INVESTIGATION AND INTERPRETATION OF CONE PENETRATION RATE EFFECTS

by

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DECLARATION

I hereby declare that, except where specific reference is made to the works of others, the contents of this dissertation are original and have never been submitted in whole or in part for consideration for any other degree of qualification at this, or any other, university.

Yusuke SUZUKI

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ABSTRACT

The Cone Penetration Test (CPT) is the most popular in-situ test for onshore and offshore site investigation. The standard penetration rate employed of 20±5 mm/s is well embedded in practice since cone penetration is expected to be fully drained in sand and fully undrained in clay at this velocity. Interpretation of geotechnical parameters from the CPT methods for sand and clay has a relatively high reliability. However this reliability is poor in intermediate soils, such as silts, clayey and sandy silts and residual soils, as penetration takes place under partially drained conditions in these materials. Very little guidance is available to practitioners for derivation of geotechnical parameters during partially drained penetration. This thesis addresses the need for such guidance by examining the rate dependency of penetrometer resistance and consequently also provides information to assist estimation of the penetration resistance and rate dependency of penetrometers with different diameters, such as piles.

The Thesis examines the rate dependency of penetrometer resistance using (i) a comprehensive series of penetrometer testing in the field and laboratory and (ii) Finite Element (FE) analyses that simulate penetrometer penetration at various velocities in a range of soils modelled using a non-linear elasto-plastic constitutive model. Interpretation of the experimental penetrometer tests is supported by a series of laboratory element tests on samples of the in-situ soils. The observations made from the experimental and numerical components of this research are combined with the existing database of measurements obtained in previous studies to formulate conclusions.

An extensive series of piezocone penetration tests was performed at three sites in Western Australia, to extend the current sparse database of variable rate piezocone tests in the field. Undrained, partially drained and fully drained conditions were observed by performing tests at rates down to 100,000 times slower than the standard rate; variable rate T-bar and ball penetration tests were conducted for comparative purposes. The effect of fines content on CPT end resistance was also examined in a series of variable rate penetration tests on kaolin-sand mixture performed in both the centrifuge and in pressurised chambers at 1g. Laboratory tests on intact and reconstituted samples assisted interpretation of the field and laboratory scale penetrometer tests.

A series of parametric numerical analyses was carried out employing the spherical cavity expansion method using the FE code PLAXIS. A coupled consolidation calculation with a non-linear elasto-plastic soil model was used to simulate variable rate cone penetration. These analyses allowed evaluation of the specific effects on penetration resistance of soil stiffness, friction angle, in-situ lateral effective stress, permeability, cone diameter and stress level. Use of a normalisation velocity term incorporating drainage length, horizontal permeability and stiffness are found to be effective to define drainage transitions.

The numerical and experimental studies show the velocity range over which conditions can be expected to be partially drained for a standard CPT; this range is defined in terms of a normalised velocity. These studies also show that ratio between drained and undrained penetration resistance is strongly dependent on the soil stiffness and friction angle and hence varies widely with soil type; typical responses for a range of common soil types are identified. Practical recommendations when conducting CPTs in intermediate soils are provided to allow improved derivation of geotechnical parameters.
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SYMBOLS AND ABBREVIATIONS

Roman

\(a\)     curve-fitting parameter; cavity radius
\(a_0\)   initial cavity radius
\(a_f\)   final cavity radius
\(A_p\)   projected area of the penetrometer in a plane normal to the shaft
\(A_s\)   cross-sectional area of the connection shaft in a plane normal to the shaft
\(b\)     curve-fitting parameter
\(B_q\)   pore pressure ratio
\(c\)     curve-fitting parameter
\(c^'\)   effective cohesion intercept
\(c_h\)   horizontal coefficient of consolidation
\(c_v\)   vertical coefficient of consolidation
\(c_vNC\) vertical coefficient of consolidation at normally consolidated condition
\(C_c\)   compression index
\(C_{c\ max}\) maximum compression index
\(C_{c\ s}\) intrinsic compression index
\(C_r\)   recompression index
\(d\)     diameter of penetrometer; drainage length; curve-fitting parameter
\(d_{CE}\) final cavity diameter
\(d_e\)   equivalent area diameter of penetrometer
\(d_o\)   outside diameter of tube sampler
\(D\)     distance from the cone centre to the inner wall of the container
\(D'\)    one dimensional constrained modulus
\(D_t\)   relative density
\(e\)     void ratio
\(e_0\)   initial void ratio (=e\(_{\text{int}}\))
\(e_g\)   intergranular void ratio
\(e_{L}\) void ratio at the liquid limit
\(e_{\text{max}}\) maximum void ratio
\(e_{\text{min}}\) minimum void ratio
\(e_{sk}\) skeleton void ratio
\(E\)     Young’s modulus
\(E_{50\ ref}\) Young’s modulus in triaxial loading at p\(_{\text{ref}}\)
\(E_{oed\ ref}\) Young’s modulus in oedometer loading at p\(_{\text{ref}}\)
\(E_{ur\ ref}\) Young’s modulus in triaxial unloading/reloading at p\(_{\text{ref}}\)
\(f_s\)    sleeve friction
Symbols and Abbreviations

\( F \)  plastic zone shape factor
\( F_f \)  friction ratio
\( g \)  normal earth gravity
\( G \)  shear modulus
\( G_0 \)  shear modulus at very small strain
\( G_{S0} \)  secant shear modulus at 50% of failure shear stress
\( G_s \)  specific gravity
\( G_{sC} \)  specific gravity of clays
\( G_{sS} \)  specific gravity of sands
\( h \)  height of the cone tip
\( H \)  sample height
\( I_c \)  soil behaviour type index
\( I_p \)  plasticity index
\( I_r \)  rigidity index
\( k \)  permeability
\( \kappa^* \)  permeability (assumed)
\( k_h \)  horizontal permeability
\( k_{h0} \)  in situ horizontal permeability
\( k_v \)  vertical permeability
\( K \)  stress ratio
\( K_0 \)  in situ stress ratio
\( K_{0NC} \)  \( K_0 \) for normally consolidated condition
\( m \)  rate exponent; power for stress level dependency of stiffness
\( n \)  stress normalisation exponent
\( N \)  resistance factor
\( N_{ball} \)  resistance factor for ball penetrometer
\( N_{cT} \)  cone resistance factor
\( N_{T-bar} \)  resistance factor for T-bar penetrometer
\( N_g \)  \( N \) times of gravitational acceleration
\( p' \)  mean effective stress
\( p_0 \)  initial mean total stress
\( p_0' \)  initial mean effective stress
\( p'_{f} \)  mean effective stress at failure (CAU triaxial)
\( p_a \)  atmospheric pressure
\( P_{\text{limit}} \)  cavity expansion limit pressure
\( p_{\text{ref}} \)  reference stress for stiffness
\( q \)  penetration resistance; deviator stress
\( q_p \)  pile end resistance
\( q_{ball} \)  ball resistance
\( q_c \)  cone resistance
\( q_{cD} \)  drained cone resistance
\( q_{cUD} \)  undrained cone resistance
Symbols and Abbreviations

$q_f$ ultimate deviator stress at failure
$q_{in}$ penetration resistance (full-flow penetrometers)
$q_m$ measured penetration tip resistance
$q_{net}$ net cone resistance ($=q_{cnet}$)
$q_{out}$ extraction resistance (full-flow penetrometers)
$q_t$ cone resistance corrected for pore pressure effects
$q_{T-bar}$ T-bar resistance
$q_{ref}$ reference net cone resistance
$Q$ normalised cone resistance ($Q_t$ and $Q_{t1}$)
$Q_D$ normalised drained cone resistance
$Q_{tn}$ normalised cone resistance with the varying stress exponent
$Q_{UD}$ normalised undrained cone resistance
$r$ radius of cone penetrometer; radius to the soil model (centrifuge)
$r_{CE}$ drainage distance
$\Delta r_{CE}$ radial displacement at the cavity wall
$R$ radius of soil domain; isotropic overconsolidation ratio
$R_e$ effective centrifuge radius for a reference for $N$ acceleration
$R_f$ failure ratio
$S_t$ sensitivity
$s_u$ undrained shear strength
$s_{u*}$ undrained shear strength corrected on strain rate
$s_{uss}$ undrained shear strength (SS)
$t$ thickness of tube sampler; time
$t_{50}$ time for 50% excess pore pressure dissipation
$t_{90}$ time for 90% degree of consolidation
$\Delta t_{CE}$ consolidation time for the cavity to be expanded
$T^*$ the normalised time factor
$T_{90}$ time factor for 90% degree of consolidation
$u$ pore pressure
$u_0$ hydrostatic pore pressure
$u_1$ pore pressure measured on the cone face
$u_2$ pore pressure measured just behind the cone
$u_b$ excess pore pressure measured at the sample base (CRSC)
$\Delta u$ excess pore pressure
$\Delta u_a$ excess pore pressure at the cavity wall
$\Delta u_f$ excess pore pressure at failure (CAU triaxial)
$\Delta u_{pred}$ predicted initial excess pore pressure
$|U|$ radial displacement at the cavity wall
$v$ penetration velocity
$v_{r,CE}$ radial cavity expansion velocity
$V$ normalised velocity
$V_0$ normalised velocity where the viscous effects start to decay
### Symbols and Abbreviations

- $V_{\text{clay}}$: volume of clay fractions
- $V_{h}$: normalised velocity using $c_h$
- $V_{h,\text{CE}}$: normalised cavity expansion in CE
- $V_{\text{ref}}$: reference normalised velocity
- $V_{s}$: shear wave velocity
- $V_{\text{sand}}$: volume of sands
- $V_{v}$: normalised velocity using $c_v$
- $V_{\text{void}}$: volume of voids
- $w_c$: water content
- $w_i$: initial water content
- $w_L$: water content at liquid limit

### Greek

- $\alpha$: unequal area ratio
- $\alpha_c$: cone apex angle
- $\alpha_{\text{int}}$: interface friction ratio between cone and soil; cone roughness
- $\delta$: interface friction angle between cone and soil
- $\Delta$: initial stress anisotropy
- $\varepsilon_a$: axial strain
- $\varepsilon_{\text{af}}$: axial strain at failure (CAU triaxial)
- $\dot{\varepsilon}_a$: axial strain rate
- $\phi'$: friction angle
- $\phi_{cv}$: constant volume friction angle
- $\phi_{\text{fa}}$: mobilised friction angle
- $\phi_p$: peak friction angle
- $\gamma$: shear strain
- $\gamma'$: effective unit weight
- $\gamma_{\text{sat}}$: saturated unit weight
- $\gamma_w$: unit weight of the water
- $\kappa$: recompression index (~$C'/2.3$)
- $\lambda$: compression index (~$C'/2.3$)
- $\Lambda$: plastic volumetric strain ratio
- $\nu'$: effective Poisson’s ratio
- $\nu_{\text{fr}}$: Poisson’s ratio for unloading/reloading
- $\mu$: rate coefficient
- $M$: slope of critical state line
- $\rho$: density
- $\rho_{\text{d,ini}}$: initial dry density
- $\sigma_1$: major principal total stress
## Symbols and Abbreviations

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tr>
<td>$\sigma_1'$</td>
<td>major principal effective stress</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>cell pressure; confining pressure</td>
</tr>
<tr>
<td>$\sigma_3'$</td>
<td>minor principal effective stress</td>
</tr>
<tr>
<td>$\sigma_a$</td>
<td>cavity expansion pressure at the cavity wall</td>
</tr>
<tr>
<td>$\sigma_h'$</td>
<td>effective horizontal stress</td>
</tr>
<tr>
<td>$\sigma_{h0}$</td>
<td>effective horizontal overburden stress</td>
</tr>
<tr>
<td>$\sigma_n'$</td>
<td>effective normal stress</td>
</tr>
<tr>
<td>$\sigma_n$</td>
<td>total normal stress</td>
</tr>
<tr>
<td>$\sigma_r'$</td>
<td>effective radial stress in cylindrical cavity expansion</td>
</tr>
<tr>
<td>$\sigma_v'$</td>
<td>effective vertical stress</td>
</tr>
<tr>
<td>$\sigma_{v0}$</td>
<td>effective vertical overburden stress</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>total vertical stress</td>
</tr>
<tr>
<td>$\sigma_{v0}$</td>
<td>total vertical overburden stress</td>
</tr>
<tr>
<td>$\sigma_{vc}$</td>
<td>effective vertical stress after consolidation</td>
</tr>
<tr>
<td>$\sigma_{vy}$</td>
<td>effective vertical yield stress</td>
</tr>
<tr>
<td>$\tau$</td>
<td>shear stress</td>
</tr>
<tr>
<td>$\tau_{\text{fail}}$</td>
<td>shear stress acting on a diagonal failure plane (SS)</td>
</tr>
<tr>
<td>$\tau_{xy}$</td>
<td>shear stress applied on the horizontal plane (SS)</td>
</tr>
<tr>
<td>$\omega$</td>
<td>angular rotational speed of the centrifuge</td>
</tr>
<tr>
<td>$\psi$</td>
<td>peak dilation angle</td>
</tr>
<tr>
<td>$\psi_0$</td>
<td>state parameter</td>
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<tr>
<td>$\psi_m$</td>
<td>mobilised dilation angle</td>
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### Subscript/Superscript

