack of resonant column data it was not possible to apply this technique at other Lower Mainland sites. It would appear to be a possible approach to obtaining realistic $K_0$ values and should be examined further. In sands, the technique may require a more complete definition of Eq. (6.61) and high quality undisturbed samples if reasonable results are to be obtained.

6.5.5 Correlations with Undrained Shear Strength

Wroth (1984) pointed out that, by simple analogy with the simple shear test the NC undrained shear strength of clay measured during the field vane test can be equated to $\sigma_h'$ in the form:

$$S_u = \sigma_h' \sin \phi'_{ps}$$

(6.63)

or, normalizing by $\sigma_v'$:

$$\frac{S_u}{\sigma_v'} = K_0 \sin \phi'_{ps}$$

(6.64)
Figure 6.32 Variation of $K$ as determined from shear wave velocity measurements in the laboratory and in situ.
where $\phi'_{ps}$ is the plane strain friction angle ($= 9/8 \phi'_{txl}$).

In Fig. 6.33(a) $K_{FVT}$ is taken to represent $K_0$ obtained from Eq. (6.64) using in situ field vane data. At Lr. 232 St. below 5 m the agreement is very good, but above 5 m the effect of OCR has to be considered since Eq. (6.64) is for NC soil. Assuming that $(K_0)_{OC}$ is a function of $(OCR)^{0.5}$, Eq. (6.64) becomes:

$$K_0(OCR)^{0.5} \sin\phi'_{ps} = S_u/\sigma_v$$

(6.65)

This modified relationship was applied to the Strong Pit data and the results are shown in Fig. 6.33(b). The $K_{FVT}$ values lie between the reference profile ($K_{TSC}$) and the LS oedometer profile ($K_{LAB}$). Further validation is required in a variety of soils but the method appears to provide reasonable and consistent values for $K_0$.

6.6 Conclusions

In Chapter 6, profiles of the reference of $K_0$ lateral stress have been determined using methods generally accepted as applicable to the soil conditions at the five research sites under study. The reference $K_0$ profiles have been used to evaluate various interpretation procedures with the aim of back-calculating the in situ pre-penetration lateral stress from full-displacement horizontal stress measurements.

Cavity expansion methods (CEM) have been used with some degree of success in both sand and clay. The main problems associated with this method of interpreting the stress increase caused by insertion of full displacement probes are related to stress/strain paths and parameter selection. From an in situ testing perspective it is appealing to use $G_{max}(SCPT)$ and $S_u(FVT)$ in the cavity expansion formulations although this only appears to be successful in
Figure 6.33 Estimated K values by means of in situ field vane data.
sensitive clays when applied to LS cone data. However, it would seem that this parameter combination gives calculated pressures that correspond well with PM limit pressures. The Houlsby and Withers parameters provide calculated profiles in reasonable agreement with field data in non-sensitive NC or slightly OC soils. In summary, CEM provide realistic data in soft sensitive and non-sensitive soils (provided appropriate parameters are selected) but grossly overestimates the stress measured in stiff OC clay and dense sand.

Several empirical approaches were reviewed and the following conclusions can be drawn:

• DMT - in sand the correlation presented by Baldi et al. (1986) provides a reasonable lower bound estimate of $K_0$. However, it is uncertain how sensitive the correlation is to stress history in OC sand.

- in clay the $K_D - K_0$ correlation suggested by Powell and Uglow (1988) produces consistent $K_0$ values at the sites researched. Based on a review of 13 sites, the Powell and Uglow correlation was modified to provide a relationship independent of soil type. The two empirical relationships, derived on the basis of numerous field studies agree well with the reference stress profiles at the clay sites.

• LS-CPTU - in sand the measured lateral stress undergoes amplification with respect to the pre-penetration value depending on the state of the soil through which the cone is penetrating. The amplification factor appears to be effected by soil characteristics such as grain size, angularity, etc. For the two sites studied, the sands have very similar characteristics, perhaps the main difference being
grain size distribution. However, at Laing Bridge South (fine sand) the amplification is approximately double that at McDonald Farm (medium to coarse sand). This emphasizes the effect of the "non-tangible" factors on the measured response and highlights the problems associated with the application of analytical/numerical techniques.

The empirical method proposed by Sully and Campanella (1989) to obtain $K_o$ from $q_c$ worked well in the Lower Mainland sand deposits and gave lateral stress profiles consistent with the reference values.

- in clay, the empirical solution of Lacasse and Lunne (1983) provided $K_{LS}$ values in reasonable agreement with the reference $K_o$ profile although the overprediction was higher at higher OCR reflecting again the effect of stress path on the measured pressures.

- **FDPMT** - in both sand and clay it appears that PM limit pressures may be predicted (calculated) with reasonable accuracy provided the expansion of the PM membrane is sufficient to erase the soil memory of the unloading that occurred during the cone penetration phase of the installation. From data presented here, $G_{max}$ may be the correct modulus to use in the cavity expansion formulation for evaluating limit pressures. The unloading analysis presented by Byrne et al. (1980) could be useful if the effect of disturbance could be classified in terms of the $\alpha_D$ parameter. $\alpha_D$ may also be subject to ensuring a sufficiently large cavity strain so as to overcome the effects of the initial unloading during cone
insertion. \( K_0 \) calculated from SCPT and SBPMT small strain moduli is very sensitive to \( \sigma_D \).

- **FVT** - studies to date have demonstrated the dependence of \( S_u \) from the field vane test on the in situ lateral stress. Provided changes in OCR are compensated for, reliable and consistent \( K_0 \) values can be deduced from undrained strength measurements. This is consistent with the already established \( S_u \)-OCR relationship for clays.

- **SCPT(\( V_s \))** - it does not appear feasible to determine \( K_0 \) based solely on in situ measurements of \( V_s \) since the results are very sensitive to the velocity constant, \( C \). However, combining field and laboratory (resonant column tests) measurements may provide a consistent approach to evaluating \( K_0 \). The usefulness of the method would however depend on how well the laboratory data reflected the in situ conditions and the sensitivity of \( V_s \) to stress ratio effects.
CHAPTER 7

7. CONCLUDING REMARKS

7.1 Overview

The thesis has explored the idea of using large strain parameters, measured during full displacement probe tests to evaluate the in situ pre-penetration lateral stress. Several factors related to both soil properties and probe geometry have been investigated which affect the lateral stress measurements. Some of these factors, especially those related to penetrometer design, can be controlled, or the effects minimized, in order to provide consistent and repeatable data. The effects of some of the factors become more important under given soil conditions, i.e. dimensional tolerances are relatively unimportant in soft soils but can completely dominate the measured response in stiff soils. Given the above, it is difficult to provide definite recommendations applicable in general to in situ full displacement probes. Some degree of standardization in probe design should be attempted, as in the case of the Marchetti dilatometer, so that results from a variety of soils can be analyzed in a consistent manner. Methods of interpretation of the measured lateral stress are empirically based – that is relying on field observation in conjunction with simplified theory based on hypothetic-deductive procedures. Results of interpretation techniques applied to the measured field data have highlighted some of the problems associated with the differences between idealized models and real soil behaviour. The interpretation methods applied herein have indicated that a single $K_0$ value cannot realistically be derived, rather that the emphasis should be on defining a limited range of possible variations.
The field data presented have demonstrated the importance of the horizontal effective stress on measured parameters from in situ testing. It would appear logical then to pursue the determination of lateral stress from in situ measurements. This approach has the advantage of providing several estimates of $K_0$ from a single sounding. Empirical methods for evaluating $K_0$ or $\sigma'_h$ have been discussed in the thesis. The reliability of estimates of this type will depend on the correlation used to predict $K_0$, the applicability of the algorithm used to interpret the field measurements, and the quality of the field data.

Correlations have been presented for predicting $K_0$ in both sand and clay for several types of probe. Of the probes available, the lateral stress cone appears to be very promising since $K_0$ can be evaluated from several contemporaneous measurements. Until these correlations become established for a wider range of soil types some reference $K_0$ values will be required to evaluate predicted values. In clays the reference values can be obtained from several techniques; laboratory lateral stress oedometers, installation of total stress cells or self-boring pressuremeter data. However, all of these methods provide reference values that may be disputed or in error to the same degree as for the empirically-derived values.

Much additional experimental research is necessary. It is considered unlikely that numerical methods will be able to resolve these major issues but can be used in conjunction with calibration chambers or field measurements for parametric studies. As discussed in the thesis, the results of calibration chamber tests have been instrumental in the development of the idea for obtaining small strain parameters from large strain measurements. From the data presented here, it is apparent that calibration chamber correlations should be applied with caution to a field environment. Ideally all CC derived correlations should be field calibrated for varying soil conditions.
Several CC correlations have been field calibrated during this study and have been shown to provide good estimates of the in situ $K_o$ reference values.

On a more general note, as discussed by previous researchers (Tavenas et al., 1975; Massarsch, 1979) the definition of $K_o$ is problematical especially for near surface data where small errors in the measured horizontal effective stress give rise to large changes in $K_o$. Similar problems were encountered when defining pore pressure parameters from CPTU. In that case, a pore pressure difference parameter was found to provide more consistent correlations.

To correlate the results of numerical analyses for differing stress ratios in clay, Houlsby and Teh (1988) define a horizontal stress factor, $\Delta$, based on a normalized stress difference:

$$\Delta = \frac{(\sigma'_v - \sigma'_h)}{2 S'_{u}}$$

(7.1)

which can vary between ±1. The adoption of a similar expression for evaluating in situ data may provide more consistent parameter correlations for soils with near-surface overconsolidated crusts. A similar definition can also be used for sands by replacing $S'_u$ by $\sigma'tan\phi'$.

The profiles of $K$ evaluated in Chapter 6 are generally in good agreement with the reference profiles. However, as stated above, small variations in the calculated horizontal stress give rise to relatively large differences in $K$. If the data were plotted as a measured or calculated stress, the agreement would be much better. The plots in terms of $K$ are probably the most unfavourable way of presenting the data. This aside, the agreement is good and it further emphasizes the applicability of the procedures used to the determination of realistic lateral stress profiles.
The research described herein has contributed to the state of present knowledge by:

• providing a detailed review of all existing in situ test methods and evaluating the standard interpretation procedures common to each one. In addition, the effects of the in situ lateral stress on large strain parameters have been evaluated and summarized so that index parameters sensitive to $K_o$ variations could be defined.

• nine different techniques to either measure or index in situ lateral stress have been considered. Both test procedure and interpretation methods have been reviewed and modified to provide for consistent $K_o$ predictions. A new non-destructive technique has been presented as a possible method for evaluating in situ $K_o$ from indirect measurements.

• index parameters have been hypothetically related to $K_o$ and the interdependence demonstrated by means of field data measured at 5 Lower Mainland research sites. The findings from the field data have been confirmed by a review of published data from other international well-documented research sites.

• a comprehensive systematic study of stress and pore pressure distribution around penetrating probes has been performed in soils of varying composition and stress history. Fundamental differences in soil response related to basic parameters have been examined using probes of varying geometry. The importance of probe geometry has been demonstrated in relation to shape (cylindrical or plate-like) and location of the measuring system. The fact that dimensional tolerances become more critical in soils as the stiffness increases has also been confirmed.
• cavity expansion methods have been employed to interpret the field measurements of stress and pore pressure in both sand and clay. It is apparent that these methods are valid only for soft soils where the effects of unloading are small. In stiffer soils where stress/strain path behaviour is more accentuated, the simplified analysis is inadequate and considerably overestimates the stresses acting on the penetrometer shaft. With full displacement probes, a limit pressure is only achieved if expansion of the probe section is carried out as in the case of a pressuremeter. If no expansion occurs, as in the case of the lateral stress cone, the measured stress may be much lower than the true limit pressure.

7.2 Equipment Details

Several types of in situ testing device have been used during this study. The following comments are related to possible modifications to the probes to allow for improved interpretation of the data.

• FVT - incorporation of a pore pressure transducer or total pressure cell on the vane would provide interesting data for evaluating stress and pore pressure changes during vane insertion and rotation. It may be possible to construct the vane as one large pressure cell.

• DMT - the UBC research dilatometer pioneered the measurement of pore pressures during DMT soundings by incorporating a transducer on the face of the expandable membrane. This is considered a necessary design modification to the standard DMT blade if research into lateral stress in terms of stress and pore pressure can be achieved using this equipment.
• LS-CPTU - this in situ testing device is probably the most versatile and promising for application to the problem of determining in situ lateral stress profiles. Modifications to the lateral stress sleeve are required however to improve data quality and sensitivity. The LS sleeve is currently being modified to use a thinner instrumented section (as opposed to the original 1 mm section). Signal processing is also being improved. It is likely that second generation LS sleeves will incorporate relocated lateral stress sections less sensitive to friction crosstalk effects. Consideration should also be given to providing accurate high resolution data for both $\sigma_{LS}$ and $u_{LS}$ with both sensors being closely spaced. These sensors should be located at least 4 diameters behind the tip to avoid the effects of unloading gradients that occur close to the tip as a result of the cone geometry. LS cones designed in this way will allow both calibration chamber and field data to be evaluated in a consistent manner. It would also be useful to modify the existing LS cone so that the stress sensing sleeve could be located at varying distances behind the cone tip. This would confirm some of the results presented here and the inferences made based on pore pressure distribution.

• FDPMT - problems with the FDPM have arisen as a result of a malfunction of the effective stress transducer. For the new pressuremeter the effective stress transducer has been replaced by a pore pressure transducer and should provide more reliable data. For a complete interpretation of PM data, measurement of pore pres-
sures during insertion and membrane expansion are necessary. Interpretation of the complete pressure expansion curve may provide the best solution for evaluating the pre-penetration in situ lateral stress.

- **TSC** - the pore pressure measuring system of the spade cell requires modification to ensure complete saturation. Provided equilibrium groundwater conditions are known, the poor saturation of the system is not a problem. However, if short term measurements are to be of use, direct reliable pressure measurements are required.

- **SBPM** - the new SBPM developed at UBC is presently undergoing tests but appears to have overcome some of the problems previously encountered with strain arm measurement. Interpretation of the complete pressure-expansion curve is thought to be the best approach for determining lateral stress, although effects of strain rate, finite pressuremeter length, creep, etc. all require further study as these factors significantly affect the shape of the curve. Installation of the probe with minimal disturbance is the key factor to future use of the SBPM for evaluating lateral stress.

7.3 **Suggestions for Future Research**

1) The installation of the self-boring pressuremeter still remains a problem which complicates the interpretation procedure. Detailed studies are required to examine basic drilling parameters especially for tests
in sands and stiff clays. For this to be accomplished, a homogeneous test site is required so that relevant comparisons of adjacent test hole data can be made. Laing Bridge South appears to satisfy these requirements for the sand stratum between 2 m depth and 18 m. Continued calibration chamber studies would help resolve many problems associated with PM design and provide data for evaluating interpretation techniques.

2) By obtaining good repeatable SBPM data in sand the horizontal stress can be determined using both the lift-off and curve fitting techniques. The curve fitting technique could then be applied to full-displacement pressuremeter data from the same site to verify the FDPM $\sigma_h$ values against SBPM results.

3) The lateral stress cone has demonstrated the usefulness of strain-gauged stress sensing friction sleeves for profiling stress changes. A circular type of lateral stress sensor has also been incorporated into the lateral stress tool (LAST) designed at UBC. This circular sensor could be located at several horizons above and below the PM membrane to evaluate the change in $\sigma_h$ as the soil is progressively sheared at the probe-soil interface. This may provide information on the ideal location for the PM lantern.

4) For full-displacement pressuremeters, the PM section should be longer than presently in use to ensure that expansion re-establishes a moving plastic radius while also minimizing end effects due to circular rather than cylindrical expansion.

5) The lateral stress cone shows considerable promise for profiling variations in the measured lateral stress. The results are, however,
affected by friction crosstalk. This can be minimized by re-designing the friction sleeve so that the stress sensitive underreamed section is on the section of sleeve that is in tension rather than on the main body which is in compression. This will reduce some of the noise on the signal.

6) A balance between wall thickness of the LS cone (and sensitivity) and durability is required if the thin-walled sleeve is to be used to measure lateral stress. The new section has been tentatively designed with a 0.75 mm wall thickness which will provide a resolution of around 1 kPa. However, improvements in the sensitivity of the friction load cell and the lateral stress pore pressure transducer are also required if reliable determinations of $K_0$ and $\delta$ are to be achieved.

7) Re-design of the LS cone tip may be useful for analytical interpretation of full-displacement data. The use of a rounded shoulder behind the tip, as in the case of the dilatometer, will produce lower stress and pore pressure gradients in this region as a result of a lesser degree of unloading. The application of cavity expansion methods may be more feasible in stiff soils if the degree of unloading can be reduced.

8) Further studies in calibration chambers of different size are warranted using probes of varying geometry. Lateral stress probes with stress sensitive sections located 4 diameters behind the tip offer the possibility of expanding the CC derived results to a field situation while at the same time not having to rely on stress measurements close to the tip.
9) The problems of in situ measurements of lateral stress are related primarily to sand and stiff clay/silt. Concerted attempts at different research centres should be made to evaluate results from a series of tests at each site so that reliable empirical relationships can be derived. This has been done with some success for the DMT. Similar studies are warranted for the LS cone and cone pressuremeter.

10) The development of non-destructive techniques would be an important step in the evaluation of the true in situ lateral stress (not subject to disturbance). The shear wave velocity technique was thought to be a good basis for developing this methodology. Results obtained here are not conclusive in this respect and further studies should be performed both in the laboratory and in the field. It may prove difficult however to distinguish between stress and structural effects on $V_s$ in the field.

11) To develop further the use of large strain parameters for predicting $K_o$, research should be conducted into the effects of stress history mechanisms on both full displacement induced lateral stress and pore pressure. Analytical and numerical studies using more complex soil models will be useful for explaining field behaviour in materials other than soft NC clays.
APPENDIX A

IN SITU MEASUREMENT OF LATERAL STRESS

- A CRITICAL REVIEW
APPENDIX A

IN SITU MEASUREMENT OF LATERAL STRESS - A CRITICAL REVIEW

A.1 INTRODUCTION

This chapter considers the various in situ test methods available for evaluating lateral stress conditions and critically reviews both the test method and the interpretation of data. However, all the methods discussed here are not generally accepted as techniques for evaluating $K_o$; rather some are $K_o$ indicators and can be used to evaluate trends or changes in stress conditions. These tests are considered here since they provide confirmation of the dependence between small strain $K_o$ and large strain parameters which is one of the main objectives of this thesis. In many cases the dependence of large strain parameters can be demonstrated from the results of calibration chamber tests and, where appropriate, these tests are also reviewed.

This chapter does not contain data obtained during the period of research for this thesis, it does however contain ideas for interpretation of test data, improvement of test methods and techniques for evaluation of lateral stresses using full displacement measurements. These ideas are discussed further in the thesis using data obtained from this research.

A.2 SELF-BORING PRESSUREMETER TEST (SBPMT)

A pressuremeter is a cylindrical probe with an expandable membrane close to its mid-section. Various types of pressuremeter are available; the different types being classified according to the method of probe installation. As the name suggests, the self-boring pressuremeter (SBPM) has the
capability to drill itself into the ground. The self-boring installation technique was developed simultaneously by research centres in France (Baguelin et al., 1972) and England (Wroth and Hughes, 1973) in order to reduce the disturbance normally associated with the Menard or prebored pressuremeter (see Section A.8). Consequently, the reliability of any geotechnical data from the SBPMT is related to the quality of the test in terms of the disturbance induced due to insertion of the probe. Disturbance may occur as a result of two dominant factors: (i) instrument related effects; and (ii) boring and installation effects. Notwithstanding this, the SBPM is generally acknowledged to be the optimum in situ test equipment for the direct measurement of in-ground lateral stresses. However, caution is generally recommended in the application of test results and it is usual to use other field or laboratory tests to provide additional confirmation.

The British and French type pressuremeters are somewhat different in their design. The British SBPM is about 1m long and has a diameter of 80mm with a 640mm long expandable section (Fig. A.1). The membrane and chinese lantern rest against a central rigid hollow cylinder. The outside diameter at the PM section and along the probe is equal to that of the cutting shoe in order to avoid stress relief and disturbance. After installation, the membrane is expanded by air pressure and membrane movement is measured by strain arms (usually at 120°) around the mid-section of the PM module. The local pore pressure is measured by a transducer on the membrane. The cavity pressure, P, strain arm movement, δ, and pore pressure, u, are monitored during the expansion stage.

The French probe, PAFSOR, is slightly larger than its British counterpart but the main difference is due to the non-rigid PM section. The internal rigid body of the probe has a smaller diameter than the cutting shoe
Fig. A.1 Details of the British-Type SBPM.
and consequently the probe is installed with the membrane slightly expanded so that the external diameter along the probe is constant. This is termed the zero position of the membrane since any inward or outward movement of the membrane will cause stress changes in the surrounding soil. The probe is expanded by fluid injection and both fluid pressure and PM cavity volume are monitored. The assumption of a constant diameter along the PM section permits the calculation of radial strain.

Both types of PM require laboratory calibration to evaluate the membrane stiffness and compliance effects. Membrane stiffness is important in soft soils whereas system compliance effects data in stiff soils. Considerable errors in measured/interpreted values may arise due to these factors. Probe design also has an important effect on measured data, i.e. hysteresis and sensitivity of strain arm performance to design details (Dalton and Hawkins, 1982; Howie et al., 1990), effect of inflation method (stress or strain controlled) on membrane calibration curves (Anderson et al., 1987), geometrical tolerances along the length of the probe and rate affects (Mair and Wood, 1987). Combine these effects with the problems associated with probe installation (Jamiolkowski et al., 1985) and it becomes evident that interpretation of SBPM data is at best difficult and often subjective.