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<tbody>
<tr>
<td>D</td>
<td>Drained</td>
</tr>
<tr>
<td>PD</td>
<td>Partially Drained</td>
</tr>
<tr>
<td>UD</td>
<td>Undrained</td>
</tr>
<tr>
<td>OC</td>
<td>Over Consolidated</td>
</tr>
<tr>
<td>NC</td>
<td>Normally Consolidated</td>
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<tr>
<td>ref</td>
<td>reference value</td>
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### Abbreviation

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>CAU</td>
<td>Consolidated (Anisotropically) Undrained</td>
</tr>
<tr>
<td>CE</td>
<td>Cavity Expansion method</td>
</tr>
<tr>
<td>CF</td>
<td>Clay Fraction</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>CPTU</td>
<td>CPT with pore pressure measurement - piezocone test</td>
</tr>
<tr>
<td>CRSC</td>
<td>Constant Rate of Strain Consolidation test</td>
</tr>
<tr>
<td>FC</td>
<td>Fines Content</td>
</tr>
<tr>
<td>FE</td>
<td>Finite Element</td>
</tr>
<tr>
<td>HS</td>
<td>the Hardening Soil model</td>
</tr>
<tr>
<td>ICL</td>
<td>Intrinsic Compression Line</td>
</tr>
<tr>
<td>LDFE</td>
<td>Large Displacement Finite Element</td>
</tr>
<tr>
<td>LI</td>
<td>Liquidity Index</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid Limit</td>
</tr>
<tr>
<td>LOC</td>
<td>Lightly Over Consolidated soil</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear Variable Differential Transformer</td>
</tr>
<tr>
<td>MC</td>
<td>the Mohr-Coulomb model</td>
</tr>
<tr>
<td>MCC</td>
<td>the Modified Cam Clay model</td>
</tr>
<tr>
<td>OCR</td>
<td>Over Consolidation Ratio</td>
</tr>
<tr>
<td>PDR</td>
<td>Penetration Data Recorder</td>
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<tr>
<td>PI</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>PL</td>
<td>Plastic Limit</td>
</tr>
<tr>
<td>RCC</td>
<td>Rowe Cell Consolidation test</td>
</tr>
<tr>
<td>SBT</td>
<td>Soil Behaviour Type</td>
</tr>
<tr>
<td>SD</td>
<td>Slurry Deposition</td>
</tr>
<tr>
<td>SS</td>
<td>Simple Shear test</td>
</tr>
<tr>
<td>UWA</td>
<td>The University of Western Australia</td>
</tr>
<tr>
<td>WP</td>
<td>Wet Pluviation</td>
</tr>
<tr>
<td>YSR</td>
<td>Yield Stress Ratio</td>
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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

The Cone Penetration Test (CPT) & Piezocone Penetration Test (CPTU) are the most popular onshore and offshore in-situ tests of the current suite of available in-situ tests. Strengths of the CPT/CPTU include its highly repeatable and cost-effective performance and the capability to obtain continuous stratigraphic profiling for various ground conditions. The test provides three primary CPT/CPTU measurements namely the cone resistance \( q_c \), sleeve friction \( f_s \) and pore pressure \( u \). These parameters not only assist in classification of the soil type but also allow estimation of geotechnical parameters along the depth penetrated by the cone. The CPT/CPTU has a high potential to provide reliable assessments of these soil properties which are essential for geotechnical design purposes.

The test procedure and the interpretation of CPT/CPTU data are well standardised and incorporated into practice (e.g. Lunne et al. 1997b). The interpretation methods have been developed from a number of theoretical and semi-empirical correlations, established by comparisons with laboratory element tests and chamber tests. In particular, estimation of geotechnical parameters for sand and clay has a relatively high reliability. This is because cone penetration at the standard penetration rate of 20±5 mm/s is expected to be fully drained in sand and fully undrained in clay.

However such reliability is poor in intermediate soils, such as silts, clayey and sandy silts and residual soils. Their soil properties, such as compressibility and permeability, are expected to be lie between those of sands and clays, and therefore the response of intermediate soils at the standard cone velocity is likely to be partially drained. The degree of partial drainage is likely to vary significantly depending on the soil composition and properties.

It is now well recognised that penetration resistance varies with penetration velocity. For soil types investigated to date, it has been found that penetration resistance generally increases as the velocity decreases due to the occurrence of local
consolidation around the penetrometer tip. In addition, the dominance of viscous effects at higher velocities in the undrained region leads to gains in penetration resistance with increasing velocity. However, it is still unclear how various soil characteristics affect the magnitude of the cone penetration resistance for any given drainage condition. The uncertainty of the drainage condition during penetrometer installation is an obstacle to the interpretation of CPT/CPTUs data. There is a poor level of confidence in the estimation of geotechnical parameters in the partially drained range and therefore research into penetrometer rate dependency has very high significance.

1.2 RESEARCH OBJECTIVES

This Thesis addresses the need for such guidance by examining the rate dependency of penetrometer resistance using a comprehensive series of penetrometer testing in the field and laboratory, and numerically simulating cone penetration at various velocities.

The key objectives of the research are to:

(i) Extend the current sparse database of in-situ variable rate piezocone tests by examining penetrations over a wide range of penetration rates to link penetrometer resistance in all drainage conditions (fully drained, partially drained and fully undrained);

(ii) Investigate effects of fines content on penetrometer resistance rate in respect to rate dependency, and evaluate these effects by combining observations from supplemental laboratory tests;

(iii) Evaluate factors affecting penetration resistance and controlling the partially drained condition by means of finite element numerical analyses, modelling the cone installation as spherical cavity expansion in PLAXIS;

(iv) Evaluate the experimental observations of penetrometer rate dependency using insights gained from the PLAXIS numerical analyses;

(v) Identify trends of penetrometer rate dependency in various soil types and provide general recommendations when conducting CPTs in practice to allow improved derivation of geotechnical parameters.
1.3 THESIS OUTLINE

This Thesis comprises the following Chapters:

**Chapter 2** reviews the literature relating to some basic knowledge on CPT and penetration rate effects observed in the field and laboratory. Some studies from the literature are highlighted to introduce evidence of penetration rate effects. The cavity expansion method is also reviewed with respect to its approach into penetrometer rate effects. The review material collated is used to assist interpretation later in the Thesis.

**Chapter 3** reports the results of the site and soil characterisation studies performed for the variable rate field tests presented in Chapter 4. This Chapter also details soil properties of reconstituted kaolin-sand mixtures, in which laboratory CPTs were performed (presented in Chapter 5 and Chapter 6). The equipment used for the field tests presented in Chapter 4 are described in this Chapter.

**Chapter 4** presents the field penetration test results from a comprehensive series of variable penetration rate test results performed at three sites in Western Australia: Gingin, Bassendean and Burswood.

**Chapter 5** describes the variable rate miniature CPTs carried out in the drum centrifuge. The effect of fines content on CPT end resistance is examined on reconstituted kaolin-sand mixtures with varying proportions of sand. Supplementary laboratory test results on those mixtures are reported in Chapter 3 and are used to assist interpretation.

**Chapter 6** describes the results of miniature piezocone penetration tests performed at various penetration rates in pressurised containers at 1g. Comparisons of the test results are made with the centrifuge studies (i.e. Chapter 5 and the previous studies from the literature) to highlight evidence of penetration rate effects in clayey soils.

**Chapter 7** presents numerical simulations of the spherical cavity expansion in PLAXIS. The velocity of cavity expansion was varied using a coupled consolidation analysis and then linked to variable rate cone penetrations. The factors affecting cone penetration resistance and drainage conditions are addressed in normally consolidated, non-dilative/contractive soils. Some insights into those factors for overconsolidated and dilative soils are explored in the same fashion.
Chapter 8 evaluates empirical observations of penetrometer rate dependency presented in Chapters 2, 4, 5 and 6 based on the insights gained from the numerical components of the research in Chapter 7. This evaluation provides deeper insights into the rate dependency of penetration resistance and expectations for a range of different soil types. Practical recommendations are provided to allow improved derivation of geotechnical parameters in intermediate soils.

Chapter 9 summarises the key conclusions from each element of the research and provides recommendations for future research in this area.

Supplemental information is provided in the following appendices:

Appendix A reports a series of monotonic simple shear test results on clayey sands and kaolin clay in the UWA (Berkeley type) simple shear apparatus (refer to Section 3.6.4).

Appendix B shows variable rate penetration test results at the Bassendean site, using piezocone, T-bar and ball penetrometers (refer to Section 4.3.4).

Appendix C tabulates the variable rate cone penetration test results on clayey sands in the drum centrifuge test (refer to Section 5.4).

Appendix D provides the papers published along with the research of this Thesis, as follows:


CHAPTER 2   LITERATURE REVIEW

2.1   INTRODUCTION

This Chapter reviews current understanding of Cone Penetration Testing (CPT) with respect to penetrometer rate effects. For the CPT and piezocone test (CPTU), testing procedures are well standardised and incorporated into practice (Lunne et al. 1997b). Interpretation methods for CPT/CPTU tests in sand and clay are relatively reliable and well established (e.g. Robertson 2009). However, interpretation methods for intermediate soils are poorly developed as conditions during cone penetration are neither fully drained or fully undrained.

This Chapter firstly provides an overview of essential features of a CPT and then reviews the existing database on rate effects collected from previous studies. The review includes some field experiments and laboratory experiments and intends to provide general observations of penetration rate effects. The numerical studies on penetration rate effects are also reviewed, together with some background of the cavity expansion method to analyse cone resistance. This review material is drawn upon later in the Thesis to assist interpretation.

2.1.1   Standard CPT/CPTU

After the first mechanical cone penetrometer was built in 1932 (Lunne et al. 1997b), electrical cone penetrometers and piezocone penetrometers (which can measure pore pressures) so-called CPT and CPTU have been widely employed as tools for both onshore and offshore site investigation. The testing procedure has been established in the International Reference Test Procedure (IRTP) (ISSMFE 1999) and followed by many standards (e.g. ASTM D5778 2007, ISO/CEN 2007). The standard cone penetrometer currently used in practice has a diameter of 35.7 mm with a cone apex angle of 60º (i.e. a projected end area of 10 cm²). In offshore site investigation, CPTUs with a larger diameter of 43.7 mm (i.e. a projected end area of 15 cm²) are often alternatively used for seabed mode testing (Lunne et al. 2011). Standard CPT/CPTU penetration is at a rate of 20 mm/s with a tolerance of ±5 mm/s.
The CPT measures a total force acting on the cone face and a total force acting on the friction sleeve. These are divided by the projected end areas (10 cm$^2$ for the cone face and 150 cm$^2$ for the friction sleeve) and then tip resistance ($q_c$) and sleeve friction ($f_s$) are accordingly calculated. The CPTU is performed with a piezocone, which additionally provides pore pressure measurements; these measurements are currently almost always made at the shoulder of the cone (so-called $u_2$ position). Due to the geometry of the piezocone, $q_c$ and $f_s$ measurements are influenced by ambient pore pressures acting on the shoulder behind the cone. Therefore, $q_c$ needs to be corrected for this effect, which is known as the unequal area effect (Lunne et al. 1997b), using the following relationship:

$$q_t = q_c + u_2 (1 - \alpha)$$

(2-1)

where $q_t$ is the corrected cone resistance and $\alpha$ is the unequal area ratio, which is ideally close to unity, although many piezocones have a typical value between 0.55 and 0.9 (Lunne et al. 1997b). A similar correction is given for $f_s$ measurement (depending on the CPTU configuration) but this correction is often not possible due to the unavailability of pore pressure measurements behind the friction sleeve (so-called $u_3$ position).