For the PAFSOR probe, only total stresses are measured. Consequently, after "ideal" installation and a relaxation period sufficient to allow dissipation of any excess pore pressures, the lateral stress exerted by the soil on the membrane should equate to the in situ horizontal stress, $\sigma_{ho}$, when the membrane is at its zero position. For the British SBPM, various methods exist for evaluating the in situ horizontal stress, some of which were originally developed for use with the Menard PM. However, since they are usually applied to SBPM data they are briefly considered here.
Critical reviews of the methods available for evaluating \( \sigma_h \) from self-boring pressuremeter data have been presented by Denby and Hughes (1982), Lacasse and Lunne (1983) and Mair and Wood (1987). These methods and subsequent developments are reviewed below, under the following groups:

- estimation of lateral stress by lift-off methods
- estimation of lateral stress by graphical methods
- computer aided procedures for estimating lateral stress
- estimation by empirical methods

A.2.1 Estimation of Lateral Stress by Lift-Off Methods

Of the methods available, the lift-off approach is only applicable to SBPM data since the assumption of near "ideal" installation is inherent in this approach. Once installed in the ground, the cavity pressure behind the membrane is gradually increased; the moment of first movement of the strain arms is recorded and denoted the lift-off pressure as this corresponds to the point where cavity pressure is equal to \( \sigma_h \). Further increase of cavity pressure expands the membrane radially until a limit pressure is reached.

Depending on equipment characteristics and soil conditions the three strain arms may not lift-off at the same pressure (Fig. A.2a). Whereas previously the data from the three strain arms was averaged to yield a single value of lateral stress, present practice is to evaluate separately the stresses from all three strain arms - this change in procedure is indicative of the uncertainty involved in \( \sigma_h \) evaluation, especially in firm to stiff clays and medium dense to dense sands. Typical pressure-deformation (p-\( \varepsilon \)) responses are shown in Fig. A.2. The initial linear portion of the curve is due to compliance (Fig. A.2b); thereafter the curve becomes nonlinear indicating the point of lift-off. In more recent designs, the compliance
Fig. A.2 Idealized p-ε responses for SBPMT.
effect is removed and lift-off occurs at first strain arm movement (Fig. A.2c).

The method is somewhat subjective and becomes more so as the quality of the insertion decreases, i.e. as disturbance effects become more pronounced. Furthermore, at lift-off the p-c gradient is high and deciding on a single value of lift-off for a particular arm can be difficult; this dilemma is accentuated in stiff soils - enlarged scale plots of the lift-off area are useful in this respect. Poor arm response, electrical noise on signal, and inadequate data capture can further complicate the problem for poorly designed systems.

Denby (1978) has suggested that initial movement of the membrane does not occur when the cavity pressure is equal to \( \sigma_h \). Based on similar experience, Lacasse et al. (1981) suggested the use of a modified lift-off method (Fig. A.3a), defined as the pressure at the start of a linear variation of radial strain with time. The method empirically accounts for compliance (rather than by calibration) and is applied to individual arms since strain arm averaging can mask the break in the strain-time curve.

The pressure at which development of excess pore pressures occurs during expansion of a cylindrical cavity around the pressuremeter has been suggested by Wroth and Hughes (1974) as an indicator of the lateral stress condition (Fig. A.3b). This implies that \( \sigma_h \) is a yield stress and is thus only applicable to normally consolidated clays. In OC clays, \( \sigma_h \) is not a yield pressure and elastic reloading will occur until the horizontal yield pressure, \( \sigma_{h y} \), is reached, at which point excess pore pressures will be generated. However, combining the excess pore pressure idea with the Marsland and Randolph (1977) technique it may be possible to evaluate \( \sigma_h \) in OC soils by an iterative procedure. Jefferies et al. (1985) use the pore
Fig. A.3 Methods for evaluating horizontal stress from SBPMT (adapted from Lacasse and Lunne, 1982).
pressure method in OC soils correcting the yield stress by means of the shear modulus obtained from an unload/reload cycle. The pore pressure method can only be applied for SBPM tests with pore pressure measurement; good saturation of the piezometer system is important. The disadvantage of the method is that it relies on an idealized theoretical relationship for elastic reloading and very localized pore pressure measurements. This aside, consistent estimates of $\sigma_h$ have been reported for soft clays.

The log strain method was proposed by Law and Eden (1982) based on SBPM tests using various cutting shoe sizes and varying degrees of strain softening (disturbance) imparted to the soil. It is essentially an inspection method relating $\sigma_{ho}$ to the break in the expansion pressure - log strain curve (Fig. A.3c).

A.2.2 Estimation of Lateral Stress by Graphical Methods

The initial yield method developed by Marsland and Randolph (1977) uses a graphical iteration technique based on an elastoplastic soil idealization. Firstly, the undrained shear strength is derived using an estimated $p^o(\sigma_h)$; subsequent iterations of $p^o$ are made until

$$p^o(\sigma_h) + S_u = p_{hy} \quad (A.1)$$

where $p_{hy}$ is the initial yield at the end of the linear portion of the expansion curve (Fig. A.3d). The method was originally developed for use with the Menard pressuremeter in stiff clays but has been applied with some success to SBPM results.

Denby (1978) and Denby and Clough (1980) suggested two definitions for evaluating $\sigma_h$ by use of a curve fitting technique based on a hyperbolic
plastic soil model. If no initial early movement of the strain arms was present, the apparent pressure, $p_0^A$, was obtained by extrapolating the modelled pressuremeter curve to zero strain. If the field curve indicated disturbance or compliance effects, then the $p_0$ value was taken as that at the value of corrected initial strain (Fig. A.3e). Arnold (1981) modified the Denby (1978) method by performing a double zero shift to account for offsets on either or both the pressure and strain axes. A hyperbolic modelling of the expansion curve is then performed using 3 points from the smoothed data curve. The determination of $p_0$ is then by inspection and is very subjective (Fig. A.3f), depending particularly on the data in the initial region of the pressure-strain curve. Prevost and Hoeg (1975) also present a graphical technique for determining soil parameters for cases of both strain softening and strain hardening soil conditions. Ideally strain controlled PM test data should be used. As the models used in the graphical procedures become more complex, a larger number of parameters have to be optimized and the fitting procedure becomes more subjective. As a result computer aided processing becomes necessary.

A.2.3 Computer Aided Procedures for Determining Lateral Stress

All the above methods for evaluation in lateral stress rely on inspection of the data at radial strains of 1% or less to determine $\sigma_h$. General solutions for stresses around piles, and cavity expansion models indicate that the complete stress-strain curve from the SBPMT should be sensitive to the initial value of horizontal stress (see Appendix B).

Jefferies (1988) and Hughes (1989) present methods whereby the complete stress-strain curve from SBPM data is utilized. The method proposed by Jefferies (1988) uses an undrained elastic plastic soil model applied to a
set of equations which consider the various stages of loading and unloading during expansion/deflation of the SBPM membrane. The problem considers three variables $G$, $S_u$ and $\sigma_h$, all of which exert different effects on the SBPM model curve (Fig. A.4). The technique proposed by Hughes (1989) for drained tests in sand uses a four parameter model $(G, \phi_{cv}, \sigma'_h, \nu)$ to evaluate a power function fitted to the field data. The main advantage of these approaches is

![Diagram]

Fig. A.4 Influence of variables for undrained SBPMT in clay (after Jefferies et al., 1988).
that the sensitivity of the calculated pressuremeter curve to parameter variations can easily be checked and be used to refine parameter selection. As both authors suggest, the proposed methods allow disturbance during insertion to be corrected for; the latter stage of the expansion test (strains greater than 2%) and the unload portion determine the best fit parameters. The Jefferies et al. (1988) model was developed for Beaufort Sea clays whereas Hughes (1989) considers granular materials. The authors report successful application of the methods to the evaluation of lateral stress for both good and poor SBPM data.

A.2.4 Empirical Methods for Evaluating Lateral Stress

The use of empirical corrections to obtain the in situ horizontal stress from SBPM data in clays is based on a limit pressure-undrained strength relationship similar to that used in bearing capacity type analyses:

\[ \sigma_h = P_L - N_p S_u \]  \hspace{1cm} (A.2)

where \( N_p \) is an empirical constant dependent on soil type:

\[ N_p = 1 + \ln(1/E_u/3S_u) \]  \hspace{1cm} (A.3)

This formulation means that \( \sigma_h \) is very sensitive to errors in \( S_u \) and \( N_p \) and is generally unreliable.

A.2.5 Conclusions

As evidenced by the above many methods exist for evaluating \( \sigma_h \) from SBPM test data. It is also apparent that most methods have been derived for
clays. For sands, where the visual lift-off method may not be possible, it appears that curve fitting followed by the application of an appropriate soil model may provide the only reliable method for evaluating $\sigma_h$. Based on ten years experience with the self-boring pressuremeter, Jamiolkowski et al. (1985) suggest that the only reliable approach to the evaluation of $\sigma_h$ from SBPM is the lift-off method.

It must be borne in mind however, that even for conditions of ideal installation, the lift-off pressure given by the SBPM does not necessarily correspond to the in situ lateral stress, as evidenced by the calibration chamber tests performed by Bellotti et al. (1987). In order to obtain reliable and repeatable measurements of $\sigma_h$ from the early part of the pressure-expansion curve, Jamiolkowski et al. (1985) suggest the need to improve the SBPM technique in the following areas:

1. Reduction or elimination of strain arm and membrane compliance either by design modifications or careful calibration. These effects are usually nonlinear and hysteretic.

2. The adoption of a probe with an internal rigid body in order to attain perfect cylindrical insertion.

3. Sensitivity of the soil to disturbance due to the cutter position relative to the cutting shoe. Theoretical studies have provided guidance for tests in clay but little experience is available in sands.


5. Drilling technique - rig vibrations and eccentric movements may be transmitted to the probe via the rods and cause disturbance, especially in sands.

6. The quality of the self-boring process is usually judged by the excess pore pressures induced during probe installation. Optimization of the
self-boring parameters (penetration rate, mud pressure, flow rate, cutter rotation (or jetting pressure) may improve the data quality by reduced disturbance.

7. Evaluation of the effects of relaxation time on the pressure-expansion curve.

A.3 PUSH-IN TOTAL STRESS CELL (TSC)

Measurements of lateral stresses against structures or within engineered fills have been obtained since the early 1950's and represent a fairly well established technology in soil mechanics. Until recently, the total stress cells were embedded either in the fill or at the soil-structure interface. Ideally, a stress cell should exhibit the same load-deformation characteristics as the soil it replaces; this condition is virtually impossible to satisfy. Installation of the cell within materials of differing stiffness was recognized as having a major effect on the measurements due to bedding errors, arching, stress concentrations etc. and consequently it was more common to employ them for measurements of the contact stress against the face of a structural element. Research into the behaviour of pressure cells can be summarized thus:

- Based on laboratory calibrations and numerical modelling, the results of measurements with embedded pressure cells have been shown to be insensitive to the soil/cell stiffness ratio (Krizek et al., 1974).
- If the soil on one side of the cell is softer than the overall soil mass, stress reductions occur.
- The measured normal stress on the cell depends only on the normal stress acting on the face on the cell and is independent of other stress components (Krizek et al., 1974).
• The response of the cell is temperature dependent and the dependence is a function of the cell prestress (Felio and Bauer, 1986).

• The stress sensitive area should be about half the total cell area to avoid registration of stress concentrations at the cell periphery (Audibert and Tavenas, 1975).

• Pressure cells in different environments (i.e. air, water or soil) gave the same calibration curves provided temperature changes were accounted for, i.e. hydrostatic calibration is adequate for interpreting field results (Krizek et al., 1974; Felio and Bauer, 1986).

• A cell factor, $F$, of less than 5 is recommended to reduce nonuniformity of stress (Tory and Sparrow, 1961) where:

$$F = \frac{E_s d^3}{E_c t^3}$$  \hspace{1cm} (A.4)

where

- $E_s$ = soil stiffness
- $E_c$ = cell stiffness
- $d$ = diameter of pressure diaphragm
- $t$ = thickness of pressure diaphragm

The concept of the push-in spade-like total pressure cell to measure in situ horizontal stress was first utilized by Massarsch (1975) in a soft clay. The Glotzl cell used was 4mm thick and was pushed into the ground protected within a steel casing. The frame was withdrawn about 0.3m above the intended depth and the cell alone advanced and then left in the ground until equilibrium was reached. The maximum membrane deflection of the Glotzl cell
is about 5μm (negligible in soft soils). Use of this displacement method gave consistent $K_q$ values for this normally consolidated deposit. Satisfactory results in soft clay ($S_u < 30$ kPa) have also been reported by Massarsch et al. (1975), Tavenas et al. (1975), Massarsch and Broms (1976) and Massarsch (1979). Multiple measurements at one depth were within ±1 kPa (Tavenas et al., 1975).

During installation of the total stress cells (TSC), the soil is displaced and excess pore pressures are generated which then decay with time. Once these excess pore pressures have dissipated the lateral stress in the ground should ideally still be higher than the pre-installation value. However, if the viscoelastic characteristics of the soil permit, the stress induced may dissipate due to creep so that no stress over and above the original $K_o$ stress remains. For this reason, it is generally accepted that no correction to the final measured lateral stress is required in soft clays, i.e.

$$\sigma_{TSC} = \sigma_{ho} + u_o + \Delta \sigma'_h$$

$$\Delta \sigma'_h = f(t) = 0$$

where $t > 1-2$ months. In fact, $\Delta \sigma'_h$ will seldom be zero but may be small enough so as not to cause noticeable deviations from the expected $K_o$ value.

However, from research performed in stiff overconsolidated soils, Tedd and Charles (1981, 1983) concluded that the TSC overreads by an amount approximately equal to one half the undrained shear strength (determined from unconsolidated undrained triaxial compression tests). The reference lateral stress to evaluate the amount of overread was taken as that obtained from
SBPM. The magnitude of the overread can be related to the modulus of the soil but Tedd and Charles (1983) argue that due to the impracticality of deciding on a relevant modulus value it is more realistic to relate the overread empirically to $S_u$. Data presented by Powell et al. (1983) for a stiff glacial till confirm the magnitude of the correction suggested by Tedd and Charles (1983). Figure A.5 presents a review of TSC data performed by

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>Site</th>
<th>True Reading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft NC</td>
<td>Ska-Edeby</td>
<td>Applied Pressure</td>
</tr>
<tr>
<td>Puddle clay</td>
<td>Cwmwernderf</td>
<td>Soil suction, OD</td>
</tr>
<tr>
<td>Glacial till</td>
<td>Cowden</td>
<td>Overburden pressure SBP</td>
</tr>
<tr>
<td>Glacial till</td>
<td>Cowden</td>
<td>Overburden pressure SBP</td>
</tr>
<tr>
<td>London Clay</td>
<td>Bell Common</td>
<td>Overburden pressure SBP</td>
</tr>
<tr>
<td>London Clay</td>
<td>Balham</td>
<td>Overburden pressure SBP, OD</td>
</tr>
<tr>
<td>London Clay</td>
<td>Brent</td>
<td>Overburden pressure SBP, DMT</td>
</tr>
<tr>
<td>Oxford Clay</td>
<td>Oxford</td>
<td>Overburden pressure SBP, DMT</td>
</tr>
<tr>
<td>London Clay</td>
<td>Reading</td>
<td>SBP, DMT</td>
</tr>
<tr>
<td>London Clay, Pl=40</td>
<td>Reading</td>
<td>SBP, OD</td>
</tr>
<tr>
<td>Soft alluvium</td>
<td>Grangemouth</td>
<td>Soil suction, SBP</td>
</tr>
<tr>
<td>Gault Clay</td>
<td>Madingley</td>
<td>SBP, OD</td>
</tr>
</tbody>
</table>

Fig. A.5 Correlation between overread and $S_u$ for spade cells in clay (adapted from Tedd et al., 1989).
Tedd et al. (1989) to evaluate the trend between overread and \( S_u \). The scatter in the data is partly attributable to errors in the actual reference stress values themselves (estimated from the test method listed in column 3 of the figure legend), as well as variations in both \( \sigma_{TSC} \) and \( S_u \).

The reliability of stress measurements using push-in spade-like total stress cells depends on several factors, namely:

- The TSC has to be installed vertically otherwise some other stress component will be measured and erroneous results will be obtained.
- The baseline value due to the cell prestress is sensitive to bending of the blade, resulting in a fictitious overread.
- The TSC is temperature sensitive and field measurements must be related to the ground temperature baseline. None of the presented case histories consider this problem. Temperature coefficients as high as 2 kPa/°C may be obtained and have important consequences especially for near surface measurements.
- The total stress cell must remain in the ground long enough for complete dissipation of excess pore pressures and termination of stress redistribution through creep (in soft clays where no correction is applied).
- Where the TSC is installed in soils with \( S_u > 30-50 \) kPa, the final measured stress requires correction.

Chan (1985) presents data from Genesee clay (\( S_u = 60-80 \) kPa) where the final lateral stresses are not corrected for overread. Little data exist to define the limiting shear strength below which no correction is required. The \( K_0 \)-OCR relationships derived for this site would suggest that some correction is necessary. Also, it would appear that the measured stresses had not
totally stabilized during the period of monitoring (about 7 days), thus any possible stress relaxation would not have occurred, further suggesting the need to correct the measured lateral stresses.

A review of all published data where stress history (OCR) and $K_o$ (from TSC) are available, leads to the following correlation (Fig. A.6):

$$K_{TSC} = 0.587 \text{ (OCR)}^{0.432}$$

(A.6)

Fig. A.6 Relationship between $K_{TSC}$ and OCR from published data.
It is worth noting that the data in Fig. A.6 have been obtained from two different commercially available spade cells. The earlier work in soft clays was performed using the Glotzl cell. Later research in stiff clays was by means of the Soil Instruments Ltd. cell. This cell has no protective cover and is usually installed at the base of a borehole by pushing over a distance of between 0.5m and 1.0m into undisturbed soil. This type of cell has been used by the author for this research and is described in detail in Chapter 3.

The method for correcting for TSC overread in stiff clays suggested by Tedd and Charles (1981) was developed for the Solinst type TSC but has been applied also to the similarly dimensioned Glotzl cells. Figure A.6 would suggest that the corrected lateral stress from TSC data represents fairly well the actual in situ conditions, as referenced by the self-boring pressuremeter. It is also evident from published data, that the scatter in measured horizontal stress from TSC is much less than that associated with the SBPM, especially in stiff heavily overconsolidated clays.

A.4 DILATOMETER TEST (DMT)

The flat dilatometer test (DMT) was introduced by Marchetti (1975) with the idea of evaluating in situ parameters from full displacement measurements. The dilatometer consists of a 14mm wide flat blade with an expandable 60mm diameter circular membrane on one side. A detailed description of the equipment and test procedure is given by Marchetti (1980).

The test consists of gradually increasing the gas pressure behind the membrane and noting the following values:

- The DMT lift-off pressure, which corresponds to a membrane displacement of 0.05mm, denoted the A reading.
• The DMT limit pressure, when a membrane expansion of 1.1mm is attained, designated the B reading.

Both of these displacements are indicated by the switching on/off of a buzzer connected to a feeler arm behind the membrane. In recent years, improvements to the procedure have been made requiring the following additional values to be recorded during the DMT sounding:

• The pushing force (usually measured at the surface) required to penetrate the blade into the ground, T (Schmertmann, 1986). Similar to the cone penetration test, the standard pushing rate is about 2 cm/s.

• The pressure when deflation of the membrane is complete and the membrane is again in contact with the seating plate (Robertson et al., 1988; Lutenegger and Kabir, 1988). This is normally designated the closure pressure, or C reading.

The sequence of readings T, A, B and C is usually performed every 200mm. In sands, the corrected C reading has been shown to approximate to the in situ equilibrium pore pressure. In clay it reflects the excess pore pressure induced due to penetration (Robertson et al., 1988). Membrane stiffness corrections ΔA, ΔB are applied to the A, B and C readings to determine the DMT pressures according to:

\[ P_0 = 1.05 (A - Z_m + \Delta A) - 0.05 (B - Z_m - \Delta B) \quad (A.7) \]

\[ P_1 = B - Z_m - \Delta B \quad (A.8) \]

\[ P_2 = C - \Delta A \quad (A.9) \]
where:

\[ \Delta A = \text{external pressure necessary to keep membrane in contact with seating (in free air)} \]

\[ \Delta B = \text{internal pressure to expand membrane to } 1.1 \text{mm displacement in free air} \]

\[ Z_m = \text{gauge reading when pressure is ventilated} \]

The \( p_0 \) value is determined by a linear extrapolation from \( p_0 \) (0.05 mm displacement) back to zero displacement using the average gradient between \( p_0 \) and \( p_1 \) (Schmertmann and Crapps, 1988). The use of the linear extrapolation shown in Fig. A.7 affects the \( p_0 \) estimate where the expansion curve is in reality nonlinear. Even though the curve between \( p_0 \) and \( p_1 \) may closely approximate a linear response, it is probably not the case for the initial pressure increase from 0 to 0.05mm displacement. However, since the interpretation of soil parameters from DMT data is based on empirical correlations, the effect may not be important. The deviation of the actual curve from the assumed linear form will be a function of soil characteristics and stress history.

Other factors affecting the determination of \( p_0 \) are the delay between halting penetration and expanding the membrane and the rate of membrane expansion. Delay in starting the test will allow the dissipation of excess pore pressure as will expansion at a slower rate. These effects are of less importance in sands where the test is essentially drained and in low permeability clays where pore pressure dissipation is slow. Since the interpretation of test data is based on empirical correlations, the test procedure should closely follow the accepted standard. The whole test should be completed within a two minute period with 15 to 30 seconds being the time
Fig. A.7 Effect of linear extrapolation on derived \( p_0 \) value from DMT.

interval for taking the A and B readings. The dilatometer pressures \( p_0 \) and \( p_1 \) are used to define the following index parameters (Marchetti, 1980):

\[
\text{Material Index, } I_D = f(A,B,u_0) = \frac{p_1 - p_0}{p_0 - u_0} \quad (A.10)
\]
Horizontal Stress Index, $K_D = f(A, u_0', \sigma_v') = \frac{P_0 - u_0}{\sigma_v'}$ \hspace{1cm} (A.11)

Dilatometer Modulus, $E_D = f(A,B) = 34.7(p_1 - p_0)$ \hspace{1cm} (A.12)

The advantage of the DMT is that the test is simple to perform, very repeatable and can be used in soils ranging from soft clays to dense sands. It also provides information on a multitude of soil parameters, albeit by empirical correlations.