Normalised CPT/CPTU parameters are used to identify soil stratigraphy and soil behaviour type (SBT). The parameters include a normalised cone resistance ($Q$), a pore pressure ratio ($B_q$) and a friction ratio ($F_r$), which are calculated from the following equations (e.g. Wroth 1988, Lunne et al. 1997b):

$$Q = \frac{q_t - \sigma_{v0}}{\sigma_{v0}'} = \frac{q_{\text{net}}}{\sigma_{v0}'}$$

(2-2)

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} = \frac{\Delta u}{q_{\text{net}}}$$

(2-3)

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} \times 100 \, (\%)$$

(2-4)

where $\sigma_{v0}$ is the total vertical overburden stress, $\sigma_{v0}'$ is the effective vertical overburden stress, $q_{\text{net}}$ is the net cone resistance, $u_0$ is the hydrostatic pore pressure and $\Delta u$ is the excess pore pressure measured at $u_2$ position.

Figure 2.1 shows the most commonly used soil behaviour type classification charts developed by Robertson (1990). The normalised CPTU parameters are used to account
for the effect of overburden stress. A further development of this soil classification framework has been proposed by Schneider et al. (2008) who plot of $Q-\Delta u/\sigma_0'$ space and $Q-B_q$ space (see Figure 2.2). These authors suggest that the use of $Q-\Delta u/\sigma_0'$ space can help assessment of the degree of partial consolidation during cone insertion. A recent study by Schneider et al. (2012) has extended the framework onto a plot of $Q-F_r$ based on analytical predictions of $Q$ and $F_r$.

For data falling with the boundaries of SBT Zone 2 to 7 on the soil behaviour type on the $Q-F_r$ chart, Robertson & Wride (1998) adopt the soil behaviour type index ($I_c$), which was proposed first by Jefferies & Davies (1993), defined as:

$$I_c = [ (3.47 - \log Q)^2 + (\log F_r + 1.22)^2 ]^{0.5}$$ (2-5)

The soil behaviour type corresponding to $I_c$ is summarised in Figure 2.1.

Robertson (2009) suggested the use of a normalised cone resistance ($Q_{tn}$) with a variable stress exponent ($n$) after discussions made on the appropriate overburden stress for normalisation by several studies (e.g. Robertson & Wride 1998, Boulanger & Idriss 2004, Moss et al. 2006). $Q_{tn}$ is defined as:

$$Q_{tn} = \left( \frac{q_t - \sigma_{v0}}{p_a} \right) \left( \frac{p_a}{\sigma_{v0}'} \right)^n$$ (2-6)

where $p_a$ is atmospheric pressure in the same unit as $q_t$ and $\sigma_{v0}$, $(q_t - \sigma_{v0})/p_a$ is the dimensionless net cone resistance, $(p_a/\sigma_{v0}')^n$ is the stress normalisation factor. Robertson (2009) recommended that the stress normalisation exponent ($n$) varies as the following function of $I_c$ and the effective overburden stress ($\sigma_{v0}/p_a$):

$$n = 0.381(I_c) + 0.05(\sigma_{v0}/p_a) - 0.15$$ (2-7)

where $n \leq 1.0$. The $n$ value is likely to range from 0.5 to 0.9 in most coarse-grained soils while $n=1.0$ for most fine-grained soils. Contours of $I_c$ (with $n=1$) are shown in Figure 2.1. When $n=1$, $Q_{tn}=Q_t$ (=Q) and $I_c$ remains the same. When $n<1.0$, $Q_{tn}$ is used in $I_c$ calculation (instead of $Q$) and therefore, iteration is required to evaluate Equations (2-5) to (2-7).
2. Literature review

<table>
<thead>
<tr>
<th>SBT zone</th>
<th>SBT</th>
<th>$I_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive fine-grained</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>Organic soil - clay</td>
<td>$I_c &gt; 3.60$</td>
</tr>
<tr>
<td>3</td>
<td>Clays - silty clay to clay</td>
<td>$2.95 &lt; I_c &lt; 3.60$</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixtures - clayey silt to silty clay</td>
<td>$2.60 &lt; I_c &lt; 2.95$</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures - silty sand to sandy silt</td>
<td>$2.05 &lt; I_c &lt; 2.60$</td>
</tr>
<tr>
<td>6</td>
<td>Sands - clean sand to silty sand</td>
<td>$1.31 &lt; I_c &lt; 2.05$</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly sand to dense sand</td>
<td>$I_c &lt; 1.31$</td>
</tr>
<tr>
<td>8</td>
<td>Very dense/stiff soil*</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>Very stoff fine-grained soil*</td>
<td>N/A</td>
</tr>
</tbody>
</table>

* Overconsolidated and/or cemented

Figure 2.1 Soil behaviour type charts and soil behaviour type index $I_c$ on: (a) $Q-F_r$ and (b) $Q-B_q$ (Robertson 1990, Robertson 2009)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1a</td>
<td>SILTS and ‘Low B’ CLAYS</td>
</tr>
<tr>
<td>1b</td>
<td>CLAYS</td>
</tr>
<tr>
<td>1c</td>
<td>Sensitive CLAYS</td>
</tr>
<tr>
<td>2</td>
<td>Essentially drained SANDS</td>
</tr>
<tr>
<td>3</td>
<td>Transitional soils</td>
</tr>
</tbody>
</table>

Figure 2.2 Soil classification chart on: $Q-\Delta u/\sigma'_{v0}$ (Schneider et al. 2008)
2.1.2 Performance of CPT interpretation methods

The CPT/CPTU measurements made during drained or undrained penetration are used to derive soil parameters for geotechnical design projects. A number of theoretical and semi-empirical correlations have been developed which essentially depend on the soil type. The interpretation methods are divided into two categories: sand and clay i.e. essentially drained and undrained based on the test at the standard penetration rate of 20 mm/s. The correlations are developed based on bearing capacity theory, cavity expansion theory and semi-empirical methods assisted using calibration chamber test data. The correlation and interpretation methods have been widely covered in the following textbooks/papers.

- Lunne et al. (1997b): *Cone penetration testing in geotechnical practice*
- Robertson (2009): *Interpretation of cone penetration tests - a unified approach*
- Schnaid (2009): *In situ testing in geomechanics: the main tests*
- Robertson & Cabal (2012): *Guide to Cone Penetration Testing for Geotechnical Engineering*
- Mayne (2014): *Interpretation of geotechnical parameters from seismic piezocone tests*

Table 2.1 presents one of the recent reviews about performance of CPTU for soil parameter derivation, which can be found in Robertson & Cabal (2012). It suggests that use of CPTU is widely applicable to estimate soil parameters in sand and clay. Robertson (2009) suggests that an approximate transition between drained (sand-like behaviour) and undrained (clay-like behaviour) conditions is likely to exist around $I_c=2.60$.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$D_r$</th>
<th>$\phi_0$</th>
<th>$K_0$</th>
<th>OCR</th>
<th>$S_t$</th>
<th>$s_u$</th>
<th>$\phi$</th>
<th>$E_G$</th>
<th>$M$</th>
<th>$G_0$</th>
<th>$k$</th>
<th>$c_h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>2-3</td>
<td>2-3</td>
<td>5</td>
<td>5</td>
<td>2-3</td>
<td>2-3</td>
<td>2-3</td>
<td>2-3</td>
<td>2-3</td>
<td>3-4</td>
<td>3-4</td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>1-2</td>
<td>4</td>
<td>2-4</td>
<td>2-3</td>
<td>2-4</td>
<td>2-3</td>
<td>3-4</td>
<td>3-4</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1=high, 2=high to moderate, 3=moderate, 4=moderate to low, 5=low reliability, blank=no applicability, *with SCPT

However, no category exists in Table 2.1 for intermediate soils such as silts, clayey and sandy silts and residual soils. These ‘nontextbook’ soils are referred to as ‘unusual’ geomaterials in Schnaid et al. (2004). The response of intermediate soils at the standard penetration rate of 20 mm/s is likely to be partially drained, and the degree of partial
drainage often varies significantly depending on the soil composition. Therefore, it is necessary to identify the drainage condition during CPT/CPTU in any soil types to minimise the uncertainties in the derivation of geotechnical parameters. Since a penetration process essentially happens over a given penetration time, one effective approach to distinguish drainage conditions is to examine penetrometer rate dependency in the soils.

2.2 EXISTING DATABASE ON RATE EFFECTS

2.2.1 Field experiments

Penetrometer rate effects in the field have been examined for over four decades (probably since the first cone penetrometer had been developed). Lunne et al. (1997b) listed a number of publications which investigated cone penetration rate dependency. In addition to this literature survey (in which publications before 1989 were collected), recent studies of in situ penetration rate effects are summarised in Table 2.2. The main observations obtained at the specified velocity range are summarised for each paper. In the section below, some of the studies are selected from the existing database to provide an overview of field observations of penetrometer rate effects.

Bemben & Myers (1974) conducted mechanical cone penetration tests at penetration rates (v) ranging between 0.2 mm/s and 200 mm/s in lightly overconsolidated varved clay. As seen in Figure 2.3, the cone penetration resistance (qc) was a minimum when v=2 mm/s, and then increased by 40% as v decreased from 2 to 0.4 mm/s. Also, a 50% increase in qc was obtained as v increased from 2 to 20 mm/s, with a slight further increase observed at v up to 50 mm/s. The authors argued that drained conditions apply for v below ~0.5 mm/s and undrained conditions apply to v above ~50 mm/s. In this case, it is interesting to note that the undrained resistance was higher than the drained resistance, and moreover, minimum resistance occurred during partially drained penetration. No pore pressure measurements were reported for this case study.
Powell & Quarterman (1988) presented CPT results at penetration rates of 0.17, 1.7 and 17 mm/s at four sites. Figure 2.4 shows the cone resistances (against depth) for penetration rate of 0.17 and 1.7 mm/s expressed as a percentage of the resistance at 17 mm/s. The test sites include glacial till (Cowden) and heavily overconsolidated clays (Lion Yard, Brent, Canons Park). Rate effects were clearly evident at all sites and approximately 10 to 20% increase in cone resistance ($q_t$) was observed over the tenfold increase in penetration rate (see Figure 2.4). The authors commented that rate effect is controlled by ground conditions such as permeability and plasticity index ($I_p$) and suggested that rate effects are higher in the clays with higher $I_p$ values.
Finke et al. (2001) conducted piezocone penetration tests at \( v = 20 \) and 200 mm/s using the 15 cm\(^2\) probes in residual silty soils at Opelika, US (see Figure 2.5). The authors observed that \( q_c \) and \( f_s \) decreased by some degree while \( u_1 \) (pore pressures measured at the cone midface) increased by about 200 kPa with the increased penetration rate. Interestingly, \( u_2 \) became positive at \( v = 200 \) mm/s but was negative at \( v = 20 \) mm/s (see Figure 2.5). In addition, the effect of the cone size was examined at the site by employing the 10 cm\(^2\) and 15 cm\(^2\) probes. The authors commented that the observed variations were rather indicative of the natural inherent soil profiles and thus effect of cone size appeared to be minor. The dissipation of excess pore pressures was rapid. For example, the time for 50% excess pore pressure dissipation (\( t_{50} \)) at the \( u_2 \) position was indicated to be about 5 to 20 seconds. A permeability (\( k_h \)) of about \( 10^{-6} \) m/s was inferred from a correlation proposed by Parez & Fauriel (1988) and the material was classified as between a sandy silt and silty sand.

![Figure 2.5 Penetration rate effect at Piedmont residuum (Finke et al. 2001)](image)

Schneider et al. (2004) conducted variable penetration rate tests at velocities between 0.01 and 20 mm/s in a high plasticity silty clay at Burswood, Australia. The penetration rates were varied by means of the progressive halving of velocities in the so-called ‘twitch’ test (House et al. 2001). Figure 2.6 (top) compares the variable rate test result
with the test result at the standard rate ($v$) of 20 mm/s. The observed penetration rate effects were further assessed in Figure 2.6 (bottom) using normalised velocities ($V$), which Finnie & Randolph (1994), and others have used for the characterisation of rate effects (described in the next section).

Viscous effects within the undrained range were inferred above the transition point at $v \sim 1$ mm (corresponding to $V=50$ on the figure) while partial consolidation effects were interpreted below the transition where $q_{net}$ started to increase again. The authors also performed T-bar and ball variable penetration rate tests and observed that the rate dependence of the resistance measured by these penetrometers was generally similar to that of the piezocone. A reduction of 15% to 20% in penetrometer resistances were apparent as the rates reduced from 20 mm/s to 1 mm/s.

![Figure 2.6 Penetration rate effect at Burswood (Schneider et al. 2004)](image-url)
Kim (2005) performed variable cone penetration rate tests in silty clay to clayey silt at Indiana, US. The penetration rate was varied between 0.01 and 20 mm/s. An example of the test results is presented in Figure 2.7 (top). The transitions of drainage conditions from undrained to partially drained occurred at velocities $v \sim 0.1$ mm/s for silty clay (7.6-8.1m depth) and at $v \sim 1$ mm/s for clayey silt (9.2-10.2m depth). A significant increase in $q_t$ was observed below the transition points. The $q_t$ values measured at the slowest velocities were about 2 to 2.5 times those at the standard penetration rate of 20 mm/s.

Kim et al. (2008) reported that the drainage transitions were captured by the normalised velocity ($V = vd/c_v$) using the vertical coefficient of consolidation ($c_v$) measured at corresponding stress in oedometer tests. The authors concluded that the drainage transition from undrained to partially drained condition occurred at $V$ between 4 and 10.

Figure 2.7 Penetration rate effect at Indiana (Kim et al. 2006)
Poulsen et al. (2012b) and Poulsen et al. (2013) reported a series of piezocone test results in sandy silt with clay bands at Dronninglud, Denmark. Penetration rates between 0.5 mm/s and 60 mm/s were used in the test series. The authors observed high fluctuations in $q_t$, $f_s$ and $u_2$ profiles due to the highly layered soil stratigraphy in the field. Therefore, the authors conducted smoothing of the data by taking an average of the data over every 50 cm. The smoothened $q_t$ and $u_2$ profiles at the velocities of 0.5 mm/s and 60 mm/s were compared as shown in Figure 2.8.

The $q_t$ values at $v=60$ mm/s tend to be higher than those at $v=0.5$ mm/s. The $u_2$ values at $v=0.5$ mm/s were more or less equal to the hydrostatic pressures at the site, but $u_2$ values recorded at $v=60$ mm/s were positive and above hydrostatic values. The authors proposed the correlation between $q_t$ and $v$, and also one between $u_2$ and $v$. It is evident that the $q_t$ and $u_2$ are dependent on penetration rate in the sandy silts although it is uncertain if the test at $v=20$ mm/s is drained, partially drained or undrained in the soils.

Figure 2.8 Penetration rate effect at Dronninglud (Poulsen et al. 2013)
Rate effects are affected even in sands. Danziger & Lunne (2012) collated experimental observations of penetration rate effects in sands. The authors found for some cases that $q_c$ at $v=2$ mm/s is higher than the $q_c$ at $v=20$ mm/s in loose fine sands and/or loose silty sands. In contrast, the $q_c$ at $v=20$ mm/s can be higher than the $q_c$ at $v=2$ mm/s in dense to very sands. The authors postulate that the former is triggered by the excess pore pressure while the latter is caused by grain crushing and possible effects of dilatancy.

Te Kamp (1982), which is one of the collected studies in Danziger & Lunne (2012), reported that an increase in $q_c$ and $f_s$ in dense and very dense saturated fine sands when the penetration rate increased from 0.033 to 100 mm/s. Te Kamp (1982) argued that this observation can be due to effects of dilatancy but unfortunately no supportive data were reported (e.g. such as negative excess pore pressures).

Another interesting study has been published by McNeilan & Bugno (1985), who conducted a series of CPTs at the standard rate of 20 mm/s in silts offshore of California. In this study, the CPT data were compared with laboratory strength and permeability test data. Figure 2.9 shows interpreted trends between cone resistance and permeability. The authors commented that for the standard CPTs in the (late Pleistocene age) silts with permeability ranging from $10^{-9}$ to $10^{-3}$ m/s, partially drainage conditions are of primary importance in understanding and applying the CPT data. The data also showed that the drained to undrained resistance ratio of this silt at the same stress level is in the range of about 8.5 to 13, noting that silt compositions were in the range of very sandy silt to clayey silt.

![Figure 2.9 Interpretation of relationship between cone resistance and permeability (McNeilan & Bugno 1985)](image-url)
General comment on field test investigations

The field evidence presented here (and summarised in Table 2.2) confirms the existence of a strong rate dependency of penetration resistance. However the significant variability in the trends observed indicates that there is considerable scope for improved understanding. It is also evident that any investigation of rate effects should cover a wide range of penetrometer velocities.
Table 2.2 Summary of data related to penetration rate effects in the field

<table>
<thead>
<tr>
<th>Reference</th>
<th>Probe</th>
<th>Measurem ent</th>
<th>Characteristics of soil</th>
<th>Penetration rate, v (mm/s)</th>
<th>Main observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bemben &amp; Myers (1974)</td>
<td>mechanical cone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;</td>
<td>varved clay</td>
<td>0.2-200</td>
<td>- undeadrained condition was obtained at v=50 mm/s; drained condition was obtained at v=0.5 mm/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>v=20 mm/s did not provide a fully undrained condition</td>
<td></td>
</tr>
<tr>
<td>Campanella et al. (1982),</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;, u&lt;sub&gt;t&lt;/sub&gt;</td>
<td>clayey silt</td>
<td>0.25-20</td>
<td>- penetration is essentially undrained above v=2 mm/s</td>
</tr>
<tr>
<td>Campanella et al. (1983)</td>
<td></td>
<td></td>
<td></td>
<td>u decreases with decreasing v at v&lt;2 mm/s, while q&lt;sub&gt;p&lt;/sub&gt; and f&lt;sub&gt;e&lt;/sub&gt; increases; the increase is noticeable in f&lt;sub&gt;e&lt;/sub&gt;</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- effective cone resistance is defined as q&lt;sub&gt;p&lt;/sub&gt; = q&lt;sub&gt;p&lt;/sub&gt;-u</td>
<td></td>
</tr>
<tr>
<td>Lacasse &amp; Lunne (1982)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, u&lt;sub&gt;t&lt;/sub&gt;</td>
<td>Onsoy clay, Drammen clay</td>
<td>2-100</td>
<td>- q&lt;sub&gt;p&lt;/sub&gt;/q&lt;sub&gt;D&lt;/sub&gt;~0.9-0.95 for both clays; q&lt;sub&gt;D&lt;/sub&gt;/q&lt;sub&gt;100&lt;/sub&gt;~1 for Onsoy clay; q&lt;sub&gt;D&lt;/sub&gt;/q&lt;sub&gt;100&lt;/sub&gt;~1.5 for Drammen clay</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>u&lt;sub&gt;2&lt;/sub&gt;/u&lt;sub&gt;100&lt;/sub&gt;~1 for both clays; u&lt;sub&gt;20&lt;/sub&gt;/u&lt;sub&gt;100&lt;/sub&gt;~1.1 for Drammen clay</td>
<td></td>
</tr>
<tr>
<td>Roy et al. (1982a)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, u</td>
<td>sensitive clay</td>
<td>0.5-40</td>
<td>- q&lt;sub&gt;p&lt;/sub&gt;/q&lt;sub&gt;D&lt;/sub&gt; is minimum at v~2.5 mm/s and increases when the rate increases non-linearly up to 40 mm/s</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- q&lt;sub&gt;p&lt;/sub&gt;/q&lt;sub&gt;D&lt;/sub&gt; increases with slower rates from v~2.5 mm/s</td>
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<td></td>
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<td>- penetration rate has no influence on the u measured on the cone face but a small effect may exist when u is measured behind the cone and along the shaft</td>
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<tr>
<td>Te Kamp (1982)</td>
<td>electrical cone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;, s&lt;sub&gt;t&lt;/sub&gt;</td>
<td>dense fine sand (on land and offshore)</td>
<td>0.2-100 (on), 0.033-20 (off)</td>
<td>- higher q&lt;sub&gt;p&lt;/sub&gt; and f&lt;sub&gt;e&lt;/sub&gt; are measured at faster penetration rates</td>
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<td></td>
<td>no obvious differences are found on rate effects between on land and offshore</td>
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<tr>
<td>Rocha Filho &amp; Alencar</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, u</td>
<td>Rio de Janeiro soft clay</td>
<td>0.5-15</td>
<td>- u was not affected by penetration rate in the range of v=5-15 mm/s, but a significant reduction in u at v=0.5 mm/s</td>
</tr>
<tr>
<td>(1985)</td>
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<td></td>
<td>- (q&lt;sub&gt;p&lt;/sub&gt; - u)/q&lt;sub&gt;D&lt;/sub&gt; increased 50-65% when the rate was decreased from 15 to 0.5 mm/s</td>
<td></td>
</tr>
<tr>
<td>McNeilan &amp; Bugno (1985)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;</td>
<td>California silts</td>
<td>20</td>
<td>- a unique relationship was developed between cone point resistance and permeability</td>
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<td>- partially drained was observed during cone testing at v=20 mm/s in one geologic deposit</td>
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<td>- the highest q&lt;sub&gt;p&lt;/sub&gt; is more than ten times the lowest q&lt;sub&gt;p&lt;/sub&gt; within a given depth zone in the silts deposit</td>
<td></td>
</tr>
<tr>
<td>Lunne et al. (1986)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;, u&lt;sub&gt;t&lt;/sub&gt;</td>
<td>glacial till (Cowden), highly OC clay (Brent Cross)</td>
<td>2-20</td>
<td>- at Cowden, u measured at the cone tip and in the middle of the cone face was negative with 20 mm/s, but positive with 2 mm/s</td>
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<td></td>
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<td></td>
<td>- u&lt;sub&gt;e&lt;/sub&gt; was negative with both rates at Cowden and at the Brent Cross</td>
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<td></td>
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<td>- piezocone tests at Cowden with 2 mm/s showed a reduction of q&lt;sub&gt;p&lt;/sub&gt; and f&lt;sub&gt;e&lt;/sub&gt;, about 10-15 % and 10-20 % compared to standard penetrations, respectively</td>
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<tr>
<td>Konrad (1987)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;, u</td>
<td>clay</td>
<td>5-20</td>
<td>- u at 4 times of the diameter above the cone base showed no significant change within the range of rates used</td>
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<td></td>
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<td></td>
<td>rate effects at other measurements were not mentioned</td>
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<tr>
<td>Powell &amp; Quarterman (1988)</td>
<td>electrical cone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;</td>
<td>glacial till, OC clays</td>
<td>0.167-20</td>
<td>- rate effects exist at four sites and were dependent on site condition such as permeability and f&lt;sub&gt;e&lt;/sub&gt;</td>
</tr>
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<td></td>
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<td></td>
<td></td>
<td>- u&lt;sub&gt;e&lt;/sub&gt; are close to zero (often in suction) so correction for pore pressure can be ignored</td>
<td></td>
</tr>
<tr>
<td>Juran &amp; Tumay (1989)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, u</td>
<td>clay, sand</td>
<td>2-100</td>
<td>- penetration rate was found to have no appreciable effect on q&lt;sub&gt;p&lt;/sub&gt;, but it did on u at the tip, mainly in sand porous only at the tip: u at 2 mm/s approaches u&lt;sub&gt;0&lt;/sub&gt; whereas u at 100 mm/s reached 4 times of u&lt;sub&gt;0&lt;/sub&gt;</td>
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<td>- pore pressure increase at the tip: u at 2 mm/s approaches u&lt;sub&gt;0&lt;/sub&gt; whereas u at 100 mm/s reached 4 times of u&lt;sub&gt;0&lt;/sub&gt;</td>
<td></td>
</tr>
<tr>
<td>Finke et al. (2001)</td>
<td>piezocone</td>
<td>q&lt;sub&gt;p&lt;/sub&gt;, f&lt;sub&gt;e&lt;/sub&gt;, u&lt;sub&gt;t&lt;/sub&gt;, u&lt;sub&gt;2&lt;/sub&gt;</td>
<td>residual silty soils, Piedmont</td>
<td>20-200</td>
<td>- q&lt;sub&gt;p&lt;/sub&gt; and f&lt;sub&gt;e&lt;/sub&gt; overall decrease with increased penetration rate, while u significantly increases; even u&lt;sub&gt;2&lt;/sub&gt; becomes positive from negative at the standard velocity</td>
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<td></td>
<td>- there seems to be minor effects of cone size on test results comparing 10 cm&lt;sup&gt;2&lt;/sup&gt; and 15 cm&lt;sup&gt;2&lt;/sup&gt; penetrometers</td>
<td></td>
</tr>
<tr>
<td>Reference</td>
<td>Probe</td>
<td>Measurement</td>
<td>Characteristics of soil</td>
<td>Penetration rate, $\nu$ (mm/s)</td>
<td>Main observations</td>
</tr>
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<td>-------------------------------</td>
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</tbody>
</table>
| Schneider et al. (2004)       | piezocone, T-bar, Ball | $q_p$, $u$, $q_{T-bar}$ | Burswood clay                             | 0.01-20                      | - $q$ and $u$ decrease when the penetration rate decreases, a slight increase in $q$ occurs at $v<1$ mm/s  
- transition point from undrained to partially drained is approximately $v=1$ mm/s  
- a ratio of $q_{T-bar}/q_{bal}$ varies from 1.1 to 0.8 while $\Delta u_{T}/\Delta u_{bal}$ decreases to 0.4 when penetrations are slower  
- T-bar and Ball probes were also used to assess undrained viscous effects  
- coefficient of consolidation is assessed by conducting “twitch” test |
| Poulsen et al. (2012b)        | piezocone              | $q_p$, $u$, $q_{T-bar}$ | Burswood clay                             | 0.01-10                      | - $q$ and $u$ decrease when the penetration rate decreases, a slight increase in $q$ occurs at $v<1$ mm/s  
- transition point from undrained to partially drained is approximately $v=1$ mm/s  
- a ratio of $q_{T-bar}/q_{bal}$ varies from 1.1 to 0.8 while $\Delta u_{T}/\Delta u_{bal}$ decreases to 0.4 when penetrations are slower  
- T-bar and Ball probes were also used to assess undrained viscous effects  
- coefficient of consolidation is assessed by conducting “twitch” test |
| Poulsen et al. (2013)         | piezocone              | $q_p$, $u$, $q_{T-bar}$ | Burswood clay                             | 0.01-20                      | - $q$ and $u$ decrease when the penetration rate decreases, a slight increase in $q$ occurs at $v<1$ mm/s  
- transition point from undrained to partially drained is approximately $v=1$ mm/s  
- a ratio of $q_{T-bar}/q_{bal}$ varies from 1.1 to 0.8 while $\Delta u_{T}/\Delta u_{bal}$ decreases to 0.4 when penetrations are slower  
- T-bar and Ball probes were also used to assess undrained viscous effects  
- coefficient of consolidation is assessed by conducting “twitch” test |
| Sacchetto & Trevisan (2010)   | piezocone              | $q_p$, $u$, $q_{T-bar}$ | Burswood clay                             | 0.01-20                      | - $q$ and $u$ decrease when the penetration rate decreases, a slight increase in $q$ occurs at $v<1$ mm/s  
- transition point from undrained to partially drained is approximately $v=1$ mm/s  
- a ratio of $q_{T-bar}/q_{bal}$ varies from 1.1 to 0.8 while $\Delta u_{T}/\Delta u_{bal}$ decreases to 0.4 when penetrations are slower  
- T-bar and Ball probes were also used to assess undrained viscous effects  
- coefficient of consolidation is assessed by conducting “twitch” test |
| Lo Presti et al. (2010)       | piezocone              | $q_p$, $u$, $q_{T-bar}$ | clay, silt, sand (variable deposits)      | 10-20                        | - in intermediate soils, both $q$ and $u$ increase but $u$ decreases with slower penetration  
- in clays, rate effects changing velocities from 20 to 10 mm/s are negligible  
- data collected from Schneider et al. (2004) and Yafrate et al. (2007)  
- the rate coefficient, $\mu$, lies between 0.10 and 0.21 in undrained penetration  
- transition point from undrained to partially drained is approximately 1 mm/s |
| Tonni & Gottardi (2009)       | piezocone              | $q_p$, $u$    | Venetian silt                             | 1.5-40                       | - highly layered soil stratigraphy was encountered (within 7-20m depth)  
- negative to low excess pore pressures (but rather fluctuated) were observed due to dilative behaviour of the silt  
- a general increase in $q$ can be observed at $v=1$ mm/s  
- drainage identification is very difficult due to dilative behaviour and the highly stratified silty sediments |
| Tonni & Gottardi (2011)       | piezocone              | $q_p$, $u$    | Venetian silt                             | 1.5-40                       | - highly layered soil stratigraphy was encountered (within 7-20m depth)  
- negative to low excess pore pressures (but rather fluctuated) were observed due to dilative behaviour of the silt  
- a general increase in $q$ can be observed at $v=1$ mm/s  
- drainage identification is very difficult due to dilative behaviour and the highly stratified silty sediments |
| Sacchetto & Trevisan (2010)   | piezocone              | $q_p$        | clay, sand (variable deposits)            | 7.4-20                       | - $q_{T-bar}/q_{bal}$ (average) ~ 0.88-0.93, $q_{T-bar}/q_{bal}$ (clay) ~ 0.93-0.95, $q_{T-bar}/q_{bal}$ (sand) ~ 0.83-0.89  
- $f_i$ and $u$ were not reported  
- $q_{T-bar}/q_{bal}$ increases with increasing penetration rate, indicating a decay in partial drainage effects |
| Scnand et al. (2010)          | piezocone              | $q_p$, $u$    | bauxite and gold mine tailings           | 1-20                         | - $u$ decreases with a slight increase in $q$ as penetration rate was reduced  
- drainage characteristic curve is determined as $U = V$, where $U=(q_2 - q_{bal})(q_{bal} - q_{max})$ and $V=vd/c_{bal}$  
- it is recommended to avoid partial drainage condition, which occurs for $V$ in the range of 0.01 to 100 |
| Boylan et al. (2011)          | piezocone              | $q_p$, $u$    | Burswood clay                             | 5-20                         | - Lochrea: penetration at 20 mm/s provides the minimum $q_{T-bar}$, viscous effects are dominant at $v=50$ mm/s  
- Vinkeveen: $q_{T-bar}$ decreases with increasing penetration rate, indicating a decay in partial drainage effects  
- Vinkeveen: no conclusion can be made if penetration at 100 mm/s is undrained or partially drained  
- Bodegraven: no clear rate dependence can be concluded |
| Poulson et al. (2012b)        | piezocone              | $q_p$, $u$    | sandy silt with clay stripes             | 0.5-60                       | - highly layered soil stratigraphy was encountered, and therefore the results were smoothed for every 50cm  
- $q$ increases when $v$ is lowered  
- $q_{T-bar}/q_{bal}$, $\zeta$ values vary from 0 to 1.7 |
| García Martinez et al. (2014) | piezocone              | $q_p$, $u$    | silty sand to sandy silt                 | 10-130                      | - penetrations at velocities from 10 to 80 mm/s showed a decrease in $q$ and an increase in $u$  
- $q$ increases as velocity further increased to 130 mm/s  
- data collected from Schneider et al. (2004) and Yafrate et al. (2007)  
- the rate coefficient, $\mu$, lies between 0.10 and 0.21 in undrained penetration  
- transition point from undrained to partially drained is approximately 1 mm/s |