Recent developments to the technique include:

- The development of a piezoblade (Davidson and Boghrat, 1983) for measuring in situ pore pressures. The piezoblade has exactly the same dimensions as the DMT but instead of an expandable membrane, it contains a flush-mounted porous stone and pressure transducer. The piezoblade confirmed that the DMT $p_2$ pressures correlate well with penetration pore pressures from CPTU and that the measured value depends on soil behaviour type (Lutenegger and Kabir, 1988).

- The development of research DMT equipment at the University of British Columbia (UBC) whereby the pore pressure transducer is mounted behind the expandable membrane (Tsang, 1987). In addition, a feeler arm located behind the membrane allows the complete pressure-displacement relationship to be monitored. An axial load cell behind the blade allows continuous monitoring of the pushing force and an inclinometer provides a check on blade alignment.

- At the Norwegian Geotechnical Institute an offshore DMT was developed where the pore pressures are measured on the rear side of the blade directly behind the membrane. The dimensions of the blade are slightly less than those of the standard (Marchetti) blade although it
is slightly thicker in order to accommodate the pore pressure transducer (Lunne et al., 1987).

A comparison of the results obtained using the UBC and NGI DMT blades was performed at three UBC research sites (By et al., 1987). Good agreement between the NGI and standard DMT results were obtained in both sand and soft clay. In sand the UBC DMT gave lower $p_0$ and $p_1$ values than the NGI/standard blades. In soft clay the UBC and NGI values are almost identical. However, the interpreted DMT indices $I_D$, $K_D$ and $E_D$ are somewhat different which may be a result of the effect of variations in the measured pore pressures. It would appear that DMT results are equipment sensitive.

The original empirical correlations suggested by Marchetti (1980) for determining stress history parameters ($K_o$, OCR, $S_u/\sigma'_v$) in sands and clays are listed below. The $K_D$-$K_o$ correlation was based on the $K_o$-OCR relationship proposed by Brooker and Ireland (1965).

$$K_o = \left( \frac{K_D}{1.5} \right)^{0.47} - 0.6 \quad \text{(A.13)}$$

For OCR = 1, $K_D$ = 2, and $K_o$ = 0.54.

$$\text{OCR} = (0.5 \frac{K}{K_D})^{1.56} \quad \text{for } 0.2 < I_D < 2 \quad \text{(A.14)}$$

$$\text{OCR} = \frac{K_D}{1.5}^{1.91} \quad \text{for } I_D < 2 \text{ (granular)} \quad \text{(A.15)}$$

$$\frac{S_u}{\sigma'_v}_{oc} = 0.22(0.5 \frac{K_D}{K_D})^{1.25} \quad \text{for } I_D < 1.2 \quad \text{(A.16)}$$
Table A.1 indicates the accuracy of the above correlations based on experience up until 1986 (Schmertmann, 1986). Only the $K_D - K_o$ correlations will be considered here. The determination of overconsolidation ratio as a precursor to $K_o$ has been considered earlier in the thesis.

Table A.1 Results of Experience with DMT for Predicting Stress History Parameters (modified from Schmertmann, 1986)

<table>
<thead>
<tr>
<th>Stress History Parameter</th>
<th>Number of Tests</th>
<th>(DMT - Measured/Reference Value) Comparisons (%)</th>
<th>Range of Average DMT Values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>Stand. Dev.</td>
</tr>
<tr>
<td>Sand &amp; Silt:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_o$</td>
<td>6</td>
<td>+10</td>
<td>23</td>
</tr>
<tr>
<td>OCR</td>
<td>5</td>
<td>+17</td>
<td>21</td>
</tr>
<tr>
<td>$u_0$</td>
<td>12</td>
<td>+1</td>
<td>12</td>
</tr>
<tr>
<td>Clay &amp; Organics:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$K_o$</td>
<td>10</td>
<td>+7</td>
<td>28</td>
</tr>
<tr>
<td>OCR</td>
<td>17</td>
<td>+4</td>
<td>30</td>
</tr>
<tr>
<td>$S_u$</td>
<td>38</td>
<td>-0.2</td>
<td>22</td>
</tr>
</tbody>
</table>

Based on field data using DMT averages from individual distinct layers or groups of tests.

NB: The application of the Marchetti correlations are restricted to soils having no cementation and that have undergone overconsolidation due to simple unloading.

In the following sections the use of the DMT index parameters to obtain stress history characteristics in both sand and clay will be examined. Due to recent interest in potential of the DMT, a significant volume of research has been performed. This is reviewed and commented upon.
A.4.1 DMT in Sand

The original Marchetti relationship (Eq. A.13) was found to overpredict $K_o$ in sands and was shown to depend on both $\sigma_h'$ and $D_r$. Hence it was necessary to separate the two effects in order to provide reliable estimates of $K_o$. This was first attempted by Schmertmann (1983) who proposed a procedure for estimating $K_o$ from $K_D$ based on limited calibration chamber testing. The relationship requires the measurement of the DMT thrust (to penetrate the blade) so that the drained angle of shearing resistance, $\phi'$, can be deduced. The correlation is given by:

$$K_o = \frac{40 + 23K_D - 86K_D(1-\sin\phi'_t) + 152(1-\sin\phi'_t) - 717(1-\sin\phi'_t)^2}{192 - 717(1-\sin\phi'_t)}$$

where

$\phi'_t$ = triaxial friction angle of the soil determined from CPT or DMT data (Schmertmann, 1982).

Jamiolkowski et al. (1985) suggest an iterative procedure to optimize the calculated $K_o$ value, and compared the obtained $K_o$ values with CC data for Ticino sand and SBPM field data from Po River sand. They conclude that the Schmertmann procedure yields reliable results. The correlation is based on a limited data base of calibration chamber tests which have not been corrected for chamber size effects. Belloti et al. (1979) show that $K_D$ from CC tests was up to 3 times the value obtained in typical field tests. Furthermore, the $p_o$ value in the CC is dependent on the operating boundary conditions, therefore any CC derived correlations should preferably be calibrated against field data. A comparison of the Marchetti (1980) and Schmertmann (1983) relationships is shown in Fig. A.8.
Marchetti (1985) modified the Schmertmann technique and produced a graphical dependence between \( K_o \), \( K_D \) and \( q_c/\sigma'_v \), where:

\[
K_D = f(K_o, \phi) \quad \text{(equation A.17)}
\]

and

\[
q_c/\sigma'_v = f(K_o, \phi)
\]

based on the bearing capacity theory of Durgunoglu and Mitchell (1975). Since the theory is complex mathematically, requiring an iterative computer procedure to optimize \( K_o \) and \( \phi \), Marchetti produced the graphical form shown in Fig. A.9, which was later modified by Robertson (1986) using the
Campanella and Robertson (1983) $q_c - \phi$ empirical relationship. Comparison with field data from SBPM in the Po River sand is also shown on Fig. A.9. The modified graphical method still noticeably overestimates $K_0$, a fact compounded by the very steep gradients of the $q_c / \sigma'_v$ contours. Again chamber size effects may be the causes of the errors involved.

![Graph](image)

Fig. A.9 Marchetti (1985) graphical form for $K_0 - K_{q_v}/\sigma'_v$ relationship modified using Robertson and Campanella (1983) $q_c - \phi'$ correlation (after Robertson, 1986).
Baldi et al. (1986) reviewed all the available CC DMT data to evaluate $K_o = f(K_D, q_c/\sigma_V')$ as suggested by Marchetti. Due to the boundary condition effect only data with the BC1 (constant boundary stress) were considered, resulting in the following relationship:

$$K_o = 0.376 + 0.095K_D - 0.00172(q_c/\sigma_V')$$  \hspace{1cm} (A.18)

Equation (A.18) was then field calibrated using Po River data by adjusting the $q_c/\sigma_V'$ multiplier. According to Baldi et al. (1986), Eq. (A.19) represents the best available (tentative) procedure for evaluating $K_o$ from DMT data in natural uncemented quartz sands.

$$K_o = 0.376 + 0.095K_D - 0.0046(q_c/\sigma_V')$$  \hspace{1cm} (A.19)

The requirement in Eq. (A.19) for both DMT and CPT data can be avoided by replacing $q_c$ with an equivalent $q_D$ (Sully and Campanella, 1989). Grain size and mineralogical composition (both affecting compressibility) may affect $K_o$ estimates in sands having different characteristics than the sand used for the CC and field calibration. Similarly, the effect of CC boundary conditions has only been considered indirectly. Further validation is obviously required.

Based on the work of Jefferies et al. (1987), described below in Section A.6, Jamiolkowski et al. (1988) evaluated the relationship between amplification factor ($K_D/K_o$) and state parameter ($\psi$) and found good correlation for CC data using Hokksund (HS) and Ticino (TS) sands:

$$K_D/K_o = a e^{(m\psi)}$$  \hspace{1cm} (A.20)
where
\[ a, m = \text{empirical coefficients} \]

However, the use of the state parameter approach for interpreting field data is complicated by several factors (Sladen, 1989) not least being the necessary a priori knowledge of \( K_0 \) (Campanella, Sully, Greig and Jolly, 1990).

Using the relationship between \( q_c \) and \( \psi \) for Ticino sand, \( \psi \) in Eq. (A.20) can be eliminated to give (Jamiolkowski and Robertson, 1988):

\[
K_D/K_0 = 0.0578 \left[ \frac{q_c - \sigma_m}{\sigma_m'} \right]^{0.92} \quad (A.21)
\]

The two constants in Eq. (A.21) are particular to Ticino sand and will vary for other sands of differing mineralogical composition, angularity, etc. Figure A.10 compares the field-adjusted correlation of Baldi et al. (1986) (Eq. A.19) with that given above. It is apparent that the CC \( K_0 - \psi \) relationship noticeably overestimates \( K_0 \). It would also appear to be extremely sensitive to small errors in \( K_0 \). Equation (A.21) results from the combination of two relationships with large inherent scatter. It is also based on very limited CC data. The error in applying the \( K_D - \psi \) relationship is also compounded at low values of \( K_D (<5) \) and \( q_c/\sigma_v' (<150) \), values commonly encountered in the field. The difference in trend for the two relationships shown on Fig. A.10 would suggest that CC is unable to correctly simulate the in situ stress response of granular soils, probably as a result of fabric and environmental differences. The effect of boundary conditions on CC results, especially in relation to stress distribution around full displacement probes requires further investigation if field-relevant correlations are to be
Calibration studies do however provide interesting insights into response mechanisms. A review of the Baldi et al. (1986), Houlsby (1988), Masood (1990), and Lawter & Borden (1990) DMT data from calibration chamber tests led to the following conclusions:

- The measured \( p_0 \) and \( p_1 \) values are dependent on both \( D_r \) and applied chamber horizontal stress, (Fig. A.11a).

- Conversely, \( p_0 \) and \( p_1 \) are totally independent of \( \sigma_v \) in the chamber (Fig. A.11b).
**Fig. A.11** Measured $p_0$ and $p_1$ values from CC tests to show (a) dependence on horizontal effective stress, and (b) insensitivity to vertical effective stress.
• $p_0$ and $p_1$ values for a particular soil are uniquely related (Fig. A.12).

Houlsby (1988) and Campanella & Robertson (1989) show that $p_0$ may not be an independent measurement but related to the DMT penetration thrust, $T_{DMT}$. Campanella and Robertson (1989) further suggest that $K_D$ and $q_c/\sigma_v'$ are related. Taking this one step further:

$$\frac{q_c}{\sigma_v'} = \frac{q_D}{\sigma_v'} = 33K_D = 33 \frac{p_0-u_0}{\sigma_v'}$$

(A.22)
Hence,

\[ P_0 - u_0 = f(q_D) \]

\[ q_D = \frac{T_{\text{DMT}}}{20} \text{ (kg/cm}^2\text{)} \] \hspace{1cm} (A.23)

where

\[ q_D = \text{the equivalent DMT bearing resistance (Sully and Campanella, 1989)} \]

and since \( p_0 \) and \( p_1 \) are interdependent (Fig. A.12), it follows that:

\[ p_1' = f_1(p_0') = f_2(q_D) = f_3(\sigma_h') \] \hspace{1cm} (A.24)

It appears that \( q_D \) is primarily dependent on \( \sigma_h' \). Due to the minor degree of stress relief that occurs as a soil element passes the singularity at the DMT wedge, \( p_0 \) and \( p_1 \) may, as a consequence, have only a secondary dependent on \( \sigma_h' \). Other soil characteristics (E.G.\( D_r \)) may exert important influences.

A relationship examining the effect of \( \sigma_h' \) (pre-penetration) and soil stiffness (\( E_D \) from DMT) on the magnitude of the stress increment due to DMT insertion was evaluated by the writer using the Baldi et al. (1986) data. The correlation assumes a pseudo-elastic soil response during initial lift-off of the DMT membrane, where:

\[ \frac{p_0 - \sigma_h'}{E_D} = \beta \] \hspace{1cm} (A.25)
For several sands the average value of $\beta$ was 0.0127 with a standard deviation of 0.0029 (±20%). Again, as with the correlations presented earlier, the relationship is ill-conditioned with respect to the accuracy of DMT measurements ($p_0, E_D$). However, in sands where drained penetration causes large stress increments this type of relationship is unavoidable at present. Theoretical developments may, in the near future, vindicate this approach. CC testing should be programmed to provide further qualitative information to allow the development of logical approach to the interpretation of in situ stress state from DMT data.

A.4.2 DMT in Clay

Similar results to those in sand have been obtained in clay using the original Marchetti correlation to predict $K_o$ from $K_D$, i.e. the under- or over-prediction of in situ horizontal stresses. In the soft and stiff clays of British Columbia the DMT correlations overpredicted both OCR and $K_o$ (Tsang, 1987; Sully and Campanella, 1990). In stiff overconsolidated clays, Powell and Uglow (1988) report overprediction of OCR and $K_o$ in high plasticity clays and underprediction in low plasticity clays. They also show that the $K_D-K_o$ correlation is different for young (<60,000 years) and old (>70 million years) clay deposits.

Lacasse and Lunne (1988) review data from Norwegian and British soft to stiff clays and show that the Marchetti correlation overpredicts $K_o$ for $1.5 < K_D < 4$, and generally underpredicts OCR at all $K_D$ (the data for soils of high PI give better agreement with the Marchetti formula than low PI data). Lacasse and Lunne (1988) and Powell and Uglow (1988) suggest essentially the same revised formula for clays:
\[ K_0 = a (K_D)^m \]  \hspace{1cm} (A.26)

where

\[ m = 0.44 \text{ (high PI) to } 0.64 \text{ (low PI)} \]
\[ a = 0.34 \]

Powell and Uglow (1988) suggest \( a = 0.34, m = 0.55 \), for young UK clays (Fig. A.13) and \( a = 0.68, m = 0.54 \) for old clays (Lunne et al., 1990).

Fig. A.13 \( K_0-K_D \) correlation for DMT in clay (after Powell and Uglow, 1988).
A review of the original data published by Marchetti and subsequent research by various authors (Lunne et al., 1990) leads to the following:

- The Marchetti correlation was produced for clays with a PI ranging from 30% to 60%.
- The value of \( \frac{S_u}{\sigma'_v} \) may be used to distinguish between old and young clays.

It appears that the stress amplification during DMT is governed by, inter alia, the plasticity and OCR of a cohesive soil. It may thus be possible to relate the error in prediction of \( K_o \) using the Powell and Uglov (1988) relationship in the form:

\[
K_o = 0.34K_D^{0.55} + f(\text{PI}, \frac{S_u}{\sigma'_v})
\]  

(A.27)

Based on the data in Table A.2, which has been taken from several publications, it would appear reasonable to use the following empirical \( K_D-K_o \) correlation for DMT in all clays:

\[
K_o = 0.34(K_D)^{0.55} + \left[ (15-\text{PI}) \left( \frac{0.55S_u}{\sigma'_v} \right)_{\text{DMT}} \right]
\]  

(A.28)

where \( \left( \frac{S_u}{\sigma'_v} \right)_{\text{DMT}} \) is the undrained strength ratio obtained from DMT correlations.

This relationship has the advantage of considering the effects of both young and old clays and the soil plasticity which have been shown to be factors affecting the \( K_o-K_D \) correlation.
Table A.2  Review of DMT Data in Clays for Evaluation of Revised $K_D$-$K_0$ Correlation

<table>
<thead>
<tr>
<th>Site</th>
<th>$K_D$</th>
<th>$K_0$ (reference value)</th>
<th>$S_u/\sigma'_V$ from DMT</th>
<th>Plasticity Index PI</th>
<th>Error in $K_0$ Using Powell &amp; Úglow (1988) Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Porto Tolle</td>
<td>2</td>
<td>0.48-0.56</td>
<td>0.30</td>
<td>30-32</td>
<td>-</td>
</tr>
<tr>
<td>Montalto</td>
<td>3.5-4</td>
<td>0.9-1.06</td>
<td>0.40-0.52</td>
<td>29-39</td>
<td>-0.2 to 0.3</td>
</tr>
<tr>
<td>Cowden</td>
<td>5-20</td>
<td>0.8-1.2</td>
<td>0.8-3.0</td>
<td>17-20</td>
<td>-</td>
</tr>
<tr>
<td>Madingley</td>
<td>7.5-17</td>
<td>1.9-3.5</td>
<td>1.1-3.3</td>
<td>40-50</td>
<td>-1.0 to -2.0</td>
</tr>
<tr>
<td>Grangemouth</td>
<td>3.3-6.1</td>
<td>0.57-1.10</td>
<td>0.53-0.71</td>
<td>30-45</td>
<td>+0.3 to +0.7</td>
</tr>
<tr>
<td>Dartford</td>
<td>5.0-8.1</td>
<td>0.9-1.2</td>
<td>0.39-0.9</td>
<td>60-120</td>
<td>-</td>
</tr>
<tr>
<td>Gorpley</td>
<td>1.9-3.0</td>
<td>0.48-0.63</td>
<td>0.28-0.41</td>
<td>19-22</td>
<td>-</td>
</tr>
<tr>
<td>Brent Cross</td>
<td>4.8-15.3</td>
<td>2.0-4.0</td>
<td>0.9-1.6</td>
<td>40-50</td>
<td>-1.3 to -2.4</td>
</tr>
<tr>
<td>Canons Park</td>
<td>9.7-13</td>
<td>3.4-5.0</td>
<td>0.95-1.6</td>
<td>30-50</td>
<td>-3.5</td>
</tr>
<tr>
<td>Drammen</td>
<td>1.8-2.4</td>
<td>0.6-0.71</td>
<td>0.36-0.55</td>
<td>~10</td>
<td>-0.1</td>
</tr>
<tr>
<td>Onsøy</td>
<td>3.5-8</td>
<td>0.6-1.2</td>
<td>0.41-1.2</td>
<td>30</td>
<td>-0.1 to -0.4</td>
</tr>
<tr>
<td>Haga</td>
<td>4.0-6.1</td>
<td>0.68-1.6</td>
<td>0.52-0.85</td>
<td>12-16</td>
<td>-0.2</td>
</tr>
<tr>
<td>Lr 232 St.</td>
<td>3.0-5.0</td>
<td>0.55-1.5</td>
<td>0.30-0.50</td>
<td>16-34</td>
<td>+0.06 to 0.1</td>
</tr>
<tr>
<td>McDonald Farm</td>
<td>1.4</td>
<td>0.55</td>
<td>0.26</td>
<td>10-20</td>
<td>-0.1</td>
</tr>
<tr>
<td>Genesee</td>
<td>2.4-8.0</td>
<td>0.9-2.3</td>
<td>0.36-1.0</td>
<td>40-80</td>
<td>-0.4 to -1.2</td>
</tr>
</tbody>
</table>

Roque et al. (1988), Benoit et al. (1990) and Lutenegger (1990) used the DMT according to the ASTM procedure but at certain depths they performed dissipation tests with the blade recording $p_0$ and/or $p_1$ with time in order to perform an effective stress analysis of the horizontal stress variation. Roque et al. (1988) define a $K_N$ parameter:

$$K_N = \frac{(\sigma'_h + \Delta \sigma'_h) + a}{\sigma'_v + a}$$  \hspace{1cm} (A.27)

where $a = \text{attraction}$.

If $a = 0$ then $K_N = K_D$, and they suggest that:
where $\alpha_{\text{DMT}}$ is an empirical factor to account for overread. For the two soft to firm overconsolidated clay sites studied they found that $\Delta \sigma_h'$ in Eq. (A.27) was both positive (Glava), and negative (Halsen). This may have resulted from the extrapolation of excess pore pressure dissipation curves to evaluate the final fully dissipated value of $p_0$.

Benoit et al. (1990) reported fully dissipated $p_0$ values in soft clay that agree well with the upper bound estimate of $\sigma_h'$ from self-boring pressuremeter tests. Lutenegger (1990) shows that the same technique in a dense glacial till considerably overpredicts $\sigma_h'$, as would be expected.

This approach employs the DMT as a push-in total stress cell (see Section A.3). Since it is a full displacement technique, for the fully dissipated condition $\Delta \sigma_h'$ will not be zero (this has been shown earlier to be the case for the TSC in clay) unless sufficient time is allowed so that creep may dissipate the induced stress concentration due to insertion. In engineering time spans, this may only be possible in soft clay. Consequently, any recorded $p_0$ pressure will overpredict the in situ condition. The use of the DMT in this form is inconvenient as dissipation of excess pore pressure may take several hours or days. It would appear more logical to install TSCs by pre-pushing with a DMT as is the technique employed at UBC (Sully and Campanella, 1989) if both types of data are required.

Clarke and Wroth (1988) present the results of DMT and SBPM tests performed in stiff clay. They conclude that $p_0$ is greater than the yield stress and that $p_1$ is greater than the SBPM limit pressure. However, they do suggest that a relationship independent of stress level and soil type exists between $(p_1-p_0)$ and $(p_L-\sigma_h)$. Since $(p_1-p_0)$ includes elastic and plastic
property components the correspondence suggests that no unique relationship between \( p_0 \), \( p_1 \) and \( S_u \) or \( G \) exists.

Treatment of DMT data to date has been based on measured total pressures with little consideration given to the effects of induced pore pressures. Lutenegger and Kabir (1988) and Robertson et al. (1988) show that the corrected C reading can be used to estimate penetration pore pressures in fine grained soils. Using the research DMT developed at UBC, Tsang (1987) and Campanella and Robertson (1989) show that:

- The DMT and SBPMT pressure expansion curves are very similar.
- Tests in sand (at McDonald Farm) are drained with no excess pore pressures generated during installation of the blade or expansion of the membrane.
- Tests in soft clay indicate high pore pressures exist immediately after penetration. Effective stresses are small and remain essentially constant throughout the test. The \( p_2 \) closing pressure is similar to the CPTU penetration pore pressure measured behind the friction sleeve, as concluded also by Lutenegger (1988).
- In stiff compacted clay, the induced excess pore pressure may be small or negative and the effective stress on the membrane may be large. The closure pressure (\( p_2 \)) appears to be unrelated to the equilibrium pore pressure or penetration pore pressure.

From the above, it would appear that the stress increment due to blade penetration in soft to firm clays may be related to the induced excess pore pressure. The \( p_2 \) reading can be used to evaluate this excess pore pressure provided that:
• The soil is sufficiently impermeable so that no pore pressure dissipation occurs during the inflation and deflation of the membrane.
• In stiff overconsolidated soils, deflation may cause the membrane to separate from the soils, resulting in unreliable pore pressure measurements.

It would seem that, for a more rigorous interpretation of DMT data, a pore pressure transducer should be incorporated into the blade. The membrane could be installed on the expandable membrane (Tsang, 1987) or more simply on the reverse side of the blade (Lunne et al., 1987). The former condition would perhaps permit a more complete treatment of test data.

A.5 HYDRAULIC FRACTURE TEST (HFT)

Hydraulic fracture testing is a process whereby the pore pressure in an isolated soil stratum is sufficiently high that it causes a crack to propagate in the soil. Crack propagation terminates when the head losses in the crack are such that no further crack development is possible. The use of large fluid pressures to fracture in situ sediments has long been used in the petroleum industry to increase well recovery since fracture propagation increases the bulk conductivity of the soil deposit. Here, fracture propagation was considered to be in the direction perpendicular to the minor principle stress, \( \sigma_3 \), and that the pressure required to cause cracking was equal to or greater than \( \sigma_3 \).

The first application of hydraulic fracture tests in the geotechnical field was to examine the cracking and erosion of embankment dam cores due to large seepage pressures (Vaughan et al., 1970). Hydraulic fracture was also
known to be a complicating factor when performing field permeability testing and recommendations pertaining to maximum pressures were outlined (Bjerrum et al., 1972). Based on a simplified elastic analysis, Bjerrum et al. (1972) showed that the critical pore pressure ratio, $u_c/\sigma'_v$, depended on the $K_0$ value of the soil. They showed that cracking could result from several phenomena:

- When $\sigma'_v$ reduces to zero due to increased seepage force the soil undergoes a slight expansion and separates from the piezometer. Tavenas et al. (1975) termed this the pressuremeter effect.
- When the effective tangential stress, $\sigma'_\theta$, reduces to the $S_u$ of the soil.
- When $u > \sigma'_v$.

and some consideration was given to the stress changes in the soil caused by piezometer insertion.

The hydraulic fracture test is performed in a previously installed piezometer after all excess pore pressures have dissipated. The equilibrium porewater pressure, $u_0$, is recorded prior to commencing the test. The fluid pressure in the system is then increased incrementally; the duration of each increment can be long enough to allow the coefficient of permeability to be calculated if desired, i.e. flow rate stabilized. Alternatively, a rapid increase in pressure can be used to trigger undrained cracking. Once the soil fractures, evidenced by a large increase in flow rate (Fig. A.14), no further pressure increase is applied; rather the change in pressure with time is monitored. As flow into the soil continues, the pressure drops. A sudden drop in flow rate again occurs when the crack closes up.

An advantage of the HFT is that it requires simple equipment and can be performed quickly and cheaply. The repeatability of the test, however, has been questioned.
Fig. A.14 Typical data from a hydraulic fracture test.

Bjerrum and Anderson (1972), and Vaughan (1972) use the crack closure pressure, $u_c$ (or critical pressure), for calculating $\sigma_h$ since the crack opening pressure, $u_f$, is a function of both $\sigma_h$ and the soil tensile strength, $\sigma_t$:

$$u_f = m \sigma_h + \sigma_t \quad (A.29)$$

where $m = 1-2$ if no previous fracture exists. If a previous fracture exists, $\sigma_t = 0$, $m = 1$, $u_f = u_c$, and ideally:
which corresponds to crack closure or re-opening.

Bjerrum and Anderson (1972) show that flow into the crack is related to $(u_c - \sigma_3)^4$, so that a sharp change will occur as $u_c$ approaches $\sigma_3$. However, this change may be masked for soils of higher permeability.

Hydraulic fracture tests can thus only be used to measure $\sigma_h$ when $K_0 < 1$ (vertical crack formed) and where the soil permeability is below about $10^{-5}$ m/s. When $K_0 > 1$, a horizontal crack will form and the closure pressure should correspond to the overburden pressure at that depth. Notwithstanding this, several cases exist where $K_0$ values greater than unity has been recorded, although 1.2 to 1.4 appear to be an upper limit. Penman (1976) suggests that for tests carried out in embankment dams, $u_c$ is always greater than $\sigma_h$ and generally approaches the mean normal stress, $(\sigma_v + \sigma_n)/2$, during construction. As construction pore pressures dissipate, the value of $u_c$ reduces and approaches $\sigma_h$.

Certain questions about the validity of the test have been raised subsequent to the often contradictory field experience, namely:

- Disturbance due to piezometer insertion causes the stress state in the soil to be modified thus giving higher stress readings. However, since the zone of influence of reconsolidation is only about 2D to 3D ($D =$ diameter of piezometer) and is relatively small relative to the length of the crack formed (Massarsch, 1978), use of the closure pressure to evaluate $\sigma_h$ should avoid the effects of stress disturbance. The effects of arching during reconsolidation should also be minimized in this way. An analysis performed by Massarsch (1978), modelling the insertion of the piezometer as the expansion of a cylindrical cavity,
suggests that fracturing will occur due to installation and that subsequent crack opening will occur on pressurization. The existence of a crack opening pressure, $u^*_f$, and crack closure pressure, $u^*_c$, that are very different would refute this simplified analysis.

- The initial stress field may also be modified by the pressuremeter effect described earlier (Tavenas et al., 1975). It is also likely that this effect on $c_g$ will be opposite to that due to the effects of disturbance so that the net effect may be negligible (Tavenas, 1975).

- The interpretation of the fracture mechanism is complex and requires assumptions relating to principal stress directions, development of seepage pressures, deformation characteristics and anisotropy of the soil being tested. More importantly, experience suggests that fracture can occur over a wide range of pressures (Wroth, 1984).

- The closure pressure may be affected by the amount of fluid penetration that occurs subsequent to crack formation. Marr (1974) evaluated results from tests in Boston Blue clay and found that,

$$\frac{u^*_f - u^*_c}{u} = 1.4 \quad \text{(A.31)}$$

- The effects of soil structure in controlling fracture propagation are indeterminate (Massarsch et al., 1975).

- Geometry of the test set up.

- Time dependent behaviour may be important.

These last two factors were investigated by Tavenas et al. (1975) and Lefebvre et al. (1981). Tavenas et al. (1975) showed that the critical
pressure in soft clay was time dependent and only stabilized 100 days after complete dissipation of excess pore pressures resulting from piezometer installation (Fig. A.15). Following the analysis of Bjerrum et al. (1972) the tangential stress change due to piezometer insertion is given by

$$\sigma'_\theta = (1-\alpha) K_0 \sigma'_V$$  \hspace{1cm} (A.32)
where $\alpha$ is a function of soil compressibility and relates to the disturbance caused by piezometer installation. It is also suggested that $\alpha = f(t)$ in soft clay and that as the soil creeps $\alpha \to 0$. Hence, after a sufficient relaxation period, $\sigma'_0$ will attain the in situ pre-disturbance value. However, in some soils, creep will be limited and some residual stress change will result. A reduction of $u_c$ of between 24% and 28% over a period of 28 months was reported by Lefebvre et al. (1981) who also studied the orientation of the fracture patterns in soft clay.

They concluded that two principal patterns were present; radiating vertical fractures which developed around the piezometer tip and an inverted fracture cone with the apex at the piezometer tip. The presence of the cone indicates the larger influence of $\sigma_v$ on the critical pressure. Lefebvre et al. (1981) also found that the length of the piezometer tip had a pronounced effect on both the fracture pattern and the critical pressure (Fig. A.16); a L/D ratio of 10 would appear to be a minimum requirement; this fact may render much of the previously reported data of little use.

The uncertainties with the test still remain and little recent research has been performed; that which is reported is very contradictory. The mechanisms of cracking, whether undrained or partially drained, have been seriously questioned as has the repeatability of the data obtained. Errors in duplicate tests range between 1% and 10%. It may well be that the HFT will revert to providing an index of intactness for the cores of earth dams, and for limiting pressures during permeability testing, from whence the test was developed.
Gloucester Test Fill
Hydraulic fracture tests
Data from Lefebvre et al. (1981)

\[ K_0 = \frac{u_c - u_o}{\sigma_v' - u_o} \]

Length of piezometer (cm)

Fig. A.16 Effect of length of the piezometer tip on calculated \( K_0 \) values from HFT (modified after Lefebvre et al., 1981).

A.6 LATERAL STRESS CONE PENETRATION TEST (LS-CPTU)

The development of cone penetrometers capable of measuring lateral stress during penetration originated from the idea that the sleeve friction stress, \( f_s \), should be related to the pre-penetration or in situ horizontal
stress. Measurement of the lateral stress via an instrumented sleeve, would be a more rational approach as it obviates the need for interpreting the soil-steel interface friction angle. Due to the effects of penetration it was expected that some degree of amplification of the in situ lateral stress would occur. This is discussed further in Section A.11 when considering the use of CPT parameters for indexing lateral stress variations.

The first lateral stress sensing cone penetrometer (LSSCP) was developed at Berkeley (Huntsman, 1985) for laboratory tests in a calibration chamber (CC). Huntsman also used a subsequent model (LSSCP Model II) for field use, the main design changes relating to the cone resistance and sleeve friction load cell capacities. The lateral stress sensors in both models are located on upper and lower friction sleeves. A section of the friction sleeve was machined from the inside to a wall thickness of 0.25 mm. Strain gauges were bonded to the inside wall to measure the circumferential strain in the section. The system is calibrated in the laboratory to correlate strain gauge output to an applied hydrostatic pressure. For the laboratory penetrometer, the strain gauges were placed in a quarter wheatstone bridge arrangement. A half bridge arrangement was used for the field (Model II) LSSCP to reduce the temperature sensitivity of the lateral stress measurement system. On each cone two lateral stress sensing locations were employed. The lower section was located a distance of one cone diameter (1D) behind the tip. For the Model I LSSCP the upper section was located nine diameters (9D) behind the tip and pore pressure was measured 10D behind the tip. For the Model II cone the upper section was at 7.5D and no pore pressure measurement incorporated (Fig. A.17).

The laboratory CC data showed that the lateral stress measured by the lower instrumented friction sleeve was dependent on both the pre-penetration
Fig. A.17 Schematic details of lateral stress sensing cone penetrometer model II developed at Berkeley (after Huntsman, 1985).
stress and the state (density) of the sand. Reliable data for the upper sleeve was not obtained since due to the distance behind the tip, adequate penetration into the chamber was not possible. Huntsman (1985) used both relative density ($D_r$) and state parameter ($\psi$) as indicators of state to evaluate the laboratory CC data. The scatter in the data using either $D_r$ or $\psi$ was identical which suggests that either parameter should provide equally reliable indices for evaluating the amplification effect of data from in situ tests.

Field tests using the LSSCP were performed in the Canadian Beaufort Sea and in the Santa Barbara channel. At the location of a caisson retained island (CRI) results of tests in a fine to medium sand with the lower friction sleeve gave a horizontal stress profile in good agreement with that determined using the SBPM. This concurs with the CC data which suggest that at relative densities of about 40% the amplification factor is unity, i.e. $\sigma_{LS} = \sigma_h$, where $\sigma_{LS}$ is the total horizontal stress measured by the LSSCP. The upper lateral stress sleeve underwent a zero shift of $-369$ kPa and the data are considered unreliable (Huntsman, 1985).

The lateral stress coefficient derived from lateral stress cone data, designated $K_{LS}$, is obtained from (Campanella et al., 1990):

$$K_{LS} = \frac{\sigma_{LS} - u_{LS}}{\sigma_v - u_0} = \frac{\sigma'_{LS}}{\sigma'_v}$$  \hspace{1cm} (A.33)

where $u_{LS}$ is the pore pressure measured at the sleeve location. Furthermore, the amplification of lateral stress, from LSSCP data, $A_{LS}$, is defined as:

$$A_{LS} = \frac{\sigma_{LS} - u_{LS}}{\sigma_h - u_0} = \frac{\sigma'_{LS}}{\sigma'_h}$$  \hspace{1cm} (A.34)
Data from the Santa Barbara tests were unreliable and again the upper sleeve data were unusable. Both lateral stress sleeves collapsed under the high normal stresses and axial loads sustained. The field data were not corrected for friction cross talk (Masood, 1990). Following the design of Huntsman (1985), Jefferies et al. (1987) used a 10 cm$^2$ piezocone but incorporating only a lower instrumented friction sleeve. Since CC tests were to provide calibration data for interpretation of field tests, the upper stress sleeve was not useful as stabilized readings could not be obtained in the chamber due to the length of the probe. An underreamed wall thickness of 0.45mm (13mm long, 1D behind the tip) was used with a full scale calibration of 4 MPa. The four channel digital horizontal stress cone (HSC) was temperature compensated and friction sleeve cross talk removed during data processing. Jefferies et al. (1987) performed both CC and field tests in Erksak sand with the HSC. They also reviewed the CC data obtained by Huntsman (1985) for Monterey #0 sand.

As mentioned in Chapter 1, to interpret the chamber effect on the HSC data, Jefferies et al. (1987) assume the same standardization factor for cone resistance, $q_c$, to apply to $\sigma'_{LS}$ since no comparison data exist. For the reasons stated earlier, the assumption that $q_c/\sigma'_{LS}$ is independent of chamber size probably introduces errors into the data. That aside, the following relationships were derived from the CC data:

$$\frac{q_c - P}{P'} = k e^{m\psi}$$

$(A.35)$

$$A_{LS} = a e^{b\psi}$$

$(A.36)$
where

\[ p = (\sigma_1 + \sigma_2 + \sigma_3) / 3 \]

\[ k, m, a, b \] are coefficients from CC tests

Jefferies et al. (1987) eloquently outline the problem of determining \( \sigma_h \) from \( \sigma_{LS} \) since the measured quantities \( (\sigma', q_c, \sigma_{LS}) \) are dependent on those being sought \( (\sigma_h, \psi) \). Since, for CPT in sands,

\[ \frac{q_c - p}{p'} = \frac{q_c}{p'} \]  \hspace{1cm} (A.37)

a simplified algorithm was deduced, given by Jefferies et al. (1987):

\[ \sigma_h' = \frac{f}{\beta} \sigma_h'^\alpha + \frac{\sigma_v}{2} = 0 \]  \hspace{1cm} (A.38)

where

\[ f = \frac{q_c}{\sigma_{LS}'} \]  \hspace{1cm} (A.39)

\[ \alpha = m/b \]  \hspace{1cm} (A.40)

\[ \beta = 2/3 \ (a/k)^{-\alpha} \]  \hspace{1cm} (A.41)

For the Erksak sand tested in the CC, \( \alpha=1 \) and a linear algorithm results:

\[ \sigma_h' (1 - \frac{3a}{2k} \cdot \frac{q_c}{\sigma_{LS}'} - \frac{\sigma_v}{2} = 0 \]  \hspace{1cm} (A.42)
which can be applied to HSC data from CC tests. It is also implicit that the same equation should hold for field data. The algorithm does not work well for CC tests data and Jefferies et al. (1987) suggest that this is a consequence of the interdependence of \( q_c \) and \( \sigma'_{LS} \) in that the method is ill-conditioned to recover \( \sigma'_h \), i.e., small errors in \( q_c \) or \( \sigma'_{LS} \) cause rapid deterioration in precision. Jamiolkowski and Robertson (1988) review the Monterey #0 sand data of Huntsman (1985) and Jefferies et al. (1987) where the constants in Eqs. (A.35) and (A.36) are:

\[
\begin{align*}
k &= 40 \\
\text{m} &= -10.7 \\
\text{a} &= 0.32 \\
\text{b} &= -12.0
\end{align*}
\]

and they show that:

\[
\frac{K_{LS}}{K_o} = 0.000789 \left[ \frac{q_c - p}{p'} \right]^{1.44} \tag{A.43}
\]

The relationship above is shown in Fig. A.18. The sensitivity of \( K_o \) to the accuracy of \( K_{LS} \) and \( q_c / \sigma'_V \) is evident. However, it must be remembered that a similar situation was found for the DMT based on CC data but that field data showed the trend to be incorrect. It appears that this may be the case for the HSC data too. Measured horizontal stresses in the field at the CRI Molikpaq using the HSC gave good correlation with data from SBPM tests. It is also interesting to note that for the range of densities present in the island fill, the amplification of lateral stress, \( A_{LS} \), appears to be unity,
thus the form of the algorithm is not important. The CC data suggest that $A_{LS} = 1$ for $\psi = -0.1 \ (D_r = 40-50\%)$ and as such $\sigma_{LS} = \sigma_h$ and the use of the linear algorithm is an unnecessary refinement.

A Model III LSSCP was developed at Berkeley (Tseng, 1989; Masood, 1990). The schematic details of the LSSCP Model III are shown in Fig. A.19. The lateral stress sections, again located at 1D and 7.5D behind the tip, were modified in an attempt to avoid the earlier problems associated with the Model II version. The lateral stress section was 25mm long and consisted of
Fig. A.19 Schematic details of the Berkeley Model III LSSCP (after Masood, 1990).
an inner rig and an outer active ring. The outer active ring comprised four arciform pieces 1.3mm thick held together by a polyurethane compound. The cavity between the inner and outer ring is fluid-filled and sealed by a membrane. The inner ring has a strain gauged diaphragm which functions as a pressure transducer responding to increases in the cavity pressure caused by inward movement of the flexible outer ring. Pore pressures are measured adjacent to both lateral stress sections. Calibration chamber tests were performed on both Monterey #0 and Ticino sands, but again no data on the response of the upper stress sensors were obtained. Based on the CC data for both types of sand, Masood (1990) suggests that \( K_o \) can be estimated from:

\[
K_o = 0.36 + 0.27 K_{LS} - 0.0008 q_c/\sigma'_v
\]  

(A.44)

which has a similar form to the Baldi et al. (1986) DMT correlation. Equation (A.44), developed for the LS section 1D behind the tip, does not represent the best correlation for the CC data but has been adjusted to provide good agreement with field data for several sand sites. Masood (1990) suggests also that in clays, \( K_{LS} \) is a function of PI. Some problems were experienced with the upper lateral stress section and so at several sites reliable data were not obtained. However, where it is possible to compare the \( \sigma_{LS} \) profiles from the lower and upper sensors, the following comments can be made:

- The stress measured at the upper section is always larger than that measured at the lower section.
- The upper stress profile is smoother with less noise than the lower profile.
• No details of pore pressure variations at each lateral stress location are given.

The Model III LSSCP has been used by the author and has been discussed within the thesis. Details of the lateral stress cone developed at UBC are also given in the main body of the thesis.

A further advantage of the lateral stress cone is the possibility of evaluating the soil-steel interface angle, $\delta$. With the instrumented friction sleeve, both normal ($\sigma_{LS}$) and shear ($f_s$) stresses are measured, hence

$$f_s = \sigma_{LS} \tan \delta \quad (A.45)$$

No results have been presented to date concerning measured values of $\delta$ using LS-CPTU data.

A.7 PIEZO LATERAL STRESS CELL (PLSC)

The piezo lateral stress cell (PLSC) was developed at the Massachusetts Institute of Technology (MIT) with the objective of evaluating both stress and pore pressure changes during the installation, consolidation and subsequent loading of piles in cohesive soils (Baligh et al., 1985). As such, it was not intended to evaluate in situ $K_o$ conditions prior to installation, rather to predict the final disturbed stress state once full consolidation of the soil around the pile had occurred. The mechanism modelled is the same as that involved during the installation of cylindrical full displacement probes and as such the PLSC can provide important information on soil behaviour under these conditions.
The PLSC consists of a 60° tip extension followed by the piezo lateral stress cell of equal diameter. The PLSC (Fig. A.20) comprises a cylindrical stress cell to measure the total radial stress and a high air entry porous disc to measure pore pressure at the shaft-soil interface (Morrison, 1984). Axial load on the lateral stress cell and temperature are also measured during the test.