(Cont.)
2. Literature review

2.2.2 Laboratory experiments

Penetration rate effects have also been investigated in the laboratory to assist interpretation of field observations. A number of experimental studies on this topic have used geotechnical centrifuges, pressure chambers and other devices. The use of the centrifuge facility to study penetration rate effects has increased since the summary of experimental studies provided in Lunne et al. (1997b). Data from several studies on penetration rate effects performed in the centrifuge and at 1g were collected for this Thesis and are summarised in Table 2.3 and Table 2.4 respectively. In the following section, some studies from the existing database of the laboratory experiments are reviewed.

Finnie & Randolph (1994) performed a series of circular foundation penetration tests (for which prototype diameters of the foundations ranged from 1 to 15 m) at penetration velocities of 0.1 mm/s to 3 mm/s. The soils used for the tests were carbonate silt and sand with coefficients of consolidation \(c_v\) of \(5 \times 10^{-5}\ m^2/s\ (1576\ m^2/year)\) for the silt and \(1 \times 10^{-3}\ m^2/s\ (31536\ m^2/year)\) for sand respectively. To account for variables that affect penetration bearing resistances, the authors proposed a normalised velocity \(V\), defined as:

\[
V = \frac{vd}{c_v}
\]  

(2-8)

where \(v\) is the penetration velocity, \(d\) is the diameter of the penetrometer/foundation and \(c_v\) is the vertical coefficient of consolidation measured in a 1-D consolidation test. The penetration tests are presented in terms of \(V\) on Figure 2.10, which suggests that the transition from drained to partially drained conditions occurs at \(V\sim0.01\) and the transition from partially drained to undrained conditions occurs at \(V\sim30\). The authors also noted that the reduction in bearing capacity for the sand in undrained conditions was much less dramatic than that seen for bearing capacity of the silt; they attributed this trend to the stronger dilation characteristics of the sand.
Penetration rate dependency in kaolin clay has been widely investigated in the beam and drum centrifuge at the UWA (e.g. House et al. 2001, Randolph & Hope 2004, Schneider et al. 2007, Lehane et al. 2009). In addition to miniature cone/piezocone penetrometers, ‘full-flow’ (T-bar and ball) penetrometers have been increasingly used after the introduction by Stewart & Randolph (1991) and Watson et al. (1998) of the T-bar and ball probe respectively. The net T-bar resistance ($q_{T-bar}$) and net ball resistance ($q_{ball}$) are generally expressed as (Chung & Randolph 2004):

$$q_{T-bar} = q_m - [\sigma_v - u_0(1 - \alpha)]A_s/A_p$$

(2-9)

where $q_m$ is the measured penetration resistance, $A_s$ is the cross-sectional area of the connection shaft and $A_p$ is the projected area of the penetrometer in a plane normal to the shaft. A comparison of net resistance profiles between various penetrometers can be found in Chung & Randolph (2004).

Figure 2.11 shows one example of a series of variable piezocone penetration rate test results originally reported in Randolph & Hope (2004). The authors used the 10 mm diameter piezocone and the velocities between 0.005 and 3 mm/s under an accelerated gravitation of 100g. Partial consolidation effects are evident at $V$ approximately between 30 and 0.3. As $V$ reduced from 30 to 0.3, the net cone resistance relative to the undrained reference resistance ($q_{net}/q_{ref}$) increased while the pore pressure ratio ($B_q$) is reduced (see Figure 2.11).
House et al. (2001) found from their variable rate T-bar penetration tests, that T-bar bearing resistance \(q\) at partially drained condition can be captured by a hyperbolic curve of the following form:

\[
\frac{q}{q_{\text{ref}}} = a + \frac{b}{1 + cv^d}
\]  

(2-10)

where \(q/q_{\text{ref}}\) is the penetration resistance normalised by the ‘undrained’ reference value, \(V\) is the non-dimensional velocity (=\(vd/c_v\)) and \(a, b, c\) and \(d\) are the fitting parameters.

Equation (2-10) does not include viscous effects which increase with penetrometer velocity. Viscous effects in undrained conditions are often referred as strain rate effects in various undrained element tests such as triaxial compression tests, direct simple shear tests and vane shear tests (e.g. Kulhawy & Mayne 1990, Lunne & Andersen 2007, Peuchen & Mayne 2007). In these tests, an increase in undrained strength relative to the reference strength is often expressed using semi-logarithmical or power law.
Chung et al. (2006) proposed the following form in order to capture both consolidation and undrained viscous effects after the Randolph & Hope (2004) formulation with a minor modification.

\[
q_{\text{net}}/q_{\text{ref}} = \left( a + \frac{b}{1 + cV_d} \right) \left[ 1 + \frac{\mu}{\ln(10)} [\sinh^{-1}(V/V_0)] \right] \left[ 1 + \frac{\mu}{\ln(10)} [\sinh^{-1}(V_{\text{ref}}/V_0)] \right] \tag{2-11}
\]

where \( \mu \) is a rate coefficient (generally varying between 0.1 and 0.2), \( V_0 \) is the value of \( V \) where the viscous effects start to reduce to zero, \( V_{\text{ref}} \) is the value \( V \) where the hyperbolic function term passes through unity. Low et al. (2008) evaluated viscous effects based on the field studies with T-bar and ball probes reported by Schneider et al. (2004) and Yafrate & DeJong (2007) (see Table 2.2). The authors found that \( \mu \) values for \( q_{T-\text{bar}} \) and \( q_{\text{ball}} \) lie between 0.10 and 0.21 using a similar expression to that of Equation (2-11).