Both the stress cell and pore pressure systems are located sufficiently remote from the tip (typically 27D) that no influence of the tip occurs on the measured quantities. Baligh (1987) showed that the sensitivity of the PLSC is related to the following factors:

a) thickness of the water film behind the sleeve, or more specifically the volume of fluid in the chamber;

b) thickness of the steel sleeve, which however, must be sufficient to avoid damage during penetration; and

c) the square of the diameter of the lateral stress section.

Morrison (1984) discusses the probe in detail and evaluates both temperature and axial load effects on the stability of the lateral stress cell. The PLSC was tested initially at the Saugus, MIT research site and later at a MIT campus site. Two types of penetration are performed:

- Type D tests which correspond to 100% dissipation of excess pore pressures after stopping at various depth intervals.
- Type P tests whereby virtually no dissipation is permitted.

Measurements of both pore pressure and total stress during penetration for both type D and P tests are very similar; any variation can probably be explained due to soil variability. No definite conclusions can be made concerning the effects on the measured parameters of permitting full
Fig. A.20 Details of the piezo lateral stress cell (after Morrison, 1984).
consolidation around the probe, apart from the initial increased resistance on renewing penetration. Morrison (1984) defines two normalized total stress parameters for evaluating field data, namely:

\[
\frac{u_{PLSC} - u_0}{\sigma_v} \quad \text{and} \quad \frac{\sigma_{PLSC} - u_0}{\sigma_v'}
\]

where \(u_{PLSC}\) and \(\sigma_{PLSC}\) are the pore pressure and total horizontal stress measured by the PLSC. It is interesting to note for penetration in Boston Blue Clay (BBC) that the two ratios are very similar suggesting that during this stage, the level of effective stress around the probe is very low (Fig. A.21). Throughout the whole BBC profile both parameters vary between 2.0 and 3.0, with the \(\sigma_{PLSC}\) some 4% to 10% greater than \(u_{PLSC}\).

Morrison (1984), Azzouz and Baligh (1986) and Azzouz et al. (1990) show that the final fully consolidated value of \(K\) measured by the PLSC may differ from the initial \(K_0\) condition. In BBC (OCR = 1.3, PI = 21±3%) initial \(K_0\) and final \(K_{PLSC}\) values are identical (\(K_0 = 0.6\)), whereas in the more plastic Empire Clay (OCR = 1.5, PI = 56 ± 10%) the final \(K_{PLSC} = 0.9\) compared to \(K_0 = 0.72\). Using various soil models and somewhat limited field data, Azzouz et al. (1990) conclude that both OCR and clay type have a significant effect on the measured final stresses. They also suggest that during penetration, the measured stress ratio, \(K_{PLSC}\):

\[
K_{PLSC} = \frac{\sigma_{PLSC} - u_{PLSC}}{\sigma_v - u_0}
\]

is controlled primarily by the undrained shearing resistance of the soil at large strain levels. Consequently sensitivity will also be an important factor.
The PLSC measures the lateral pressure on the surface of a thin steel sleeve. The water filled cavity behind the sleeve transmits the pressure to a hydraulic transducer within the body of the cell. Complete de-airing of the system is thus important. Furthermore, the lateral stress measurements are very sensitive to axial load. While it may be possible to account for this by cross talk calibrations, it is more difficult to consider the change and/or redistribution of this load with time. The PLSC is delicate and thus only has been used for measurements in fine grained soils (silts and clays).
A.8 PREBORED PRESSUREMETER TEST (PBPM)

The prebored pressuremeter (PBPM) was developed by Menard in France and has been used very successfully for foundation design via the application of empirical correlations. These correlations have been developed using results from numerous case histories (Gambin, 1988).

As suggested by the name, the PBPM is placed in a previously drilled hole in order to carry out the test. In certain conditions, the probe can be placed in a metal lantern with a conical tip and the whole assembly can be driven into the ground. Once installed the test is performed by expanding the membrane until a limit pressure is reached. This is generally completed using between 8 and 14 equal volume or pressure increments, although 10 is the average number.

A typical set of PBPM expansion curves is shown in Fig. A.22. The shape of the curve is very dependent on the disturbance to the prebored hole and the effect of PBPM insertion (Tavenas et al., 1975). Usually the hole is slightly larger than the probe and curve (a) in Fig. A.22 results. At low pressures the probe expands rapidly to come into contact with the borehole wall and the soil is recompressed to the in situ horizontal stress ($p_{\text{n}}$). Thereafter, a linear portion results as the soil deforms elastically until yield occurs at $p_{\text{f}}$. Large increases in volume for small pressure increments occur after yield until the asymptotic limit pressure is attained. Where the probe and borehole has the same diameter, curve (b) results and the selection of $p_{0}$ becomes problematical. For curve (c) the borehole is smaller than the probe diameter, no recompression occurs and $p_{0}$ cannot be determined.

Initially, it was thought that $K_{o}$ could be calculated from $p_{\text{OM}}$ which corresponds to the start of the linear pressure-volume expansion; it can also
be determined from the creep curve (Baguelin et al., 1978). Theoretically, $P_{OM}$ should equal $\sigma_h$ if no disturbance/relaxation of the soil occurs. Disturbance, however, may considerably modify the PBPM curves within the limits shown in Fig. A.22. Also, accurate definition of $P_{OM}$ is difficult since limited data points are available at the early stages of the pressure controlled test; strain increment expansion is thus preferred if $P_{OM}$ is required. As a consequence, PBPM tests are not widely used for determining horizontal stress in the ground. Baguelin et al. (1978) in fact recommend

Fig. A.22 Effect of borehole conditions on PBPM results
(after Tavenas et al., 1975).
estimating $K_0$ from other soil data rather than using measured $p_{OM}$ values from the PBPM. Tavenas et al. (1975) performed tests in a sensitive clay and conclude that the SBPM is not well suited to determination of $\sigma_h$. Hartman and Schmertmann (1975) used a finite element technique to evaluate the elastic phase of the expansion curve and suggest that $p_{OM}$ may bear no relation to $\sigma_h$.

To overcome the problems associated with the direct use of $p_{OM}$ for calculating $K_0$, several methods have been suggested for evaluating a corrected pressure from the PBPM curve. Several of these were discussed in Section A.2, related to the self-boring pressuremeter as they have also been applied to interpretation of SBPMT data. Probably the most consistent of these is the Marsland and Randolph (1977) graphical technique discussed earlier. An elastic plastic soil model is used, $S_u$ calculated and iterations performed until:

$$p_{OM} + S_u = p_f$$

The method was developed for PBPM tests in stiff clays where mixed success has been reported. Powell (1990) suggests the use of a modified Marsland and Randolph technique wherein the shear stress, $\tau$, calculated at the break point (using the Palmer type analysis) is used in the iteration of Eq. (A.47) and not the peak shear stress. Mair and Wood (1987) suggest that PBPM tests should not be used for evaluating $\sigma_h$ unless the Marsland and Randolph iteration can be applied which requires an approximately linear response until yield occurs. This is probably only the case for high plasticity heavily overconsolidated clays (Powell and Uglow, 1985).

A recent development of the PBPM is the high pressure dilatometer (Clark and Allan, 1989) developed for performing tests in very stiff clays and soft
rocks. As with the other types of PM test, lift-off pressure does not correspond to the horizontal total stress and so a graphical technique has to be used for iterating best estimates of $\sigma_h$.

Tanimoto et al. (1981) use a PBPM with a built-in acoustic emissions transducer and pre-amplifier. The emission counts are recorded for each pressure increment and show marked changes at particular stress states. These changes facilitate determination of $p_0(p_y)$ and $p_f$ during PBPMT. It may be advantageous to install this equipment in SBPM or FDPM for similar purposes.

A.9 PUSH-IN PRESSUREMETER TEST (PIPMT)

Development of the push-in pressuremeter began in 1975 and was spurred by the need to measure soil stiffness in the offshore environment where the implementation of self-boring techniques was not considered feasible (Henderson et al., 1979). (Several offshore self-boring pressuremeters have since been developed.) The PIPM, also known as the stress probe, was developed as a wireline system comprising:

- hollow PM with protected adiprene membrane
- spacer section up which soil can pass
- pressure developer for membrane expansion

The front end of the PM has a inward tapered cutting shoe ($<10^\circ$) of 40% area ratio. The PM section is located about 0.9m (21D) behind tip and has an L/D ratio of 10. Probes of similar dimensions have been developed by others (Huang and Haefele, 1988).
The PIPM is installed by pushing from the base of an existing borehole and the soil passes up through the sampler and is recovered by retainers, if necessary. The pressure developer is used to pump fluid into the PM so that membrane expansion occurs.

On completing the PM test, the probe is withdrawn and the sample can be examined. Test interpretation is based on the prebored PM methodology since pressure-volume curves are produced. Henderson et al. (1979) state that the disturbance due to PIPM insertion depends on the degree of plugging that occurs in the sample tube; a plugged PIPM effectively becomes a full displacement PM. However, where plugging does not occur, they suggest that initial $p_0$ values correspond well to the estimated $\sigma_h$. In order to reduce disturbance, a 50mm diameter thin walled sample is first taken at the depth of interest. The 82mm diameter PIPM is then inserted, the cutting shoe shaving off the undersized hole.

Henderson et al. (1979) do not present data for evaluating $\sigma_h$, although they discuss a volume adjustment technique similar to that used with the PBPM to take into account the initial disturbance. Fyffe et al. (1986), Lacasse et al. (1990), Powell & Uglow (1985), and Powell (1990) all present data from PIPM tests performed at established research sites with little or no references to the determination of $\sigma_h$. In fact, they suggest that the PIPM cannot be used for this purpose. Huang and Haefele (1990) show that lift-off, $P_0(\text{PIPM})$, from the PIPMT is about 50% more than the SBPMT value and that in general:

$$P_0(\text{FDPM}) > P_0(\text{PIPM}) > P_0(\text{SBPM})$$

However, it should be possible to obtain estimates of $\sigma_h$ from PIPM data based on techniques developed for the PBPM. The main problem associated with the
PIPM is the varying degrees of disturbance that may occur to the penetrated soil; a qualitative estimate of the disturbance can only be made once the test has been completed and the sample recovered. It is considered that the PIPM may be superceded by newer more versatile offshore penetrometers which permit evaluation of the lateral stress in addition to modulus and strength.

A.10 FULL DISPLACEMENT PRESSUREMETER TEST (FDPMT)

The first purpose-built full displacement pressuremeter was developed at Laboratoire Central des Ponts et Chaussées in France (Baguelin and Jezequel, 1983). The 89mm diameter PM section (L/D = 4) was located behind a piezocone unit of the same diameter. Baguelin and Jezequel (1983) adopted the PBPM (Menard) technique for data interpretation without addressing the problem of lateral stresses.

Robertson (1982), with the aim of simplifying the PM installation process, presented data from FDPM tests; the FDPM consisted of a conical point inserted into the end of a SBPM. Tests performed at a sand site in the Lower Mainland of BC gave a wide scatter of $\sigma_h$ measurements. (Similar scatter was also obtained with SBPM measurements). Further work using the same system, supplemented by a Pencel FDPM, was performed by O'Neill (1985). Tests performed in a sensitive clay at 232nd St. in Langley gave $K$ values greater than one throughout the profile for both stress and strain controlled expansion tests. Even though the stress values are high it is interesting to note that (apart from near surface scatter above the water table) the profile of $K_{FDPM}$ with depth reflects closely the stress history variation in the deposit (Fig. A.23). Furthermore, no pore pressure measurements at the PM section were made and the $K_{FDPM}$ was calculated from:
Fig. A.23 $K_{\text{FDPM}}$ and OCR profiles in a sensitive clay.

$$K_{\text{FDPM}} = \frac{\sigma_{\text{FDPM}} - u_0}{\sigma_v - u_0} \quad (A.48)$$

Since excess pore pressures are generated during installation;
\[ \sigma_{FDPM} = \sigma_h' + u_0 + \Delta \sigma (=\Delta u) \]

Correcting for the effect of \( \Delta u \) may provide more reasonable \( K \) values especially if \( \Delta \sigma = \Delta u \) and no dissipation has occurred.

In 1986, details of the Fugro full displacement pressuremeter were published (Withers et al., 1986). An advantage of the cone pressuremeter is that \( q_c, f_s, u \) and the PM expansion test can all be performed in the same hole. Using a 15 cm² piezocone unit below the PM section the idea was that insertion of the probe induced repeatable degrees of disturbance that were operator independent (unlike the SBPM or the PIPM). Withers et al. (1986) suggested that if the FDPM was expanded to cavity strains of 20-30% (full expansion is to 50%) the plastic zone is moved out into soil unaffected by the mode of insertion and that the derived strength and stiffness parameters will be unaffected by the disturbance. Hughes and Robertson (1985) had previously shown that the unload-reload moduli in sand were not sensitive to the mode of insertion. Withers et al. (1986) report \( \sigma - u_0 \) values for sand similar to expected \( \sigma_h' \) values.

The most comprehensive field studies using the FDPM in sands and clays were performed at UBC using both the Fugro cone pressuremeter and the UBC seismic cone pressuremeter (Hers, 1989; Howie, 1991); in addition to CPTU and PM data, the UBC SCPM (Fig. A.24) allows in situ downhole shear wave velocities to be measured (Campanella and Robertson, 1986). Hers (1989) obtained estimates of the horizontal stress using the equation for limit pressure, assuming both cylindrical and spherical cavity expansion theory. He found that spherical theory gave \( K_0 \) values that best agreed with interpreted dilatometer values, but higher than the best estimates of \( K_0 \) for the soil. Similar results were also obtained by Powell (1990) using the same
Fig. A.24 Details of the UBC seismic cone pressuremeter (after Campanella and Robertson, 1986).
technique. The Houslby and Withers (1988) unloading analysis was used to determine $S_u$ and $G$ for cavity expansion calculations. The analysis is especially sensitive to the selection of an appropriate $S_u$.

Howie et al. (1990) and Campanella, Howie, Sully, Hers and Robertson (1990) evaluated the SCPM in sand and clay and concluded that instrument characteristics are important to the measurement of $\sigma_h$, especially in the case of the SCPM where the PM section is slightly undersized. While direct measurements of $\sigma_h$ may not be possible, they suggest that interpreted lift-off values may be feasible, and, in conjunction with data from other in situ tests, may provide an upper bound to the lateral stress condition. As demonstrated in Chapter 1, $\sigma_h$ measurements at McDonald Farm using both SBPM and FDPM probes show large scatter; the range of calculated $K_o$ is between 0 and 3; the SBPM results showing as much scatter as $\sigma_h$ from FDPM where $\sigma_h$ was also obtained from the lift-off pressure.

Recent research at Oxford University (Houlsby and Yu, 1990; Schnaid and Houlsby, 1990) has confirmed the use of cavity expansion theories for interpreting FDPM results. Calibration chamber tests suggest that both limit pressure, $p_L$, and $q_c$ are related to the horizontal effective chamber pressure, $\sigma'_h$ (Fig. A.25). The FDPM CC data gives the following relationships:

$$D_r = 9.0 \left[ \frac{q_c - \sigma'_h}{p_L - \sigma'_h} \right] - 30 \quad (A.49)$$

$$D_r = \frac{q_c - \sigma'_h}{3\sigma'_h} \quad (A.50)$$

Solving Eqs. (A.49) and (A.50) gives $\sigma'_h = f(p_L, q_c)$ as the root of a quadratic equation. A comparison between the estimated and calculated $\sigma'_h$ values is shown in Fig. A.26. The agreement is very good, especially at low $\sigma'_h$. 


Fig. A.25 Measured limit pressure as a function of $D_r$ and $\sigma_h'$ (after Schnaid and Houlsby, 1990).

Fig. A.26 Estimated and measured $\sigma_h'$ values from FDPM tests in CC (after Schnaid and Houlsby, 1990).
Before Eq. (A.50) can be applied to a field situation the relationship needs to be corrected for CC size effects; Eq. (A.49) is independent of chamber size whereas Eq. (A.50) is not.

The FDPM is a relatively new piece of equipment and more basic research is required prior to its general use in geotechnical engineering. Due to the conditions of the test, initial attempts at interpretation, especially to obtain lateral stress may be empirically based.

A.11 STEPPED BLADE TEST (SBT)

The Stepped Blade Test was developed at Iowa State University and first reported by Handy et al. (1982). The idea of using a plate configuration with mounted pressure cell was based partly on the earlier work by Marchetti with the dilatometer. The blade was initially designed as a three-stepped flat penetrometer with blade thicknesses of 3.0, 4.5 and 6.0mm. A fourth step with a thickness of 7.5mm (Fig. A.27) was later added to resolve problems associated with data interpretation from the 3 stepped blade (Lutenegger and Timian, 1986).

Each blade is instrumented with a passive pneumatic total stress cell to measure the horizontal pressure acting on the face of the blade. The blade is 63.5mm wide with the steps spaced at 100mm. The philosophy behind the blade was to provide an instrument which would be capable of:

i) inducing repeatable but differing degrees of disturbance to the soil from which a "zero disturbance" stress value may be interpolated (Fig. A.27); and

ii) avoiding the necessity of long dissipation times usually associated with other in situ stress measurement techniques.
The stepped blade test is usually performed in prebored holes, advancing the blade to a short distance below the base of the borehole into undisturbed soil. Recently, however, some tests have been performed by pushing directly from the ground surface (Lutenegger and Timian, 1986). The test is carried
out by pushing the blade into the ground at a rate of 1-2 cm/s until the first sensor on the blade (t = 3.0mm) is at the desired depth. The pressure acting on the sensor is then measured by gradually increasing the pressure behind the sensor membrane until membrane lift-off occurs. The blade is advanced sequentially until measurements on all the sensors on the blades of differing thickness have been completed at the depth of interest.

Based on laboratory calibration tests, Handy et al. (1982) proposed that the total stress-displacement (thickness) relationship is given by:

\[ p_0 = a p_1 e^{-nt} \]  \hspace{1cm} (A.51)

where

- \( p_0 \) = in situ horizontal total stress
- \( p_1 \) = pressure on a step of thickness, \( t \)
- \( a, b \) = regression parameters from \( t \)-log \( p \) plot of data

The value of 'a' has been suggested as unity which gives b as the slope of the linear \( t \)-log \( p \) relationship having an intercept of \( p_0 \) at \( t=0 \) (Fig. A.27). This recommendation is based on laboratory tests under conditions such that the results should be considered qualitative rather than quantitative (Jamiolkowski et al., 1985). More recent results would suggest that an 'a' value of 0.5 might be more appropriate. Data collected by Handy et al. (1982) suggest values of b in the range 0.12mm\(^{-1}\) for soft soils to 0.48mm\(^{-1}\) for stiff soils.

The acceptance of the SBT for evaluation of lateral stress conditions has not occurred due to various problems, some related to operation, others to interpretation of data. Firstly, the equipment is delicate and is easily
damaged when penetrated into undisturbed ground, even in soft soil. To resolve this, a reinforcing steel rib has been added to the rear side of the blade. In addition, measurement of lift-off pressures was sometimes erroneous due to membrane sticking—this has been corrected by use of a back-pressured pneumatic pressure cell (Handy et al., 1990). More importantly, doubts as to what is exactly being measured have arisen.

According to the log p-t relationship in Fig. A.27, as the stepped blade is penetrated, successively higher pressure measurements should be obtained. As reported by Handy et al. (1982) this is not always the case and thus casts doubts over the methodology employed in the SBT.

The results obtained are represented as the five generalized log p-t data trends are shown in Fig. A.28 and the following points relate to the data interpretation in terms of characteristic soil behaviour:

- The linear t-log p plot in Fig. A.28a is claimed to be compatible with the e-log p' consolidation relationship for soils and thus indicative of a consolidating response (Handy et al., 1990). Since the SBT is an undrained test when performed in fine grained soils no consolidation would ideally occur. The idealized semilogarithmic relationship may be a consequence of laboratory tests performed on non-saturated materials.

- In some instances, the first step (t = 3.0mm) pressures were higher than those measured on the second step (t = 4.5mm). This behaviour was not seen in the laboratory test with remoulded samples and is designated as an elastic response (Fig. A.28b). This may be the correct interpretation for cemented materials and very stiff heavily overconsolidated clays. However, very little data exist to verify this mechanism. Conversely, self-boring pressuremeter tests in
Fig. A.28 Typical log pressure - thickness plot from SBT and inferred soil response (after Handy et al., 1990).
Taranto Clay (heavily overconsolidated, cemented hard clay) suggest that the elastic response range occurs at displacements much below that induced by the first step penetration during the SBT. Furthermore, the "elastic" response has been recorded in soft normally consolidated clays.

- At some stage during the SBT a limit pressure is reached and no further increase in pressure occurs (Fig. A.28c,d). This response is most common in soft clay where Handy et al. (1990) suggest it reflects the development of excess pore pressures caused by thicker blade penetration. However, although SBT data does not exist to verify the above, it is almost certain that in saturated soils development of excess pore pressure occurs even for the thinner blades. The attainment of a blade limit pressure does not infer development of excess pore pressure, rather that the soil is at a state of failure within the zone of influence of pressure measurement.

Notwithstanding the above, the acceptance of the stepped blade test as a reliable indicator for lateral stress conditions probably depends on the rigorous evaluation of test data by means of basic research. Much of the data in the literature have been obtained at geologically/geotechnical complex sites (Handy et al., 1982,1990). Data obtained from geotechnically well-known sites suggest that the horizontal stresses obtained from the SBT are much higher than those from other in situ stress measurement techniques (Lutenegger and Timian, 1986a,b). Further verification of the technique is required before confidence can be placed in the interpreted data.
It would appear that pore pressure measurement should be incorporated as a routine component of test procedure. The results of research with other full displacement probes suggest that the generation, redistribution and dissipation of excess pore pressures control the soil response around probes. Indeed, the extrapolation of pressures measured during the SBT may have little to do with the effective stress increase in the ground caused by penetration but rather reflect the changing pore pressure effects.

The results of the extrapolation method provide a value of the total horizontal stress, \( \sigma_{SBT} \), for a zero thickness with no bedding errors (Jamiolkowski et al., 1985). The lateral stress coefficient from the SBT, \( K_{SBT} \), is then calculated as:

\[
K_{SBT} = \frac{\sigma_{SBT} - u_0}{\sigma'_v} \tag{A.52}
\]

Since it is proposed that \( \sigma_{SBT} \) will contain an excess pore pressure component, it must follow that \( K_{SBT} > K_o \) where \( \Delta u \) is positive.

The development of an electric \( K_o \) stepped blade was reported by Tse (1988). Both total pressure and porewater pressures were measured on the two-stepped blade (3.0 - 4.5mm) and showed that the effective stresses were only 30-50% of the measured total stresses for the loamy alluvium tested. Furthermore, as suggested above, the increase in pressure between the two steps resulted almost entirely due to pore pressure increase. This would confirm the necessity for an effective not total stress extrapolation thus requiring the monitoring of stresses and pore pressures with time. Failing this, an interpretative technique needs to be developed so that the penetration effects can be fully accounted for, even if only by empirical correlations. In addition, in some soils the viscoelastic behaviour may also
be important in determining the final equilibrium lateral stress after penetration.

A.12 TAPERED BLADE TEST (TBT)

Lutenegger and Timian (1986) suggest that the successive step deformations associated with the stepped blade test may cause significant remolding of the soil close to the blade. They suggested the use of a continuously tapered blade as an alternative way of obtaining the pressure-thickness relationship. Mitchell (1988) reports that such a device has been developed and is undergoing evaluation. The instrument reportedly has seven stress sensors along one face of a blade with a 3 degree taper (Fig. A.29). It is probable that this test method will also require the measurement of excess pore pressures at locations along the blade in order to permit a rational interpretation of the measured total stresses. Furthermore, due to the taper, the measured stresses will not be in the horizontal plane and as a result, some component of the pushing force required to install the blade may be recorded.

A.13 FIELD VANE TEST (FVT)

The in situ vane shear test has been used primarily for evaluating the undrained shear strength of fine grained soils for incorporation in undrained (\(\phi=0\)) stability analysis of footings, slopes and embankments. The test, in its modern form was presented by Cadling and Odenstad (1950) and consists of measuring the torque required to rotate a cruciform vane of certain dimensions. The field vane test may be performed below the base of boreholes or by pushing the vane inside a sheath for protection; in either case, the vane
is advanced at least 0.5m below the level of disturbed soil before the test is commenced. Assuming the formation of a cylindrical shear surface around the vane, the relationship between the maximum applied torque, $T$, and the undrained shear strength, $S_u$, is given by:

$$S_u \text{(FV)} = \frac{2T}{\pi D^3 (\frac{H}{D} + \frac{a}{2})} \quad \text{(A.53)}$$
where:

\[ D = \text{diameter of vane} \]
\[ H = \text{height of vane} \]
\[ a = \text{factor that depends on assumed shear distribution (a = 2/3, if distribution is uniform as is usually assumed).} \]

For the general case of \( a = 2/3 \) and \( H/D = 2 \) (ASTM D-2573) the relationship becomes:

\[ S_{u(FV)} = \frac{6T}{7\pi D^3} \tag{A.54} \]

This interpretation suggests that the vertical surface of the failure cylinder contributes 86% of the resistance to total torque. More recent research suggests that the stress distribution on the horizontal faces of the vane is highly non-uniform with large stress concentrations occurring at the corners (Fig. A.30). As a result, if no strength anisotropy exists, the vertical surfaces contribute 94% of the total torque (Wroth, 1984).

It has long been acknowledged that the results of the FVT are influenced by the in situ effective stresses (Schmertmann, 1975). Consequently, considering the dominant effect of the vertical failure surface of the FVT, it would appear logical that the obtained undrained shear strength would be an indicator of the lateral stress condition.

For a uniform stress distribution on a vane with \( H/D = 2 \) in a soil with no strength anisotropy (drained or undrained), Schmertmann (1975) suggested that:

\[ \frac{S_u}{\sigma'_{V}} = \frac{c'}{\sigma'_{V}} + \left[ \frac{6K + 1}{7} + \frac{\Delta \sigma_p}{\sigma'_{V}} + \frac{\Delta \sigma_r}{\sigma'_{V}} \right] \tan \phi' \tag{A.55} \]
Fig. A.30 Shear stress distribution on the cylindrical surface described by a rotating vane - a comparison of various theories (after Chandler, 1988).

where

\(c', \phi'\) = the triaxial drained strength parameters for the clay which are unaffected by vane insertion

\(\Delta \sigma'_p\) = average effective stress change due to vane penetration

\(\Delta \sigma'_R\) = average effective stress change due to vane rotation
If one assumes that during the test $\Delta \sigma = \Delta u$ and that $c'/\sigma'_v$ is negligible, then for normally consolidated to lightly overconsolidated soils:

$$S_u = \left(\frac{6K + 1}{7}\right) \sigma'_v \tan \phi'$$  \hspace{1cm} (A.56)

which clearly demonstrates the effect of $\sigma'_h$ on $S_u$. The error in using this simplified form increases as the undrained cohesion, $c'$, increases, is accentuated in near surface overconsolidated layers where $\sigma'_v$ is small and relies on the undrained status of the test.

Wroth (1984) presents a speculative evaluation of the FVT, likening the failure mode to that in the direct simple shear test. Evidence reported by Chandler (1988) would suggest that this is a valid approximation at least up until the moment of failure. Assuming that the direct stresses on the vane do not alter during shear, the maximum shear stress, $\tau_{\text{max}}$, mobilized by the vane is given as:

$$\tau_{\text{max}} = \sigma'_h \sin \phi'_{\text{ps}}$$  \hspace{1cm} (A.57)

where $\phi'_{\text{ps}}$ is the drained friction angle for plane strain conditions. Thus,

$$\frac{S_u}{\sigma'_v} = K_o \sin \phi'_{\text{ps}}$$  \hspace{1cm} (A.58)

A comparison of Eqs. (A.56) and (A.58) is shown in Fig. A.31 for various $K_o$ and $\phi$ values. The $\phi'_{\text{ps}}$ values have been calculated using the empirical relationship:
Fig. A.31 Undrained strength ratios for differing $K_o$ conditions as given by Eqs. (A.56) and (A.58).

\[ \phi'_{ps} = \frac{9}{8} \phi'_{tr} \]  

(A.59)

Aas et al. (1986) propose a method to evaluate $K_o$ from a combination of FVT and triaxial ($CK_oU$) undrained strength data using a graphical technique in normalized stress space. Again, by assuming $\Delta \sigma' = 0$ during the test, Aas et al. (1986) suggest that the effective normal stresses in the horizontal
Direction acting on the vane blades at failure are given as \( K \sigma'_{vo} \pm S_{uv} \) where \( S_{uv} \) is the strength measured tangentially to the vertical cylinder. If shear stresses equal to the remoulded strength, \( S'_{uv} \), exist on the soil-vane interface, then the minor principal (horizontal) stress at failure, \( \sigma'_{hf} \), is given by:

\[
\sigma'_{hf} = K \sigma'_{vo} - (S_{uv} - S'_{uv})
\]  

(A.60)

where \( \sigma'_{hf} \) can be obtained from \( CK_U \) triaxial tests on samples taken at the same depth as the field vane test with \( \sigma'_{1} = \sigma'_{vo} \). The assumption here is that \( \sigma'_{hf} \) is the same in the FVT as in the triaxial compression test. \( S_u \) and \( S'_{ur} \) from the FVT are substituted for \( S_{uv} \) and \( S'_{uv} \) in Eq. (A.60). The method appears to work well for the Norwegian clay sites studies. However, it would appear that an a priori knowledge of \( K_o \) is necessary in order that the \( CK_U \) stress path may be determined. It would be interesting to see how the assumed laboratory \( K_o \) values affect the final field estimates. Error is also introduced by the fact that

\[
S_u (TC) \neq S_u (FVT)
\]

(A.61)

which becomes more pronounced in low plasticity clays.

In a recent paper Becker et al. (1988) also conclude that the results of FVT are controlled by the current in situ effective stresses. Vane strengths are normalized by the mean normal stress, \( I'_o \), where:

\[
I'_o = \frac{\sigma'_{vo} + 2\sigma'_{ho}}{3} = \frac{1 + 2K_o}{3} \sigma'_{vo}
\]

(A.62)
The resulting scatter in normalized strength ratios is much less than that normally recorded when data is normalized with $c'_{vo}$ but the data are inconclusive since the $K_o$ values used are predominantly greater than 1.

More recently, the vane test has been applied to granular soils to evaluate the steady-state strength (Finn, 1989) and relative density (Atkinson and Jessett, 1988). It is unlikely that this type of test could be used to evaluate the lateral stress condition since appreciable effective stress changes may occur due to penetration and rotation of the vane.

The dependence of $S_u$ on the stress history (OCR) of a soil is well-known. Based on the above, it may be possible to use a simplified relationship to evaluate $K_o$ based on $S_u/c'_{vo}$ measurements.

A.14 SELF-BORING LOAD CELL (SBLC)

The self-boring load cell is the original Camkometer developed at Cambridge University (Hughes, 1973; Wroth and Hughes, 1974). The SBLC does not have an expandable membrane. The probe is fitted with two bending web load cells (C and D) located on opposite sides of the probe near its mid-point. The circular stress sensitive areas have a diameter of 44.4mm; the probe diameter is 80.4mm. The pressure cells are flush with the surface of the instrument and produce an electrical output proportional to the difference between the external applied pressure (on the cell surface) and the internal gas pressure which is controlled by a surface regulator. Dalton and Hawkins (1982) quote a capacity of 280 kPa for the pressure cells with a maximum deflection of the web cell of 19 μm. Two pore pressure sensors are located between the total stress cells (Fig. A.32), and these are also
Fig. A.32 Self-boring load cell with cutter detail (after Dalton and Hawkins, 1982).

referenced to the internal gas pressure. The instrument is installed in the ground to the depth of interest by a self-boring method. Self-boring is achieved using a cutter in the shoe of the instrument and a circulating slurry which removes the soil cuttings to the surface (Fig. A.32).

Dalton and Hawkins (1982) performed tests in a stiff clay (Gault clay) and often it was necessary to wait days, in some instances even weeks, before
it was possible to perform the test (complete dissipation of excess pore pressures). Several tests were performed with the SBLC, including measurements at the same depth after various degrees of rotation of the probe. Their results are shown in Fig. A.33 for two complete circuits of rotation. Dalton and Hawkins (1982) suggest that the results indicate the non-isotropic nature of the in situ stresses; considering the maximum difference in stress measurement of 100 kPa at any one orientation, this finding is somewhat contentious. Also the variation in direction of major horizontal stress in the three test holes (spaced 2m apart) is inconsistent.

Fig. A.33 Results of SBLC in stiff clay after two complete rotations of probe (20° steps) at a depth of 5m (after Dalton and Hawkins, 1982).
From the data presented it would appear that the SBLC data are consistently lower than the SBPM results and show similar scatter.

Very little published data exist for the SBLC. Tedd and Charles (1983) present a comparison of SBLC, SBPM, TSC and capillary measurement data in London Clay; again the SBLC gives consistently lower results. More recently Huang and Haefele (1990) performed SBLC tests between 3.5m and 7.5m in a soft light overconsolidated (OCR = 1.5-3.0) marine clay. The authors concluded that a consistent direction of major lateral stress was evident from the field data, but that the differences in stress measurements were within the accuracy of the measurements. They suggest that in a stiff deposit where $\sigma_{\text{hy}} > \sigma_h$, a passive measuring device such as the SBLC is likely to provide more reliable measurements of $\sigma_h$ than the SBPM. No published data exist for the use of the SBLC in sand.

A.15 CONE PENETRATION TEST (CPT/CPTU)

The cone penetration test (CPT) with pore pressure measurement, also known as the piezocone test (CPTU), is now widely regarded as the optimum test for in situ profiling. Considerable insight into soil behaviour and response is also possible, generally via the use of well-established empirical relationships. Recent publications have provided excellent reviews of the capabilities of CPT/CPTU sounding (Schmertmann, 1978; Campanella and Robertson, 1988; Lunne et al., 1989). The increase in popularity of CPTU testing has been spurred by its practicality and as a result of considerable research both in the field and in the laboratory by means of calibration chambers. Furthermore, promising theoretical studies have provided a more
fundamental background to understanding soil response during penetration, removing the initial stigma of a completely empirical approach.

The results of CPT in sands in calibration chambers (CC) led to the following empirical relationship between cone resistance \( q_c \) and relative density, void ratio and effective stress (Schmertmann, 1976):

\[
q_c = C_0 \left( \sigma' \right)^{C_1} \left( e \right)^{C_2} D_r
\]  

Based on later data, Baldi et al. (1986) state that:

- In normally consolidated sands a unique relationship between \( D_r \) and \( q_c \) exists through \( \sigma'_v \).
- For both NC and OC sands, the unique relationship must relate to \( \sigma'_h \).
- The relationship derived from CC for any sand is dependent on sand compressibility, stress history, CC boundary conditions and initial state.

The CC data clearly showed the primary dependence of both \( f_s \) and \( q_c \) on \( \sigma'_h \). Results obtained by Veismanis (1974) demonstrated the dependence of \( q_c \) on both \( K_o \) and OCR in sand. Subsequent tests by Schmertmann (1978) also showed the dependence of \( q_c \) on stress history and he derived the empirical relation below:

\[
\frac{q_{OC}}{q_{NC}} = 1 + 0.75 \left[ \left( \frac{K_{OC}}{K_{NC}} \right)^{0.42} - 1 \right]
\]  

where \( q_{OC} \) and \( q_{NC} \) are the cone resistances measured in overconsolidated and normally consolidated sand, which, however, was not of practical use as some a priori knowledge of stress and penetration history is required.
Robertson (1982) suggested the possibility of evaluating $K_q$ from sleeve friction measurements during CPT based on a review of CC test data. Deducing the maximum dilation angle, $\nu_{max}$, from $q_c/\sigma_v'$, he produced Fig. A.34 showing the change in horizontal stress as a function of $\nu_{max}$. The method, however, appears to be ill-conditioned for estimating $K_q$ and is very sensitive to the choice of $\delta$.

\[
\text{Assume } \delta = 30^\circ
\]

\[
\sigma'_{HO} = K_0 \sigma'_v
\]

\[
f_s = K\sigma'_v \tan \delta
\]

\[
\sigma'_{H0} = \text{initial horizontal stress}
\]

\[
f_s = \text{sleeve friction}
\]

\[
\sigma'_v = \text{initial vertical stress (assumed constant)}
\]

Fig. A.34 Change in $K$ due to cone penetration in sand (after Robertson, 1982).
Huntsman (1985) evaluated CC data and showed that the measured sleeve friction was dependent on the applied horizontal chamber stress (Fig. A.35). Due to problems evaluating the penetrometer-soil interface friction angle, Huntsman developed a lateral stress sensing cone to provide direct measurements of $\sigma_h$ during CPT.

Masood (1990) presents a $K_o - f_s$ method based on CC data from tests performed at Berkeley. As in the Robertson (1982) method, he assumes that:

![CALIBRATION CHAMBER DATA FOR DRY SAND](image)

Fig. A.35 Influence of $D_r$ and chamber boundary conditions on $f_s - \sigma_h$ relationship from CC test data (Modified after Huntsman, 1985).
f_s = \sigma'_h \tan \delta \quad (A.65)

but uses the following assumptions:

\[ \sigma'_h = K_p \sigma'_v \]

\[ K_p = \text{passive lateral stress coefficient induced during CPT} = \tan^2(45 + \phi'/2) \]

\[ \delta = \text{soil-steel interface friction angle, } \phi'/3 \]

Combining the above into Eq. (A.65):

\[ f_s = \sigma'_v \tan^2(45+\phi'/2) \tan(\phi'/3) \quad (A.66) \]

Equation (A.66) is combined with the Jaky/Schmidt expressions:

\[ (K'_0)_{OC} = (K'_0)_{NC}(OCR) \sin \phi' \quad (A.67) \]

\[ (K'_0)_{NC} = 1 - \sin \phi' \quad (A.68) \]

to produce the relationship in Fig. A.36. Masood (1990) suggests that in sands, the OCR can be evaluated from the DMT. Estimates of \phi' presumably come from CPT data; this is problematical in clay. Masood (1990) applies the method to four known research sites (sand and clay) and reports good correspondence between measured and calculated horizontal stresses.

Whereas Baldi et al. (1986) suggest that \( q_c \) is a function of both \( \sigma'_v \) and \( \sigma'_h \), Houlsby and Hitchman (1988) suggest an almost total dependence on \( \sigma'_h \). Their data for a range of stress histories show that:
Fig. A.36 $K_0$ from sleeve friction measurements during CPT (after Masood, 1990).

$$\frac{q_c}{p_a} = A \left( \frac{c_h'}{p_a} \right)^{0.6}$$  \hspace{1cm} (A.69)

where $A$ is a constant related to the strength of the sand. They suggest that Eq. (A.69) may not give sufficiently accurate estimates of $K_0$. 
The correlations presented above indicate the dependence of both $q_c$ and $f_s$ on in situ $\sigma'_h$ for CPT in sand. In clays, where excess pore pressures are generated, similar correlations are also possible. Mayne and Kulhawy (1990) used cavity expansion theory to suggest a relationship between $K_o$ and both $(q_t - \sigma_v)/\sigma_v'$ and $\Delta u_v/\sigma_v'$. Field data obtained by the authors show that:

$$K_o = A \left( \frac{q_t - \sigma_v}{\sigma_v'} ; \frac{\Delta u_v}{\sigma_v'} \right)$$

which, although considerable scatter exists, agrees with the general trend suggested by the theory.

Further consideration of the stresses around full displacement probes is given in Appendix B.

A.16 IN SITU SHEAR WAVE VELOCITY MEASUREMENTS

The dependence of the low amplitude shear wave velocity on stress level has long been recognized both independently and in relation to the small strain shear modulus, $G_o$:

$$G_o = \rho \frac{V_s^2}{s}$$

where

- $\rho$ = bulk density of material
- $V_s$ = shear wave velocity
At small strains $V_s$ is dependent only on $\sigma'$ and $\varepsilon$ (or $D_r$):

$$V_s = C \sigma'^n$$  \hspace{1cm} (A.72)

with little or no effect of OCR (Hardin and Drnevich, 1972; Lee, 1985). The value of the exponent $n$ in Eq. (A.72) has been shown theoretically to be equal to $1/6$ where $\sigma' = \sigma'_v + \sigma'_h$ (mean normal stress). Roessler (1979) suggested that $V_s$ is dependent primarily on the stresses in the direction of wave propagation and in the direction of particle motion; the third principal stress having a negligible effect on $V_s$. This idea was extended by Knox et al. (1982) to demonstrate the effects of inherent anisotropy on $V_s$. The three individual stress method was proposed by them for describing the stress dependence of $V_s$:

$$V_s = C_s (\sigma'_a)^{n_a} (\sigma'_b)^{n_b} (\sigma'_c)^{n_c}$$  \hspace{1cm} (A.73)

Based on a series of tests in a large cubical sand specimen, Lee (1985) recommends this last approach as correctly modelling the characteristic cross-anisotropic behaviour of natural deposits. A similar conclusion is made by Yan and Byrne (1990) from $V_s$ measurements in a hydraulic gradient similitude model. The directions of wave propagation and particle motion associated with seismic shear waves from downhole and crosshole tests are shown in Fig. A.37.