Lehane et al. (2009) examined a range of variables that affect the rate dependence of penetration resistance in kaolin, using T-bar and ball penetrometers. The variables included penetration velocity, diameter of the penetrometers, coefficient of consolidation, effective stress level and overconsolidation ratio (OCR<5). The authors captured rate dependency on penetration resistance presuming viscous effects are present irrespective of the drainage conditions (see Figure 2.12). The following relationship was proposed by combining the viscous term to the inviscid component, defined as:

\[
q/\sigma_{v_0} = \left[ a + \frac{b}{1 + cV} \right] \left[ \frac{v/d}{(v/d)_{\text{ref}}} \right]^m \tag{2-12}
\]

where \( m \) is the rate exponent, and viscous effects are presumed to occur when \( v/d \) exceeds \((v/d)_{\text{ref}}\).
Silva (2005) conducted variable rate piezocone penetration tests on dilative silts in the centrifuge facility. The soils prepared were silica flour with a relatively low permeability of about $6 \times 10^{-8}$ m/s (measured in the triaxial cell). The authors used two different sample preparation methods to create dense (vibro-compacted) silt with a void ratio of about 0.73 and loose (sedimented) silt with a void ratio of up to 0.84.

Penetration test profiles at rates varying from 0.1 to 4.0 mm/s in the dense silts are shown in Figure 2.13. The observed excess pore pressures ($\Delta u$) were negative due to the dilatancy of the silica flour and greater negative $\Delta u$ values were obtained as the penetration velocity was increased. Over the same velocity range, $q_t$ and $f_s$ values increased, which suggests that the drained resistance was lower than the partially drained resistance. In fact, the $q_t$ measured at $v=4.0$ mm/s was approximately twice the $q_t$ at $v=0.1$ mm/s (see Figure 2.13).

Schnaid et al. (2004) argued that undrained penetration occurs in silty soils when the $B_q$ value is between 0.3 and 0.5. The $B_q$ value of 0.3 seems to give a lower boundary, below which partial drainage is likely to prevail. Based on the tests in the dilative silts, Silva (2005) commented that $B_q$ values were approximately zero due to dilative behaviour, indicating that $B_q$ is not the ideal parameter to assess drainage conditions for these soils. Similar comments on the use of $B_q$ to define drainage conditions can be found in Tonni & Gottardi (2009); see Table 2.2.
Jaeger et al. (2010) reported a series of variable rate cone penetration tests in a sand-clay mixture in the centrifuge facility (see Figure 2.14). The mixture comprised 25% kaolin and 75% sand with a plasticity index (PI) of 9.5% and critical state friction angle ($\phi'_{cs}$) of 28°. Penetration rate effects in the mixture were evident and the drained to undrained net cone resistance ratio was approximately 17, which is much higher than that observed in kaolin clay. The authors also commented that drained conditions occur at $V$, defined according to Equation (2-8), less than about 0.01 while undrained conditions exist when $V$ is greater than about 20.
A similar (but independent) study is reported by Kim et al. (2008) and Kim et al. (2010), who conducted variable penetration rate tests in a calibration chamber using a mixture comprising 25% kaolin and 75% Jumun sand. The authors also observed the full range of drainage conditions and reported that the drained to undrained resistance ratio (using $q_t$) was in the order of 3 to 4. When comparing the two independent studies involving 25% kaolin and 75% sand, Schneider (2010) concluded that the results were fairly comparable except for the very low values of normalised resistances ($Q_t$) recorded in the centrifuge by Jaeger et al. (2010).

General comment on laboratory experiment investigations

While penetration rate effects in kaolin have been investigated extensively in the centrifuge, the number of centrifuge investigations in other soil types is very limited. One measure of the rate dependence of penetration resistance is the drained to undrained resistance ratio and available evidence indicates this ratio varies widely. The normalised velocity ($V$) defined as Equation (2-8) was found to be useful to characterise drainage conditions. DeJong & Randolph (2012) and DeJong et al. (2012) collated several studies of penetrometer rate effects and concluded that undrained conditions generally exist at $V>30$ and drained conditions exist at $V<0.3$. The general applicability of this range in $V$ needs to be determined for a range of soil types including in-situ soils and is one of the primary aims of this thesis.
Table 2.3 Summary of data related to penetration rate effects in centrifuge

<table>
<thead>
<tr>
<th>References</th>
<th>Model (gravity field)</th>
<th>Probe</th>
<th>Measure probe</th>
<th>Characteristics of soil</th>
<th>Penetration rate, v (mm/s)</th>
<th>Main observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corte et al. (1991)</td>
<td>beam (6 g)</td>
<td>cone</td>
<td>q, d=20mm</td>
<td>dry and saturated</td>
<td>0.55 - 0.92 (dry),</td>
<td>- the slower speeds show slightly higher q, than the faster speeds in both dry and saturated condition</td>
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<td>Fontainebleau sand</td>
<td>0.63 - 2.17 (saturated)</td>
<td>- it is stated that other test results using fine sand did not show noticeable rate effects ranging from 0.5 to 10 mm/s</td>
</tr>
<tr>
<td>Renzi et al. (1994)</td>
<td>beam (100 g)</td>
<td>piezocone</td>
<td>q, u2, d=11.3mm</td>
<td>Pontida silty clay</td>
<td>2-60</td>
<td>- both q, and u2 increase with increasing penetration rate</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td>- q, increases by 15 % at rates from 2 to 60 mm/s</td>
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<tr>
<td>Finnie &amp; Randolph (1993),</td>
<td>beam (100 g)</td>
<td>circular</td>
<td>q, bearing pressure</td>
<td>model sand and model silt</td>
<td>0.1-3</td>
<td>- non-dimensional bearing modulus was examined under drained to undrained conditions, which were defined as V= vd/c,</td>
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<tr>
<td>Erbrich (2005)</td>
<td></td>
<td>foundations of prototype diameter l(cone) to 15m</td>
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<td>- as V becomes high, reduction in bearing pressure for the silt is much more dramatic than reduction for the sand</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>- transition from undrained to partially drained condition occurs at V<del>30; transition from partially drained to fully drained conditions at V</del>0.01</td>
</tr>
<tr>
<td>Van der Poel &amp; Schenkeveld (1998)</td>
<td>beam (35.6 g)</td>
<td>cone</td>
<td>q, d=11.2mm</td>
<td>Eastern Scheldt sand</td>
<td>1-250</td>
<td>- the influence of penetration velocity is smaller than the accuracy and therefore no effects of penetration velocity on cone resistance is concluded</td>
</tr>
<tr>
<td>House et al. (2001)</td>
<td>drum (100 g)</td>
<td>T-bar</td>
<td>q1-bar, d=5mm</td>
<td>kaolin clay</td>
<td>0.0059-3</td>
<td>- the bearing resistance becomes a minimum at a particular rate due to partial consolidation and viscous effects</td>
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<td></td>
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<td>- 'twitch' test was proposed, and by which c, may be estimated</td>
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<td>- 'twitch' test is when the penetration rate is successively halved with penetration advancement of 1 or 2 diameters at each stage</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- the transition from undrained to partially drained conditions occurs at V~10</td>
</tr>
<tr>
<td>Randolph &amp; Hope (2004)</td>
<td>beam (100 g)</td>
<td>piezocone</td>
<td>q2, u2, q7r, d=10mm, T-bar</td>
<td>kaolin clay</td>
<td>0.005-3</td>
<td>- according to the B, for piezocone, and T-bar penetration resistance, transition from undrained to partially drained condition occurs at V<del>30 and 10 respectively, and transition from partially drained to fully drained occurs at V</del>0.3 and 0.1 respectively</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td>- the effects of partial consolidation are much sharper for the T-bar compared to that for the piezocone</td>
</tr>
<tr>
<td>Silva &amp; Bolton (2004)</td>
<td>beam (50 g)</td>
<td>piezocone</td>
<td>q, q2, d=12mm</td>
<td>three-layered silica sands</td>
<td>0.4-8</td>
<td>- penetration rates appear to have little influence on both q, and u2 in fine sand with a pore fluid of which is 50 times more viscous than water</td>
</tr>
<tr>
<td></td>
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<td></td>
<td>- negative Δω was observed, indicating dilative tendency of the silica flour</td>
</tr>
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<td></td>
<td>- negative Δω increases with increasing penetration rate</td>
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<td></td>
<td></td>
<td>- q, increases with increasing penetration rate</td>
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<td></td>
<td></td>
<td>- q, at the maximum velocity was about twice the q, at the minimum velocity in dense silts</td>
</tr>
</tbody>
</table>

27
<table>
<thead>
<tr>
<th>References</th>
<th>Model (gravity field)</th>
<th>Probe</th>
<th>Measure</th>
<th>Characteristics of soil</th>
<th>Penetration rate, $v$ (mm/s)</th>
<th>Main observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chung (2005), Chung et al. (2006)</td>
<td>beam (100 g)</td>
<td>cone</td>
<td>q, tip</td>
<td>Burswood clay</td>
<td>0.00078 (twitch)</td>
<td>- 'twitch' tests were carried out with different shape penetrometers</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=10mm,</td>
<td></td>
<td></td>
<td></td>
<td>- all penetrometers show partial consolidation effects as end resistances were increased up to about twice of reference (undrained) end resistances</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ball d=11.9mm,</td>
<td></td>
<td></td>
<td></td>
<td>- an equivalent area diameter, $d_e$, is suggested to use for a normalised velocity</td>
</tr>
<tr>
<td></td>
<td></td>
<td>plate d=11.2mm,</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>T-bar d=5mm</td>
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<tr>
<td></td>
<td></td>
<td>with different aspect ratios</td>
<td></td>
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</tr>
<tr>
<td>Schneider et al. (2007)</td>
<td>beam (160, 40, 30, 100 g)</td>
<td>piezocone</td>
<td>q, $u_2$</td>
<td>NC kaolin (160 g), LOC 95% silica flour + 5% bentonite (100 g), HOC 95% silica flour + 5% bentonite (30 g)</td>
<td>0.0004-3</td>
<td>- the transition from undrained to partially drained conditions occurs at $V$=100; fully drained condition was not achieved until $V$&lt;0.04 although $u_2$ was almost zero at $V$~0.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=10mm</td>
<td></td>
<td></td>
<td></td>
<td>- the resistance reduces by a factor of 2 at V from 0.04 to 100 in NC kaolin; a rapid reduction in the resistance by a factor of over 4 at V from 0.3 to 3 occurs in LOC SFB</td>
</tr>
<tr>
<td>Lehane et al. (2009)</td>
<td>drum (150, 75, 30 g)</td>
<td>T-bar (drum)</td>
<td>$q_{T-bar}, q_{ball}$</td>
<td>kaolin with OCR = 1, 2, 5</td>
<td>0.003-100</td>
<td>- viscosity is presumed to affect the penetration resistance irrespective of drainage conditions</td>
</tr>
<tr>
<td></td>
<td>beam (100, 50 g)</td>
<td>d=3 to 7mm,</td>
<td></td>
<td></td>
<td></td>
<td>- variables that affect penetrometer resistance were changed, that include penetration velocity, probe diameter, coefficient of consolidation, effective stress and OCR</td>
</tr>
<tr>
<td></td>
<td></td>
<td>T-bar (beam)</td>
<td></td>
<td></td>
<td></td>
<td>- the T-bar resistance ratios of the minimum $q_{T-bar}$ to the drained $q_{T-bar}$ vary with OCR; the ratio $~4$ for OCR=1 and the ratio $~2.5$ for OCR=5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=3.5 to 10mm,</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>ball (drum)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=8,12mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jaeger et al. (2010)</td>
<td>beam (100g)</td>
<td>piezocone</td>
<td>q, $u_2$</td>
<td>Mixture of 25% kaolin and 75% sand</td>
<td>0.003-3</td>
<td>- drained condition appears to occur at $V$&lt;0.01; undrained condition exist for $V$&gt;20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=10mm</td>
<td></td>
<td></td>
<td></td>
<td>- the ratio of drained to undrained resistance is approximately 17</td>
</tr>
<tr>
<td>Mahmoodzadeh et al. (2011)</td>
<td>beam (100 g)</td>
<td>piezoball</td>
<td>$q_{ball}, u_2$</td>
<td>carbonate muddy silt</td>
<td>0.003-2.5</td>
<td>- the transition from undrained to partially drained conditions occurs at V=50; transition from partially drained to fully drained conditions occurs at V=0.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=15mm</td>
<td></td>
<td></td>
<td></td>
<td>- the ratio of drained to undrained (using piezoball) of about 3.3 is reported</td>
</tr>
<tr>
<td>Oliveira et al. (2011)</td>
<td>drum (50 g)</td>
<td>cone</td>
<td>$q_c$</td>
<td>silty tailings</td>
<td>0.008-23.333</td>
<td>- the transition from undrained to partially drained conditions occurs at V=75; transition from partially drained to fully drained conditions occurs at V=1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=9mm</td>
<td></td>
<td></td>
<td></td>
<td>- the ratio of drained to undrained of about 3.4 is reported</td>
</tr>
<tr>
<td>Cassidy (2012)</td>
<td>drum (100 g)</td>
<td>T-bar</td>
<td>$q$</td>
<td>calcareous silt</td>
<td>0.015625-1 (twitch)</td>
<td>- the transition from undrained to partially drained conditions occurs at V~10 for both T-bar and spudcan penetrations using an area equivalent diameter in V</td>
</tr>
<tr>
<td></td>
<td></td>
<td>d=5mm, spudcan</td>
<td></td>
<td></td>
<td></td>
<td>- the ratio of maximum to undrained resistance reached $~16$</td>
</tr>
</tbody>
</table>
Table 2.4 Summary of data related to penetration rate effects under normal gravity

<table>
<thead>
<tr>
<th>References</th>
<th>Model</th>
<th>Prob</th>
<th>Measurement</th>
<th>Characteristics of soil</th>
<th>Penetration rate, v (mm/s)</th>
<th>Main observations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dayal &amp; Allen (1975)</td>
<td>cylindrical steel molds of</td>
<td>cone</td>
<td>q_c, f_s</td>
<td>pottery clay (clayey silt) and silica sand</td>
<td>1.3-811.4</td>
<td>- clay: q_c and f_s increase with increasing penetration rate</td>
</tr>
<tr>
<td></td>
<td>46cm diameter and 61cm height</td>
<td>d=35.6mm</td>
<td></td>
<td></td>
<td></td>
<td>- a linear relationship was found between q_c/q&lt;sub&gt;c&lt;/sub&gt; and v/v&lt;sub&gt;c&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- sand: insignificant rate effects were found on q_c and f_s</td>
</tr>
<tr>
<td>Luger et al. (1982)</td>
<td>cone (α&lt;sub&gt;c&lt;/sub&gt;=90°)</td>
<td>q</td>
<td></td>
<td>Clay (35 to 240 kPa)</td>
<td>1-2000</td>
<td>- through over 130 tests in clays, influence of penetration rate was found at v</td>
</tr>
<tr>
<td></td>
<td>d=25mm with d=20mm shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>greater than 20 mm/s</td>
</tr>
<tr>
<td>Te Kamp (1982)</td>
<td>0.25m diameter Rowe</td>
<td>cone</td>
<td>q&lt;sub&gt;c&lt;/sub&gt;</td>
<td>North Sea sand</td>
<td>0.2-20</td>
<td>- the resistance linearly increases with v&lt;sup&gt;0.2&lt;/sup&gt;</td>
</tr>
<tr>
<td></td>
<td>consolidation cell</td>
<td>d=7.95 mm</td>
<td></td>
<td>(OCR = 1 and 8.4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Almeida &amp; Parry (1985)</td>
<td>850 mm diameter tub</td>
<td>cone</td>
<td>q&lt;sub&gt;c&lt;/sub&gt;, u&lt;sub&gt;1&lt;/sub&gt;, u&lt;sub&gt;2&lt;/sub&gt;</td>
<td>kaolin, Gault clay</td>
<td>0.2-20</td>
<td>- possible effect of penetration rate was not proven with the applied scale model</td>
</tr>
<tr>
<td></td>
<td>d=10mm, piezocone d=12.7mm</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Kim (2004), Kim &amp; Tumay (2007)</td>
<td>calibration chamber (specimen size 525 mm in diameter and 815 mm in height)</td>
<td>piezocone d=11.3mm</td>
<td>q&lt;sub&gt;c&lt;/sub&gt;, u&lt;sub&gt;1&lt;/sub&gt;, u&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Mixture of 33% kaolin and 67% fine sand (OCR=1 and -10)</td>
<td>3, 6 (20 refer to Lim (1999))</td>
<td>- corrected net cone resistance at higher penetration rate is greater</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>- q&lt;sub&gt;mc&lt;/sub&gt; increases by 10% at v from 3 to 6 mm/s; q&lt;sub&gt;nc&lt;/sub&gt; increases by 8% at v from 6 to 20 mm/s</td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>- both δu&lt;sub&gt;1&lt;/sub&gt; and δu&lt;sub&gt;2&lt;/sub&gt; similarly increase with an increase in penetration speed</td>
</tr>
<tr>
<td>Kim et al. (2008), Kim et al. (2010)</td>
<td>calibration chamber (specimen size 1.2 m in diameter and 1 m in height)</td>
<td>piezocone d=11.3mm</td>
<td>q&lt;sub&gt;c&lt;/sub&gt;, u&lt;sub&gt;2&lt;/sub&gt;</td>
<td>Mixture of 25 % kaolin and 75 % Jumun sand, Mixture of 18 % kaolin and 82 % Jumun sand</td>
<td>0.01-20</td>
<td>- drained to partially drained to undrained conditions were observed with this range of penetration speeds</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>- the ratio of drained to undrained cone resistance was of the order of 3-4 using q&lt;sub&gt;c&lt;/sub&gt;/q&lt;sub&gt;nc&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- the transition from undrained to partially drained conditions occurs at V of 1-10; transition from partially drained to fully drained conditions at V of 0.05</td>
</tr>
</tbody>
</table>
2.3 ANALYSIS OF CONE RESISTANCE

2.3.1 Cavity expansion method

The spherical and cylindrical cavity expansion (CE) methods have been widely used for various geotechnical problems including cone penetration and bearing capacity of deep foundations such as pile foundation. A full description of the cavity expansion method and its applications is published in Yu (2000). Spherical cavity expansion has often been used to estimate the cone/pile end resistance (e.g. Vesic 1972, Ladanyi & Johnston 1974, Randolph et al. 1994) as schematically illustrated in Figure 2.15. Cylindrical cavity expansion has been used to assist interpretation of pile shaft resistance (e.g. Randolph & Wroth 1979), pressuremeter (e.g. Yu 1993) and was also extended to estimate tip resistance (e.g. Salgado et al. 1997). A summary of the CE analytical solutions for CPT in both cohesive and cohesionless soils is presented in Yu & Mitchell (1998), who also reviewed other analytical approaches of cone penetration such as bearing capacity theory, strain path method, incremental finite element method and use of calibration chambers.

Figure 2.15 Schematic illustrations of spherical cavity expansion and tip resistance: (a) Ladanyi & Johnston (1974) and (b) Randolph et al. (1994)

According to the cavity expansion method, soil deformation around the cone/pile tip is simplified by modelling an expansion of the spherical cavity. As the cavity expands, a pressure inside the cavity eventually reaches a limit value so-called a limit pressure.
(\(p_{\text{limit}}\)). This is then related to a cone tip/pile end resistance \((q_c\) or \(q_b\)). The relationship between the spherical cavity expansion limit pressure \((p_{\text{limit}})\) and the cone tip resistance \((q_c)\) often assumes that the normal stress acting on the cone face is equal to \(p_{\text{limit}}\), as illustrated in Figure 2.15. By defining a cone apex angle to be \(\alpha_c\), the vertical equilibrium on the cone face is obtained as:

\[
q_c = p_{\text{limit}} + \frac{\tau}{\tan \left( \frac{\alpha_c}{2} \right)}
\]  

(2-13)

where \(p_{\text{limit}}\) and \(\tau\) are the normal and shear limit stress acting on the cone face. For a typical cone penetrometer, \(\alpha_c\) is 60°, i.e. \(1/\tan(\alpha_c/2) = \sqrt{3}\).

For undrained expansion, the cohesion \((c')\) is set equal to the undrained shear strength \((s_u)\) and the friction angle is set to zero so that \(\tau = s_u\). Ladanyi & Johnston (1974) deduced the following form for undrained expansion from Equation (2-13).

\[
q_c = p_{\text{limit}} + \sqrt{3} s_u \alpha_{\text{int}}
\]  

(2-14)

where \(\alpha_{\text{int}}\) indicates the interface friction ratio between the cone and soil.

In sands (i.e. drained) with relatively small cohesion, Randolph et al. (1994) presented the following form:

\[
q_c = p_{\text{limit}} (1 + \tan \delta \tan \alpha_c)
\]  

(2-15)

where \(\delta\) is the interface friction angle between the cone and soil, which is taken equal to the soil friction angle \((\phi')\), and \(\alpha_c\) equals to 60° for the typical cone (but can be taken as \((45+\phi/2)\) for the soil wedge under a flat pile).

### 2.3.2 Spherical cavity expansion in PLAXIS

Previous studies have successfully modelled spherical cavity expansion in the finite element code PLAXIS (e.g. Xu & Lehane 2008, Tolooiyian & Gavin 2011, Suzuki 2010, Suryasentana & Lehane 2014). The applicability of the spherical cavity expansion analysis in PLAXIS was confirmed based on the comparisons of the computed limit pressure to the closed-form solutions. In drained analysis, Xu & Lehane (2008) compared their analyses with the closed-form solution of Yu & Houlsby (1991) developed with Mohr-Coulomb model. In undrained analysis, Suzuki (2010) compared
the analyses with the closed-form solution for Modified Cam Clay model presented by Cao et al. (2001). Furthermore, Suryasentana & Lehane (2014) verifies the validity of using the spherical cavity expansion to predict CPT \( q_c \) values by comparison with field CPT data in sand. These previous studies indicate that there is potential for the spherical cavity expansion method in PLAXIS to be further extended to model variable rate cone penetration test using a coupled consolidation analysis; this is the subject of Chapter 7.

### 2.3.3 Modelling variable rate CPT using cavity expansion

A relationship between cavity expansion \( p_{\text{limit}} \) and CPT \( q_c \) is required for partially drained conditions i.e. the existing undrained and drained formulations are insufficient. Silva (2005) investigated cone penetration rate effects by modelling a cylindrical cavity expansion at variable expansion rates using the CAMFE FE code, which was originally developed by Carter (1978). The author expresses the shear stress \( \tau \) as a function of the effective normal stress \( \sigma'_n \):\

\[
\tau = c' + \sigma'_n \tan \phi' = c' + (\sigma_n - u) \tan \delta
\]  

(2-16)

where \( c' \) is the effective cohesion, \( \sigma_n \) is the total normal stress, \( u \) is pore pressure and \( \delta \) is the interface friction angle between cone and soil. Assuming the limit pressure acts normally on the cone tip (i.e. \( \sigma_n = p_{\text{limit}} \)) with negligible effects of \( c' \), Equation (2-16) leads to the following relationship, which is proposed in Silva (2005):

\[
q_c = p_{\text{limit}} + \sqrt{3}(p_{\text{limit}} - u) \tan \delta
\]  

(2-17)

LeBlanc & Randolph (2008) and Jaeger (2012) performed the cylindrical cavity expansion analyses using the finite difference code FLAC (Itasca). LeBlanc & Randolph (2008) derived the following form to relate \( p_{\text{limit}} \) to \( q_c \) by assuming the vertical stress \( \sigma'_v \) and the radial stress \( \sigma'_r \) are the maximum and minimum principal stresses respectively.

\[
q_t = \sigma'_v + u
\]  

(2-18)

\[
\sigma'_v = \sigma'_r \frac{1 + \tan \delta / \tan \left( \frac{\alpha_c}{2} \right)}{1 - \tan \delta \tan \left( \frac{\alpha_c}{2} \right)}
\]  

(2-19)
where $\delta$ is the interface friction angle and $\alpha_c$ is the cone apex angle. LeBlanc & Randolph (2008) assumed $\sigma'_r$ is equal to $p_{\text{limit}}$ in the cylindrical CE analyses.

### 2.3.4 Previous studies on rate effects

Previous numerical analyses on penetrometer rate effects are reviewed in this section. Silva (2005) and Silva et al. (2006) compared their cylindrical cavity expansion results using Modified Cam Clay model in CAMFE with the centrifuge piezocone test results presented by Randolph & Hope (2004), as shown in Figure 2.16. The CEMFE analyses simulated the increase in net cone resistance $q_{\text{net}}$ and corresponding pore pressure $\Delta u$ reduction due to partial consolidation effects. It is seen that the cylindrical cavity expansion analyses predict a slower increase in $q_{\text{net}}$ as the degree of partially drainage increases compared to the experimental results. Silva (2005) suggested that the discrepancy may be partly related to the selected value of the coefficient of consolidation in the normalised velocity ($V$) term and a possible influence of the different drainage conditions occurring in the centrifuge model.