Adopting the terminology of Lee (1985), the various stress methods available for describing $V_s$ dependences are (the subscripts A and I refer to wave motion in the anisotropic and isotropic planes where the horizontal plane is assumed to be isotropic in terms of stress, i.e. $\sigma_2 = \sigma_3$):
Fig. A.37 Directions of wave propagation and particle motion for shear waves in crosshole and downhole tests (modified after Stokoe et al., 1985).

a) Mean Normal Stress Method

\[
(V_s)'_A = C_D (\sigma_o)'_{\text{nm}} 
\]  \hspace{1cm} (A.74)

\[
(V_s)'_I = C_C (\sigma_o)'_{\text{nm}} 
\]  \hspace{1cm} (A.75)
where

\[ \sigma'_o = \left( -\frac{1 + 2K}{3} \right) \sigma'_v \]  \hspace{1cm} (A.76)

b) Average Stress Method

\[ (V_s)_A = C_D \left( \frac{\sigma'_v + \sigma'_h}{2} \right) \]

\[ = C_D (\sigma'_v)^{nt} \left( \frac{1 + K_o}{2} \right) \]  \hspace{1cm} (A.77)

\[ (V_s)_I = C_C \left( \frac{\sigma'_h + \sigma'_h}{2} \right) \]

\[ = C_C (\sigma'_v)^{nt} (K_o)^{nt} \]  \hspace{1cm} (A.78)

Equation (A.78) applies where particle motion and propagation are in the horizontal plane.

c) Individual Stress Method

\[ (V_s)_A = C_D (\sigma'_v)^{na} (\sigma'_h)^{nb} \]

\[ = C_D (\sigma'_v)^{na+nb} (K_o)^{nb} \]  \hspace{1cm} (A.79)

\[ (V_s)_I = C_C (\sigma'_h)^{na} (\sigma'_h)^{nb} \]

\[ = C_C (K_o \sigma'_v)^{na+nb} \]  \hspace{1cm} (A.80)
The ratios between the crosshole and downhole shear wave velocities for each of the stress methods can be written as:

(a) \[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_D}{C_C} \]  \hspace{1cm} \text{(Mean normal stress)}  \hspace{1cm} (A.81)

(b) \[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_D}{C_C} \left( \frac{1 + K_o}{2K_o} \right)^{nt} \]  \hspace{1cm} \text{(Av. stress)}  \hspace{1cm} (A.82)

(c) \[
\frac{(V_s)_A}{(V_s)_I} = \frac{C_D}{C_C} \left( \frac{1}{K_o} \right)^{na} \]  \hspace{1cm} \text{(Individual stress)}  \hspace{1cm} (A.83)

Equations (A.82) and (A.83) can be rewritten to give $K_o$ in terms of $(V_s)_A$, $(V_s)_I$, $C_D$, $C_C$, $nt$, and $na$. The problem then is one of obtaining the various constants incorporated in the above equations. A summary of the values of the exponents in Eqs. (A.74) to (A.80), based on laboratory tests, is shown in Table A.3. $C_D$ and $C_C$ are related to soil characteristics and density. It is often the case that laboratory measured parameters differ from those measured in the field due to the effects of structural anisotropy, fabric, aging, boundary conditions, etc. Gillespie (1990) evaluated downhole $V_s$ measurements in terms of the vertical effective stress, $\sigma'_v$ and $G_o$, where

\[
(V_s)_A = C_D (\sigma'_v)^n \]  \hspace{1cm} (A.84)

Therefore,

\[
G_o \propto (\sigma'_v)^{2n} \]  \hspace{1cm} (A.85)
Table A.3 Summary of Laboratory Results for $V$ Determination - Slope of $V - \sigma$ Relationships for Dry Sand (modified after Stokoe et al., 1985)

<table>
<thead>
<tr>
<th>References</th>
<th>nm</th>
<th>nt</th>
<th>na</th>
<th>nb</th>
<th>Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lawrence (1965)</td>
<td>0.25</td>
<td>-</td>
<td>0.06-0.16</td>
<td>0.08-0.17</td>
<td>Pulse test (small cylindrical sample)</td>
</tr>
<tr>
<td>Hardin &amp; Black (19866)</td>
<td>0.25</td>
<td>-</td>
<td>0.11-0.13</td>
<td>0.13-0.14</td>
<td>Resonant column test</td>
</tr>
<tr>
<td>Kuribayashi et al. (1975)</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>RCT</td>
</tr>
<tr>
<td>Iwasaki et al. (1978)</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>RCT</td>
</tr>
<tr>
<td>Schmertmann (1978)</td>
<td>0.19-0.47</td>
<td>-</td>
<td>0.09-0.12</td>
<td>-</td>
<td>Pulse test (cylindrical chamber)</td>
</tr>
<tr>
<td>Tatsuoka et al. (1979)</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>RCT</td>
</tr>
<tr>
<td>Roesler (1979)</td>
<td>0.25</td>
<td>-</td>
<td>0.149</td>
<td>0.107</td>
<td>Pulse test (cubical sample)</td>
</tr>
<tr>
<td>Uchida et al. (1980)</td>
<td>0.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>RCT</td>
</tr>
<tr>
<td>Knox et al. (1982)</td>
<td>0.2</td>
<td>0.11-0.18</td>
<td>0.12</td>
<td>0.09</td>
<td>Pulse test (large scale txl)</td>
</tr>
<tr>
<td>Allen &amp; Stokoe (1982)</td>
<td>0.24</td>
<td>0.24</td>
<td>0.12</td>
<td>0.11</td>
<td>RCT</td>
</tr>
<tr>
<td>Yu and Richart (1984)</td>
<td></td>
<td></td>
<td>0.25</td>
<td>0.12-0.14</td>
<td>0.11-0.14</td>
</tr>
<tr>
<td>Lee and Stokoe (1985)</td>
<td>0.20</td>
<td></td>
<td>0.10</td>
<td>0.10</td>
<td></td>
</tr>
<tr>
<td>Stokoe and Ni (1985)</td>
<td>0.22</td>
<td>0.11</td>
<td>0.11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stokoe et al. (1985)</td>
<td>0.18</td>
<td></td>
<td>0.09</td>
<td>0.09</td>
<td>RCT</td>
</tr>
<tr>
<td>Lew &amp; Campbell (1985)</td>
<td></td>
<td></td>
<td></td>
<td>0.28-0.4</td>
<td></td>
</tr>
<tr>
<td>Thomann &amp; Hryciw (1990)</td>
<td></td>
<td></td>
<td>0.125</td>
<td>0.125</td>
<td>Bender element oedometer</td>
</tr>
<tr>
<td>Yan and Byrne (1990)</td>
<td>0.24</td>
<td>0.24</td>
<td>0.12</td>
<td>0.12</td>
<td>Hydraulic gradient similitude model w/ bender elements</td>
</tr>
</tbody>
</table>
He reports 2n values of 0.9 ± 0.1 in clays and 0.7 ± 0.15 in sands at well documented sites. The implicit assumption involved here is that $K_o$ is constant with depth, so some scatter should be expected as a result. Lee (1985) reports n values from in situ tests that range from 0.2 to 0.39, having an average value of 0.31 as compared to 0.35 obtained by Gillespie (1990). Laboratory values of n range between 0.22 and 0.28. The sensitivity of $K_o$ obtained from Eqs. (A.82) and (A.83) will depend on the accuracy of shear wave velocity measurements, the $C_D/C_C$ ratio and, to a lesser extent, the exponent na or nt. $C_D/C_C$ values between 1.05 and 1.1 are reported in the literature for sand.

The small strain shear modulus, $G_o$, can also be related to the operating stress condition via the general equation:

$$G_o = K_g (\sigma_o')^m$$

(A.86)

where:

$K_g$ = a shear modulus constant dependent on $D_r$

$m$ = an exponent usually taken as 0.5 for sands and 0.85-1.2 for clays depending on PI

Various correlations between $K_g$ and $D_r$ exist. However, it is considered more appropriate to use the $V_s-\sigma'$ corelation where possible.

It appears that low amplitude ($V_s$)$_A$ and ($V_s$)$_I$ measurements from downhole and crosshole tests may allow estimates of $K_o$ to be made. Furthermore by applying Eqs. (A.82) and (A.83) bounds for $K_o$ can be determined. The advantage of the method in that essentially no disturbance occurs which may alter the parameter being measured.
A.17 ELECTRICAL METHODS

The objective of in situ measurement of electrical properties of soil is (Arulanandan, 1987):

- to characterize the soil in a non-destructive manner
- to predict soil parameters ($K_o$, OCR, $\lambda$, $M$) and permit rational analyses of engineering problems

Since electrical properties are due to the interplay of mineralogy and pore fluid composition, they can be related to basic soil behaviour in much the same way as the more common physical indices (LL, PL, w%). To characterize soil behaviour, Arulanandan and his co-workers have defined several electric indices, namely:

$$F = \text{formation factor} = \frac{\sigma_s}{\sigma_m}$$ (A.86)

where:

- $\sigma_s$ = conductivity of the electrolyte
- $\sigma_m$ = conductivity of the soil and electrolyte mixture
- $F$ = a function of porosity, shape and size distribution.

The average formation factor $\bar{F}$ is defined as:

$$\bar{F} = \frac{F_v + 2F_h}{3}$$ (A.87)

where $F_v$, $F_h$ are the $F$ values in the vertical and horizontal directions, respectively. For anisotropic particulate systems, $F_v \neq F_h$. Also, the anisotropy index, $A^2$ and the average shape factor are defined as:
\[ A^2 = \frac{F_v}{F_H} \]  
(A.88)

\[ \bar{f} = -\Delta \log \frac{F}{\Delta \log n} \]  
(A.89)

\( \bar{f} \) is derived from laboratory tests.

The above factors are determined in situ using a conductivity probe which allows \( F_v \) and \( F_H \) to be measured separately. Meegoda and Arulanandan (1986) developed a correlation between an electrical index (\( A^4 \cdot \bar{f} \)) and \( K_o \) for normally consolidated clays. Using soils of varying PI, 10% to 50% clay content with a \( K_o \) range of between 0.4 and 0.8, laboratory measurements were used to develop the relationship shown in Fig. A.38. The application of Fig. A.38 to field situations should be viewed with caution as it is likely to produce \( K_o \) values of the low side. Since the laboratory tests do not account for the effects of fabric and aging, both \( K_o \) and \( A^4 \cdot \bar{f} \) will be different than for the field situation. If, fortuitously, both parameters are affected to the same degree by the lab to field transition, the relationship in Fig. A.38 will be correct. The method applies to normally consolidated clays and is empirically based as no rational interpretation is given. It is based loosely on the \( K_o \)-PI correlation suggested by Alpan (1967).

A.18  PUSH-IN LATERAL STRESS TOOL (LAST)

Similar to the installation of the push-in pressuremeter, the lateral stress tool (LAST) developed at UBC has been designed for the offshore environment where it can be pushed into the soil or allowed to fall and penetrate in the same way as a free fall sampler. The LAST measures the lateral stress at two points in the outside wall of the sampler where a small
circular section has reduced wall thickness (as opposed to the cylindrical section used in the lateral stress cone). At these locations the wall thickness is reduced to 0.5mm and strain gauged at the centre. Just below the stress sensitive area is a porous filter connected to a pore pressure transducer. The LAST is penetrated into the ground and left for a period of
time while measurements of both pore pressure and lateral stress are taken at programmed time intervals. All data are stored downhole and downloaded upon retrieval of the tool.

The equipment was designed at UBC for use in the Offshore Drilling Project (ODP). The equipment was first deployed in early 1990 at several offshore locations. None of the field data are yet available. Prior to further field use the probe requires some modifications in order to provide increased reliability and better temperature characteristics.
APPENDIX B

EVALUATION OF STRESS DISTRIBUTION AROUND FULL DISPLACEMENT PROBES
APPENDIX B
EVALUATION OF STRESS DISTRIBUTION AROUND FULL DISPLACEMENT PROBES

B.1 INTRODUCTION

The installation of full displacement probes in the ground induces stress and/or pore pressure increments to the surrounding soil as a result of the imposed strains. Depending on the nature of the soil, the nature of the increment may be almost wholly in terms of effective stress (drained penetration) or excess pore pressure (undrained penetration). To back-figure the initial stress states from the interpretation of the full displacement measurements, the pre- and post-penetration stresses should be related:

\[ \sigma_{FD} = \sigma_h' + \Delta \sigma + u_o \]  \hspace{1cm} (B.1)

where:

- \( \sigma_{FD} \) = total stress measured by full displacement probe
- \( \sigma_h' \) = initial in situ effective stress
- \( \Delta \sigma \) = stress increment due to penetration
- \( u_o \) = equilibrium pore pressure

For undrained penetration \( \Delta u \) is a function of the normal and shear stress changes that occur around the penetrating probe. The magnitude and variation of the induced stress or pore pressure increments around the probe are important in relation to what is being measured and where is the best place to measure it. Theories to evaluate the measurements obtained during full displacement probe installation comprise the following:
The application of these theories to the interpretation of full displacement testing is considered below. The value of any test of this type obviously depends on the repeatability and reliability of the interpretation method. The stress distribution as a result of probe penetration is complex and consequently initial attempts at interpretation were based on semi-empirical correlations. This procedure is usually reliable on a local basis provided an adequate database exists. Alternately, rational theory (however simplified) can be applied to relate measurements to aspects of soil behaviour. Some degree of semi-empiricism is usually incorporated even here. For most of the available theories, the induced stresses can only be evaluated at the tip of the penetrometer or along the shaft (where the stress is assumed to be independent of distance behind the tip). Furthermore, as demonstrated in the thesis, none of the existing theories adequately describe the degree of unloading effects that occur as the soil passes the geometrical singularity associated with many penetrometer designs.

B.2 THEORIES FOR CYLINDRICAL PENETROMETERS

Two general shapes of probe are in general use for performing in situ tests, namely cylindrical and flat penetrometers. Flat penetrometers are a relatively new addition and consequently much of the early work relates to cylindrical probes. Most of the analysis methods are based on the use of a
stress-based approach. Recent developments consider the strain controlled nature of soil deformation during full displacement probe insertion.

The factors controlling probe penetration in granular soils are numerous and include $\phi$, $D_r$, $\sigma'_h$, compressibility, crushability, grain size distribution, etc. In clays, $E_u$, $S_u$, PI and $S_t$ are primary factors.

For all types of soil other less well defined factors such as, inter alia, microfabric, aging, environmental phenomena and penetration rate may have varying significance depending on local conditions. The inclusion of all these factors into a comprehensive type of analysis is impossible; usually a limited number of the principal parameters are considered. Consequently some degree of empiricism is involved.

B.2.1 Bearing Capacity Methods

The bearing capacity method is based on the plasticity approach of Prandtl (1921) where penetration of a wedge is considered as an incipient failure problem. The approach was initially developed for surface-loaded strip footings on a rigid plastic half space. By making assumptions related to the controlling failure mechanism the approach was extended to the deep penetration problem. The basic correspondence between the probe tip resistance, $q_w$, and soil strength for cohesive soils, $S_u$, is given by:

$$q_w = N_w S_u + \sigma'$$  \hspace{1cm} (B.2)

where $N_w$ is a bearing capacity factor which incorporates factors related to depth of embedment and geometry. The geometrical factor also takes into account the application of plane strain theory to axisymmetric conditions. In the earlier correlations for sands, the penetration resistance was related
to the drained friction angle whereas later work by Durgunoglu and Mitchell (1975) considered additional factors such as $K_o$, cone angle, cone roughness, etc. None of the methods consider the effect of soil compressibility.

All the bearing capacity theories use an empirical shape factor to adapt the plane strain solution to the axisymmetric conditions for a cylindrical probe. The inconsistency in this approach was demonstrated by Kay and Parry (1982) who performed CPT and large diameter plate tests in a well-documented overconsolidated clay. CPT data gave $N_K = 20$ whereas a value of $N_K = 9$ was obtained from plate load test in boreholes, illustrating the difference between the two failure mechanisms.

It is now generally accepted that bearing capacity methods are inadequate for analysis of deep penetration problems due to the incompatible failure mechanisms and boundary conditions. Furthermore, no reliable estimates of failure stress are possible.

B.2.2 Cavity Expansion Methods (CEM)

The solutions for the expansion of spherical or cylindrical cavities in elastic plastic metals were first developed by Bishop et al. (1945), and later applied to pile tip resistances in clays by Gibson (1950). Numerous cavity expansion solutions are available at present, having been developed for:

- evaluating tip resistance of piles/penetrometers
- interpreting the results of pressuremeter tests
- predicting skin friction resistance of piles

Drained and undrained, closed form and analytical solutions are available.
B.2.2.1 Undrained CEM

The first application of CEM was to evaluate bearing capacity factors for deep penetration problems, where it was assumed that probe penetration is equivalent to expanding a spherical cavity in an infinite elastic plastic medium from zero radius to a radius equal to that of the penetrometer/pile. Based on the work of Bishop et al. (1945) the following equations for fully saturated clays for which $\phi = 0$ were derived:

$$P_{sc} = \frac{4}{3} \left[ \ln I_r + 1 \right] S_u + \sigma_v \quad (B.3)$$

$$P_{cc} = \left[ \ln I_r + 1 \right] S_u + \sigma_v \quad (B.4)$$

$$I_r = E_u / 3S_u = G/S_u \quad (B.5)$$

where

- $P_{sc}$ = the limit pressure for a spherical cavity expansion
- $P_{cc}$ = the limit pressure for a cylindrical cavity expansion

Bishop et al. (1945) suggest that $q_c$ lies somewhere between $P_{sc}$ and $P_{cc}$.

The advantage of the CEM is that it is analytical and simple, and thus facilitates solutions based on more realistic and complex soil models. Notwithstanding this, most development in CEM have used simple elastic plastic behaviour, occasionally incorporating strain hardening or strain softening characteristics (Prevost and Hoeg, 1975; Ladanyi, 1963). Furthermore, the form of Eqs. (B.3) and (B.4) vary slightly depending on the assumptions made. Meyerhof (1951), Skempton (1951) and Ladanyi (1963) have
an additional $S_u$ term in the equations to compensate for the effect of cohesion on the vertical normal stress. However, results presented by Levadoux (1980) show that the presence of shaft friction does not significantly alter the stress field around a penetrometer (for $I_r = 50-200$) and thus should not be included in the analysis. The importance of the study by Ladanyi (1963) was that he demonstrated the independence of the strain field around an expanding cavity to soil properties and that it was uniquely determined by geometrical conditions. Triaxial stress-strain curves representing the strain path of the soil during cavity expansion were used to evaluate spherical cavity response. Another variation in the methods relates to the reference stress, $\sigma$, used in the equation for SCE or CCE. The total vertical stress, $\sigma_v$, was used in the early CE formulations, being later replaced by the mean normal stress, $\sigma_m$ (Vesic, 1972). Baligh (1975) developed a CCE relationship referenced to the in situ horizontal stress, $\sigma_h$. The state of stress after expansion of the cavity for initial isotropic stress condition ($\sigma_h = \sigma_v$) is different from that obtained with anisotropic initial conditions ($\sigma_h \neq \sigma_v$). The difference is due to the predicted vertical stress (Levadoux, 1980):

\[
\text{Isotropic} \quad \sigma_z = \frac{\sigma_r + \sigma_\theta}{2} \quad (B.6)
\]

\[
\text{Anisotropic} \quad \sigma_z = \frac{\sigma_r + \sigma_\theta}{2} \quad r_o \leq r \leq R_p \quad (B.7)
\]

\[
\sigma_z = \sigma_v \quad R_p \leq r \quad (B.8)
\]

($r_o = r$ in full displacement problems)
i.e., the stress is discontinuous at the plastic boundary.

Working from cavity expansion theory, Baligh (1975) re-defined the bearing capacity equation as:

\[ q_c + P_{cc} = N_h S_u + \sigma_h \]  \hspace{1cm} (B.9)

where

\[ N_h = (1 + \ln I_r) + 11 \]  \hspace{1cm} (B.10)

which gave a range for \( N_h \) of \( 16 \pm 2 \). Baligh obtained this solution by attempting to model the continuous nature of penetration. The total resistance to penetration is comprised of the work to push a conventional wedge plus the work to maintain the cylindrical cavity open once the wedge had passed (Fig. B.1). This part of the equation considers a hydrostatic stress state.

Various empirical correlations between cavity expansion and excess pore pressure have been obtained using CEM as a theoretical base. Vesic (1972) adopted Henkel’s pore pressure equation and derived an expression for the excess pore pressure at the cavity wall:

\[ \Delta u_{sc} = \left[ \frac{4}{3} \ln I_r + 0.943 a_f \right] S_u \]  \hspace{1cm} (B.11)

\[ \Delta u_{cc} = \left[ \ln I_r + 0.817 a_f \right] S_u \]  \hspace{1cm} (B.12)

where \( a_f \) is Henkel’s pore pressure parameter at failure, which empirically attempts to account for pore pressure generated in shear.
Randolph et al. (1979) developed a similar relationship for the maximum excess pore pressure at the cavity wall:

$$\Delta u = S_u \ln I_r \quad (B.13)$$

This assumes no shear induced pore pressure as the case for an ideal linear elastic perfectly plastic soil.

Adjacent to a penetrometer the soil has experienced large shearing strains and some strength reduction occurs. This is not considered in the simple elastic plastic models. Randolph et al. (1979) attempt to model the strength reduction by defining a minimum strength at the cavity wall with linear increase (with $\ln r$) to a maximum strength at $R_p$. Levaudoux (1980)
following stress-strain curves for plane strain simulation:

• \((\sigma_r - \sigma_\theta) \) vs \(\varepsilon_r\)

• \((\sigma_z - \sigma_{oct}) \) vs \(\varepsilon_r\) where \(\sigma_{oct} = \frac{\sigma_r + \sigma_\theta + \sigma_z}{3}\)

• \(\Delta u_s \) vs \(\varepsilon_r\) where \(\Delta u_s = \Delta u - \Delta \sigma_{oct}\)

Using these curves, the variation in \(\sigma_z\), \(\sigma_\theta\) and \(\Delta u\) can be derived for the cavity expansion case.

The Levadoux (1980) method has been shown to compare well with results obtained by Randolph et al. (1979).

In summary, the total stresses resulting from expansion of a cylindrical cavity in an elastic plastic material with anisotropic initial stresses are presented in Table B.1. The solutions are identical for the case of a spherical cavity, except that the \((1 + \ln I_r)\) term is multiplied by 4/3. The equations for the resulting effective stresses in the plastic region are summarized in Table B.2 (for an isotropic elastic material \(\Delta u\) in the plastic region is zero). The relevant soil parameters are obtained from CIU tests with pore pressure measurement where \(E = E_{s0}\) (secant). The associated total and effective stress paths are indicated in Fig. B.2.

For strain softening soils, a post-peak strength reduction occurs which affects the total stress path during expansion of the cavity (Fig. B.3). The undrained expansion of the cavity produces a radial strain equal to:

\[
\varepsilon_r = \frac{r_0^2 - r_1^2}{2r_1^2}
\]  

(B.14)
Table B.1
Total Stresses Due to Undrained Expansion of a Cylindrical Cavity

\[
\frac{R_p}{\sqrt{r_0^2 - r_1^2}} = \sqrt{\frac{G}{S_u}}
\]

**Total stresses in elastic region** \( r > R_p \)

\[
\begin{align*}
\sigma_r &= + S_u \left( \frac{R_p}{r} \right)^2 + \sigma_{ho} \\
\sigma_\theta &= - S_u \left( \frac{R_p}{r} \right)^2 + \sigma_{ho} \\
\sigma_z &= \sigma_{vo}
\end{align*}
\]

**Total stresses in plastic region** \( r_0 < r < R_p \)

\[
\begin{align*}
\sigma_r &= S_u \left( 1 + \ln \frac{R_p}{r} \right) + \sigma_{ho} \\
\sigma_\theta &= S_u \left( -1 + \ln \frac{R_p}{r} \right) + \sigma_{ho} \\
\sigma_z &= S_u \ln \frac{R_p}{r} + \sigma_{ho}
\end{align*}
\]
Table B.2

Effective Stresses Due to Undrained Cylindrical Cavity Expansion

<table>
<thead>
<tr>
<th>( R &gt; r &gt; r_o )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{r_p}{r} = \left( \frac{C}{S_u} \right)^{1/2} )</td>
</tr>
<tr>
<td>( \Delta u = K_o \bar{\sigma}_{vo} + 2S_u \ln \frac{R}{r} - \frac{S_u}{\sin \phi} )</td>
</tr>
<tr>
<td>( \bar{\sigma}_z = \frac{S_u}{\sin \phi} )</td>
</tr>
<tr>
<td>( \bar{\sigma}_r = S_u \left( \frac{1}{\sin \phi} + 1 \right) )</td>
</tr>
<tr>
<td>( \bar{\sigma}_\theta = S_u \left( \frac{1}{\sin \phi} - 1 \right) )</td>
</tr>
</tbody>
</table>

(which considers conservation of volume). The post-peak behaviour is assumed as (Bouckovalas and Marr, 1981):

\[
\frac{\sigma_r - \sigma_\theta}{2} = S = A - B \ln |e_r| \tag{B.15}
\]

Equation (B.14) into Eq. (B.15) gives:

\[
S = C + D \ln \frac{r}{(r_o^2 - r_1^2)^{1/2}} \tag{B.16}
\]
\[ \sin \alpha = \tan \phi \]
\[ \phi = \text{friction angle of the soil} \]

For the strength variation given earlier, i.e.:

\[ S = S_{up} \text{ at } r = R_p \] (B.17)

\[ S = S_{ur} \text{ at } r = r_o \] (B.18)

we have for \( r_o \leq r \leq R_p \):

\[ S_{ur} = S_{up} \left[ \frac{1}{S_t} + (1 - \frac{1}{S_t}) \frac{\ln r/r_o}{\ln R_p/r_o} \right] \] (B.19)
Fig. B.3 Effect of strain softening on effective stress path during cavity expansion (after Bouckovalas and Marr, 1981).
where

\[ S_t = S_{up}/S_{ur} \quad (B.20) \]

Using the radial equilibrium condition we get (for plastic zone):

\[ \sigma_r = K_o \sigma_v' + u_o + S_{up} \left[ 1 - \frac{2}{S_t} \ln \frac{r}{R_p} - \frac{1}{\ln r/o} \left( 1 - \frac{1}{S_t} \right) \ln \frac{r}{R_p} \ln \frac{R_p}{r_0} \right] \quad (B.21) \]

\[ \sigma_\theta = \sigma_r - 2S_{ur} \quad (B.22) \]

\[ \sigma_z = \sigma_r - S_{ur} \quad (B.23) \]

\[ R_p/r_0 = (G/S_{up})^{0.5} \quad (B.24) \]

\[ \Delta u = \sigma_r = \left( 1 + \frac{1}{\sin \phi} \right) S_{ur} \quad (B.25) \]

Comparison of this simplified method with the CAM Clay method of Randolph et al. (1979) showed that the stresses were uniformly underpredicted by 0.6 \( S_u \) in the plastic zone. Also the predicted pore pressures are 0.9 \( S_u \) smaller resulting in an overestimate of effective stress of 0.3 \( S_u \). This may arise due to the unconservative nature of CAM clay predictions for plane strain soil behaviour (Bouckovalas and Marr, 1981).

Konrad and Law (1987) consider the spherical expansion problem and relate the limit pressure to the normal total stress acting on the cone face (Fig. B.4). A reduced undrained shear strength is employed in the analysis,
the reduction being dependent on the generated excess pore pressure. The 
procedure was used to calculate $S_u$ from $q_C$ and good agreement with the Vesic 
model was obtained.

In general, undrained cavity expansion methods have met with limited 
success. Problems exist over which soil parameters should be used in the 
analysis. Considering the fact that CEM provide a 1-D solution to an axi-
symmetric problem (which represents an oversimplification of soil behaviour 
since the strain paths are different), parameter selection may be of second-
ary importance. No account is taken of the shape of the penetrometer, or the 
shear stresses as the soil moves up the shaft. However, the cavity expansion 
theories do model aspects of behaviour measured in the field, namely the 
unloading of the soil as it passes the tip and provides reasonable estimates.

Fig. B.4 Stress field on cone and spherical cavity expansion for inter-
pretation of CPTU data (after Konrad and Law, 1987).
of excess pore pressures generated during penetration. Displacement patterns measured during probe installation show very little vertical movement once the tip has passed; hence plane strain conditions exist which can be represented by the expansion of a cylindrical cavity.

B.2.2.2 Drained CEM

For expansion of a cavity in sand, full drainage is considered ($\Delta u = 0$) and the analysis is performed in terms of effective stresses. As for clay, the plane strain assumption is acceptable provided the induced shear stresses are of the same relative importance. In addition, the volume change that occurs during penetration must be considered when computing the radius of the plastic zone, $R_p$. The Mohr-Coulomb criterion of failure is usually employed (Vesic, 1972):

$$\frac{\sigma_{\theta} + c'\cot\phi}{\sigma_{r} + c'\cot\phi} = \frac{1 - \sin\phi'}{1 + \sin\phi'}$$  \hspace{1cm} (B.26)

The volume change condition is given as:

$$r_o^2 - r_i^2 = R_o^2 - (R - \zeta)^2 + \frac{(R^2 - r_i^2)\Delta}{p_o}$$  \hspace{1cm} (B.27)

where

$\Delta$ = average volumetric strain in the plastic zone

$\zeta$ = outward radial movement of elastic plastic boundary

Baligh (1975) modified Vesic's solution to consider a nonlinear strength envelope, $\phi = f(\sigma')$: 
\[ \tan \phi_t = \tan \phi_o - \tan \alpha \log \left( \frac{\sigma'_o}{\sigma'_o} \right) \]  \hspace{1cm} (B.28) 

where

- \( \sigma_o \) = unit stress
- \( \phi_o \) = friction angle at \( \sigma'/\sigma_o = 2.72 \)
- \( \phi_t \) = tangent friction angle
- \( \alpha \) = constant angle defining curvature of envelope

A 10% difference in expansion pressures between the Baligh and Vesic methods was obtained.

The Vesic (1972) solution considers the effects of volume change in the plastic region or alternatively, excess pore pressures can be evaluated if the process is undrained (as discussed earlier).

For the Vesic (1972) solution, putting initial cavity radius equal to zero for the full displacement condition we obtain the following expression for the ultimate cylindrical cavity expansion (limit) pressure, \( P_{cc} \):

\[ P_{cc} = \sigma'_h \left( \frac{2 \sin \phi'}{1 + \sin \phi'} \right) \frac{R}{r_o} + c \cot \phi' \left[ \frac{2 \sin \phi'}{1 + \sin \phi'} \right] \left( \frac{R}{r_o} \right) - 1 \]  \hspace{1cm} (B.29) 

where:

\[ \frac{R}{r_o} = \left( \frac{I_{rr}}{\cos \phi} \right)^{1/2} \]  \hspace{1cm} (B.30) 

\[ I_{rr} = \frac{I_r}{1 + I_r \Delta \sec \phi} \]  \hspace{1cm} (B.31)
To reduce the problem of the complex equations, Vesic presented charts and tables for simplifying the expressions. Hence, Eqs. (B.31) and (B.29) can be written as:

\[
I_r = \frac{E}{(1+\nu)(c'+\sigma'\tan\phi')} = \frac{G}{c'+\sigma'\tan\phi'} \tag{B.32}
\]

The value of \(\zeta\) (as a function of \(I_r\), \(\phi\) and \(\Delta\)) can be obtained from tables (Vesic, 1972). Values of \(F_c\) and \(F_q\) are also given. Vesic also provides a similar table and figure for the spherical factors. The main problem associated with the application of the Vesic method is determining the reduced rigidity index, \(I_{rr}\), due to the uncertainty in \(\Delta\). Vesic (1972) presented tables of \(\Delta\) for \(I_r\) and \(\phi\) values where it was apparent that the effect of volume change in the plastic zone is most pronounced in relatively incompressible soils, i.e. for \(I_r = 250\) (stiff clay, dense sand) a 1% volumetric strain in the plastic zone reduces \(I_{rr}\) by a factor of 4. In this case, for \(\phi = 30^\circ\), \(P_{cc}\) is halved. Alternately, using triaxial data, the volume change factors can be derived using stress paths similar to that involved in the cavity expansion process. The evaluation of the most appropriate \(\Delta\) value is iterative in nature. Vesic (1975) later suggests the following relationship:

\[
I_{rr} = 3/f_s(CPT) \tag{B.35}
\]

but little or no experience exists to evaluate this.
Carter et al. (1986) present an analysis of cavity expansion in cohesive frictional soils which considers the presence of elastic strains in the plastic region. Comparison of their result with that of Hughes et al. (1977), which does not consider elastic strains in the plastic region, indicates that the difference between the two methods is small at small strain levels (<10%) and when \( G/\sigma'_h \) is large. At larger deformations, the elastic strains in the elastic zone are appreciable.

Carter et al. (1986) present the following large strain closed form solution for which initial conditions can correspond to the full displacement situation:

\[
\frac{2G}{\sigma'_h} = \frac{N-1}{N+k} \left[ T \left( \frac{P_L}{\sigma'_R} \right) \gamma - Z \frac{P_L}{\sigma'_R} \right]
\]

where:

\[
T = (k+1)(1 + \frac{k\chi}{\alpha + \beta})
\]

\[
Z = (k+1) \frac{k\chi}{\alpha + \beta}
\]

\[
\sigma'_R = \frac{1+k}{N+k} N \sigma'_h
\]

\[
\alpha = k/M
\]

\[
\beta = 1 - k\left( \frac{N-1}{N} \right)
\]
\[ \gamma = \frac{1+\alpha}{1-\beta} \] (B.42)

\[ M = \frac{1+\sin\psi}{1-\sin\psi} \] (B.43)

\[ N = \frac{1+\sin\phi}{1-\sin\phi} \] (B.44)

\[ \chi = \frac{k(1-v)\kappa v(M+N) + [(k-2)v+1]MN}{[(k-1)v+1]MN} \] (B.45)

where

\( k = 1 \) (cylindrical cavity)

\( k = 2 \) (spherical cavity)

\( \sigma_R \) = radial stress at the elastic plastic boundary

\( \psi \) = dilatancy angle

The method would suggest that the lift-off pressure from a full displacement probe should equal the limit pressures depending on the effect of the unloading as the soil passes the tip. This point is discussed further in the thesis.

Greenuw et al. (1988) use the spherical form of the Carter et al. (1986) formulation and compare it with cone bearing determined in the calibration chamber. The \( q_c \) data is not corrected for chamber effects and Greenuw et al. (1988) show that:

\[ F = \frac{q_c}{P_L} = 3.6 \pm 0.5 \] (B.46)

This suggests that the spherical cavity expansion formulation of Carter et al. (1986) may provide a reasonable model for cone penetration. It would
appear that some adjustment to be calculated values is however required.

Apart from the two methods presented above, no other solutions were found which enable the stresses on full displacement probes to be evaluated. Similarly, in a review of stresses around driven piles, Jardine and Potts (1988) conclude that only empirical correlations exist to evaluate the state of stress around driven pile in sands. These correlations however assume that the radial stress does not vary during loading after initial driving. Usually the value of $K_s$, the average coefficient of earth pressure on the pile shaft, is interpreted from skin friction measurements where:

$$f_s = K_s (\sigma'_v)_{av} \tan \delta$$  \hspace{1cm} (B.47)

where

$$\sigma'_v \text{_{av}} = \text{average overburden stress along length of pile}$$
$$\delta = \text{angle of skin friction}$$

Load tests on instrumented piles have shown that $K$ also varies along the pile length, being a minimum just behind the toe, increasing to maximum at some distance below the ground surface. The few data available for normally consolidated sand show that $K_s$ also varies due to the method of installation (Fig. B.5).

B.2.3 Finite Element Methods

The use of the finite element method to predict stresses induced by full displacement probes models the continuous steady state penetration as a progressive incremental occurrence. Large displacements and strains are
permitted using very complicated soil behaviour models. Rather than the more common stress boundary, velocity boundary conditions are specified.

De Borst and Vermeer (1984) analyzed the condition of a penetrometer placed in a prebored hole using incremental displacement controlled FE analysis. The incremental displacements were performed until a limit pressure was reached. Only the stresses around the tip were considered.

Carter et al. (1979) performed an FE analysis for clay using a CAM clay model. Reasonable agreement was obtained with the cavity expansion model of Randolph and Wroth (1970).
Houlsby and Teh (1988) compared the results of FE analysis similar to that described above with the results from the strain path method. The modelling of the incremental approach of a cone in a prebored hole gives much lower stresses on the cone (about 30%) than those obtained using the strain path approach. The error arises from the different in situ lateral stresses used in the two analyses. The incremental FE analyses use lateral stresses equal to the existing in situ stress state whereas the strain path method utilizes the higher stresses induced during penetration.

B.2.4 Strain Path Analysis

Based on a review of experimental observations of deep penetration problems Baligh (1985) suggested that this behaviour is essentially strain controlled and that the associated deformations are not sensitive to material characteristics. Furthermore, since the conditions are heavily constrained kinematically soil deformations can be estimated with a reasonable degree of accuracy. Hence, by changing the reference point, the penetration of a probe in a homogeneous material could be modelled as the steady state flow of soil past the penetrometer, the streamlines being obtained by inviscid flow theory. This forms the basis of the strain path method. Several types of strain path analysis techniques have been presented:

- The original approach presented by Levardoux and Baligh (1980) whereby the strain field is estimated using the method of sources and sinks.
- The semi-analytical method using conformal mapping proposed by Tumay et al. (1985), also based on flow of an inviscid material for axisymmetric conditions.
- Teh (1987) also developed a similar technique to that of Levardoux and Baligh (1980) but corrected the strain path method solution to give stress equilibrium.
• Huang (1989) extended the above axisymmetric solutions to the 3D case and adapted a numerical technique known to aeronautical engineers as the panel method to the analysis of bodies of arbitrary shape.

Levadoux and Baligh (1980) use a smooth rounded tip as a simple pile model whereas the other methods consider actual probe geometry.

The general result of the first three methods are considered here. The method proposed by Huang (1989) is discussed in section B.3.

The strain path method (SPM) applied to homogenous incompressible inviscid saturated clay showing elastic perfectly plastic behaviour has shown that penetration causes large strains in the soil, involves significant strain reversals and rotations of principal strain directions. Axial and radial tensile strains exist close to the cone during penetration, the distribution and magnitude of which depend on the angle of the cone. Very large strains and strain rates occur for soil within an area of 10 to 15 times the radius of the cone. Higher strain rates occur for larger cone angles. The flow line solutions do not satisfy stress equilibrium as they are an approximation, which is pronounced at the cone shoulder. Various approximate stress solutions were tried by Teh (1987) to correct for this inequilibrium. In an approximate manner this was solved by using the results of the velocity field in an incremental finite element formulation. The uncertainties in the stresses at the cone shoulder were reduced but not completely eliminated. The results of the modelling are highly dependent on the soil properties used, i.e. hyperbolic stress-strain behaviour gives more realistic results than elastic-plastic model. Shear stress levels in the soil depend only on the \( S_u \) value employed; in practice, the stresses will be reduced due to strain softening, anisotropy and rate behaviour. Close to the
probe the excess pore pressures are principally due to octahedral stress changes; shear induced pore pressures are secondary.

\[ \Delta \sigma_{\text{oct}} = 0.63 \Delta u (\text{bilinear}) - 0.75 \Delta u (\text{hyperbolic}) \]  

(B.48)

Furthermore, the octahedral stress changes, which are proportional to \( S_u \) and \( G \) also control \( q_t \). The yield strain is also an important controlling parameter on \( q_t \) and \( \Delta u \) and it controls the development of the plastic zone in front of the penetrometer tip.

Figure B.6 shows the strain contours obtained for the analysis of a penetrating 60° cone. Far behind the tip (approximately 3D) the strain contours are parallel to the penetrometer axis. The variations of \( e_r \) (Fig. B.6a) and \( e_\theta \) (Fig. B.6b) are comparable to solutions based on cylindrical cavity expansion theory. The singularity of the cone shoulder is evident in Fig. B.6c. Below the shoulder, all vertical strains are compressive while above this level they are all tensile. Stress contours around the cone also suggest the applicability of CEM at large distances behind the tip. However, due to the fact that CEM's neglect the effects of strain history they will usually overpredict both stresses and pore pressures near the shaft of a penetrometer (Baligh, 1986). This is illustrated in Fig. B.7. Examination of the stresses near the tip would suggest poor predictive capacity of CEM.

B.3 THEORIES FOR FLAT PROBES

Interpretation of stresses around flat penetrometers has been performed almost exclusively based on empirical relationships. The development of a 3D
Fig. B.6 Strain contours around a 60° cone penetrometer (after Teh, 1987).
method for flow of a fluid around an arbitrary body was the first attempt to apply theoretical principles to the problem. As mentioned above, both bearing capacity and cavity expansion methods can be used incorporating geometry factors to account for shape differences.

The panel method was applied to the penetration problem by Huang (1989). The technique is a numerical method for obtaining an approximate solution to
the governing equation for the steady flow of an ideal fluid about a 3D body. The body surface is represented by a number of plane quadrilateral elements (panels). An example of the distribution of panels on a cone and dilatometer (DMT) tip is shown in Fig. B.8.

Fig. B.8 Panel representation of cone and dilatometer tips (after Huang, 1989).

Comparison of results obtained for the cone and dilatometer suggest that (Huang, 1989):

- For the cone, the component of strain associated with cavity expansion was significantly larger than other components justifying CEM for interpretation of CPT.
• No predominant shearing mode existed for the DMT although the 3D analysis of the plate penetration gave a higher cavity strain component than did the 2D analysis.

• Comparison of 2D and 3D analyses for the DMT show that the effects of the finite length of the blade are important. Displacements and strain fields at the edge of the blade are complicated and dissipate more rapidly than for the 2D case.

• The difference between the strain fields induced by CPT and DMT are different and are not solely related to the difference in cone angles. 3D modelling of plate penetration is thus necessary to take into account the finite width of the blade.

• The stresses behind the cone tip are expected to be higher than those behind the DMT due to the larger induced disturbance.

B.4 COMPARISON WITH FIELD MEASUREMENTS

Few comparisons exist between predicted and measured stresses, those available generally involve cavity expansion predictions of excess pore pressure in clays. For $I_r$ varying between 100 and 250, theoretical predictions of normalized excess pore pressure, $\Delta u/S_u$, range from 6.1 to 7.4 for the spherical case and from 4.6 to 5.5 for the cylindrical case. Higher values, depending on $\alpha_\ell$, are obtained if the semi-empirical formulae of Vesic (1972) are used as they consider also a component of shear induced pore pressure. The normalized pore pressure is given by

$$\Delta u / S_u = N \Delta u$$  \hspace{1cm} \text{or}  \hspace{1cm} \Delta u = N \Delta u \cdot S_u$$

(B.49)
Results presented by Baligh et al. (1981), Battaglio et al. (1981), Lacasse and Lunne (1982), Roy et al. (1982) and Campanella, Sully and Robertson (1988) for NC to lightly OC clays suggest reasonable agreement between predicted and measured pore pressures, especially for locations behind the tip. In some cases, better agreement is obtained using the empirical approach of Vesic. At higher OCR, the data presented by Coop and Wroth (1989) suggest that CEM considerably overpredicts $\Delta U$. The rationale for selecting representative $G$ and $S_u$ values for each site is however problematical.
APPENDIX C

REFERENCES
REFERENCES

Abbreviations used for reference list:

- **STP1014** Vane Shear Testing in Soils: Field and Laboratory Studies, American Society for Testing and Materials, Philadelphia.
- **CPT-UK88** Penetration Testing in the U.K., Birmingham.
- **CGC** Canadian Geotechnical Conference.
- **ECSMFE** European Conference on Soil Mechanics and Foundation Engineering.
- **ESOPT-1** First European Symposium on Penetration Testing
- **ESOPT-2** Second European Symposium on Penetration Testing.
- **ICOLD** International Congress on Large Dams
- **ICRAGEEESD** International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis.
- **ICSMFE** International Conference on Soil Mechanics and Foundation Engineering.
- **IN SITU '86** Use of In Situ Tests in Geotechnical Engineering, American Society of Civil Engineers, Special Publication No. 6.
- **ISOPT-1** First International Symposium on Penetration Testing, Orlando.
- **ISMOSP** In Situ Measurement of Soil Properties, American Society of Civil Engineers, Speciality Conference, Raleigh, N.C.
- **ISIST** International Symposium on In Situ Testing, Paris
- **ISP3** Third International Symposium on Pressuremeter, British Geotechnical Society, Oxford, UK.
- **JGE,ASCE** Journal of Geotechnical Engineering, American Society of Civil Engineers.
- **JGED,ASCE** Journal of the Geotechnical Engineering Division, American Society of Civil Engineers.
- **JSMFD,ASCE** Journal of the Soil Mechanics and Foundation Division, American Society of Civil Engineers.
- **SCPAE,ASCE** Symposium on Cone Penetration and Experience, Geotechnical Engineering Division, ASCE.


Gillespie, D.G. (1990) Evaluating shear wave velocity and


Schmertmann, J.H. (1983) Revised procedure for calculating Ko and OCR from DMT's with Id>1.2 and which incorporates the penetration force measurement to permit calculating the plane strain friction angle. DMT Workshop, 16-18 Mar., Gainesville, Fla.


Vaughan, P.R. (1972) Oral discussion on HFT. Proc. 5th ECSMFE, Madrid, 2:72-75.