Jaeger (2012) compared their cylindrical cavity expansion analyses using the MIT-S1 model (Pestana & Whittle 1999) in FLAC with the variable CPTU data experimentally obtained on the sand-kaolin mixture in Jaeger et al. (2010); this comparison is shown in Figure 2.17. Jaeger (2012) demonstrated the potential of this approach for predicting drained and undrained penetration resistances, but as with the Silva (2005) numerical predictions, the analyses of Jaeger (2012) also predict a relatively a slow increase in cone resistance as the degree of partially drainage increases compared to the experimental results. Jaeger (2012) commented that part of the evident discrepancies are due to (i) the assumed relationship between the cylindrical cavity expansion limit pressure and cone penetration resistance and (ii) differences in drainage for the cylindrical cavity expansion and cone penetration. The cylindrical cavity expansion was modelled by a one-dimensional row using the axisymmetric geometry configuration, which may not be suitable to simulate a combination of vertical and horizontal drainage around the cone tip.
Figure 2.16 Comparison of the cylindrical cavity expansion results with the centrifuge results in kaolin (Silva 2005)
Figure 2.17 Comparison of the cylindrical cavity expansion results with the centrifuge results in the sand-kaolin mixture (Jaeger 2012)
Effects of the initial state parameter ($\psi_0$) (e.g. Been & Jefferies 1985) on penetration rate effects have been examined in cylindrical cavity expansion analyses. LeBlanc & Randolph (2008) modelled the silt soil behaviour using the critical state based soil model and presented effects of permeability and initial state parameter on the normalised effective cone penetration resistance (see Figure 2.18). The authors commented that the transition from undrained to drained behaviour tends to occur in the permeability range, $k=10^{-8}$ to $10^{-18}$ m/s in dense states and between $k$ values of $10^{-4}$ and $10^{-7}$ m/s in loose states.

![Figure 2.18 Effects of in situ state and permeability on penetration resistances (LeBlanc & Randolph 2008)](image)

Jaeger (2012) also examined effects of the initial state parameter ($\zeta_0$), (which is the initial void ratio minus the void ratio on the critical state line for triaxial compression) on drained and undrained penetration resistances in sands with up to 35% fines content (see Figure 2.19). It can be seen that soils with lower fines contents, which are likely to show relatively fast drainage characteristics, exhibit a strong dependence on the in situ state parameter in the undrained condition. Jaeger (2012) found that the drained to undrained resistance ratio could range from 0.3 to 11.5 based on a series of the cylindrical cavity expansion analyses.
A recent study by Yi et al. (2012) modelled variable cone penetration rate tests in fine-grained soils using a large deformation finite element analysis with a linear elastic perfectly plastic soil model. Effects of the modulus ratio \((G/p')\) and the friction angle \((\phi')\) on rate dependence were examined. The authors found that the drained to undrained cone resistance ratios increased with increasing modulus ratio but were relatively insensitive to friction angle. The numerical analyses provide a reasonable fit with the Randolph & Hope (2004) experimental data in kaolin for the \(G/p' = 35\) to 70 (see Figure 2.20).

Einav & Randolph (2005) and Zhou & Randolph (2009) have shown significant influences of strain rate dependency and strain-softening on the undrained penetration resistance of ‘full-flow’ (i.e. T-bar and ball) penetrometers. Although these factors tend to have compensating effects on the penetration resistance, the effects may result in widely varying resistance factors \((N)\) when compared to the plasticity based theoretical resistance factors e.g. see Randolph (2004). The effects of strain rate dependency and strain-softening on the CPT resistance in undrained conditions have been explored in a recent study reported by Zheng et al. (2014). The authors indicated that cone factors \((N_{kt})\) are highly sensitive to rate dependency and about 30% increase in \(N_{kt}\) can be found when certain degrees of strength degradation and rate dependency are incorporated.
General comment on numerical analysis investigations

Although numerical analyses have provided some very useful insights into penetration rate effects, there appears to be room for improvement in current methodologies to model penetration rate dependency using coupled-consolidation cavity expansion analyses. The factors affecting cone resistance and drainage conditions during penetration are investigated in this thesis in Chapter 7.

Cylindrical cavity expansion has been used to predict penetration resistance in many previous studies. Spherical cavity expansion is, however, considered more appropriate for examination of partial drainage effects as the dissipation around the tip of a cone can be taken as being approximately spherical. Therefore the analyses presented in Chapter 7 adopt spherical cavity expansion.
CHAPTER 3 FIELD TEST PROCEDURES AND SOIL PROPERTIES

3.1 INTRODUCTION

Full scale CPTs were performed in Gingin, Bassendean and Burswood in Western Australia and a full description of these tests is provided in Chapter 4. This Chapter presents the results from laboratory classification and element tests on the soils at these sites and also presents corresponding soil property data for reconstituted clayey sands which were investigated in laboratory CPTs (in the centrifuge and at 1g), the result for which are presented in Chapter 5 and 6. Before describing these studies, this Chapter firstly describes the equipment used for in situ penetration tests.

Some of the undrained simple shear test results presented in this Chapter, Section 3.6.4 have been published in Suzuki & Lehane (2014c); Appendix D.

3.2 UWA CPT APPRATUS

3.2.1 Piezocone, T-bar and Ball penetrometers

The piezocone penetrometer, which was locally fabricated and owned by UWA, was used for this project and is shown in Figure 3.1a and 3.1b. The piezocone penetrometer has a diameter of 35.7 mm and a projected end area \( A_p \) of 10 cm\(^2\). The penetrometer measures tip resistance \( q_c \), sleeve friction \( f_s \), and pore water pressure. Plastic porous filters were mostly used to measure pore pressures at the shoulder of the cone \( u_2 \) position). Temperature and inclination can also be measured inside the friction sleeve during testing.

Figure 3.1c and 3.1d show T-bar and ball penetrometers used at the Bassendean site. The T-bar probe consists of a cylindrical bar of 250 mm×40 mm in length and diameter, which provides an area of the penetrometer in the plane normal to shaft \( A_p \) of 100 cm\(^2\).
The ball probe has a diameter of 75 mm with $A_p$ of 44 cm$^2$. Both probes were used by replacing the piezocone conical tip and maintaining the CPT friction sleeve.

![Photographs of the penetrometers used: (a) and (b) piezocone, (c) T-bar and (d) ball penetrometer](image)

**3.2.2 Calibration details**

Two load cells to measure tip resistance and friction sleeve, and a pore pressure transducer were calibrated in the laboratory prior to use in the field. The load cells were calibrated separately against actual loads applied in staged increments. The pore pressure transducer was placed in water in the calibration chamber and then air pressure inside the chamber was increased with an external pressure gauge attached. Readings from the pore pressure transducer were calibrated against output from another pressure gauge attached to the chamber. An unequal area ratio ($\alpha$) effect (Lunne et al. 1997b) on the cone was measured in this manner. A displacement transducer was calibrated to be used for measurement of actual penetration and extraction movements. The calibration details are listed in Table 3.1.
It is important to carefully take into account temperature effects on the penetrometer measurements due to zero shifting of the strain gauges. Figure 3.2 shows calibration results obtained due to temperature changes while the piezocone was left in air in the laboratory (without any axial load applied) during a typical summer’s day in Perth. Recordings from the two load cells and the pore pressure transducer are plotted on Figure 3.2 against temperature change (expressed in volts) monitored by a sensor mounted in the penetrometer. Interestingly, linear relationships are observed for each sensor apart from when the output from the temperature sensor ranged between 1.15 and 1.2 volt, which requires a careful correction of strain gauge for temperature to process data. All of the field test results in this study were corrected for temperature effects based on simplified relations, such as shown by the black dotted lines in Figure 3.2.

![Figure 3.2 Temperature calibrations for the load cells and pore pressure transducer](image)

During a series of penetration tests using the built-in-house piezocone, it was found that friction sleeve readings were less accurate and thus less reliable than the two other
readings (i.e. tip resistance and pore pressure measurements). For this reason, the focus of this study is given to rate dependency of tip resistance and pore pressure responses.

### 3.2.3 Equipment

The lightweight CPT rig was manufactured at UWA specifically for the previous project at the Burswood site (Schneider et al. 2004). Figure 3.3 shows a photograph of the UWA CPT rig. The weights of the rig and the vehicle are 290 kg and 1350 kg respectively, which give a maximum reaction force to penetration of 1640 kg (equivalent to 16 kN). The rig uses a lead screw to drive down or up (i.e. penetration or extraction). The movement is controlled by a motor mounted on top of the entire CPT frame via a gearbox. A program called “Syncpos” is used to pre-set all controlling parameters such as starting and ending base points of travelling (i.e. displacement), acceleration, deceleration and velocity of penetration and extraction. A penetration and extraction speed could be varied within a range of 0.001 mm/s to 20 mm/s with the original gearbox. The author also arranged for use of another gearbox which allows a penetration rate five times slower to be achieved (i.e. 4 mm/s down to 0.0002 mm/s). The data recording system is called ‘Penetration Data Recorder (PDR)’ developed at UWA. The output data are recorded at 20 Hz and then averaged over 10 to 20 mm intervals for presentation purposes.

![Figure 3.3 Photograph of the UWA CPT rig at the Bassendean site](image-url)
Plastic filter elements were saturated with glycerine under vacuum suction in the laboratory. These were soaked in glycerine and transported to site in a sealed container to maintain saturation. Assembly of the porous filter and the cone tip used a funnel technique (see Figure 3.4a). An on-site saturation technique was employed for the second test series at Bassendean and the tests at Burswood, and involved use of a small vacuum chamber fabricated in house and application of suction after the filter and the tip were assembled (see Figure 3.4b and 3.4c).

It was necessary to dig a hole for some cases especially for the tests in summer because a hard crust had formed at ground surface. Boreholes were pre-drilled either by a hand auger and a scoop, a drilling rig or a dummy sharpen cone tip made in house (see Figure 3.4d). The depths of predrilled holes are indicated in the penetration profiles (Chapter 4).

Figure 3.4 Photographs of equipment used for field tests: (a) funnel, (b) vacuum system, (c) vacuum chamber and (d) dummy cone
3. Field test procedures and soil properties

### 3.2.4 Test procedure

Table 3.2 summarises the general testing procedure in an operation order. The numbers shown in this table site: ‘1’ for the Gingin site, ‘2’ for the Bassendean site Phase I, ‘3’ for the Bassendean site Phase II, and ‘4’ for the Burswood site, respectively.

<table>
<thead>
<tr>
<th>Testing step</th>
<th>Site 1</th>
<th>Site 2</th>
<th>Site 3</th>
<th>Site 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>• predrill by hand auger and scoop about 0.2 m</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• predrill by drilling rig down to about 3 m</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• move vehicle in position</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• attach four stands to vehicle by jacking up so no tyres touch ground</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• adjust levels for both longitudinal axis and transverse axis</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• repeat this procedure (if necessary) to conduct penetration perpendicularly into the aimed location</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• predrill by using dummy cone, driving in about 2.5 m</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• extract and remove dummy cone</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• assemble cone penetrometer with output cables through rods</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• assemble porous filter and conical tip using funnel method</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• use T-bar and Ball probes instead of cone probe (no saturation on site)</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• use vacuum chamber filled with glycerine and apply suction to improve adequate saturation</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• cover cone tip with membrane to keep saturation before penetration</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• clamp CPT rods with frame and lower cone tip to ground surface</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• prepare displacement transducer ready to be used and set zero for all recordings in PDR</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• commence recording, and drive down at specified penetration rate</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• add rod, release clamps, drive up only the moving frame, re-clamp the rods and drive down to reach aimed depth</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• halt penetration after passing through surface crust to stabilise temperature</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• stop for dissipation test</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• change penetration rate as planned for variable penetration rate tests</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• extract and remove all rods as per reverse of installation</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
<tr>
<td>• re-locate and resume next testing</td>
<td>v</td>
<td>v</td>
<td>v</td>
<td>v</td>
</tr>
</tbody>
</table>

All of the in situ tests at three sites are designated as follows:

- 1st (set) letter(s): G, indicating Gingin; Bass for Bassendean; Belm for Belmont park in Burswood
- 2nd letter: C, indicating Cone; T for T-bar; B for ball probe to be used
- 3rd letter: S, indicating standard test at 20 mm/s; D for dissipation test; R for rate test with a specified velocity/velocities
- 4th letter: T, indicating Test, and followed by number of the penetration test.
3.3 SOIL CONDITIONS AT GINGIN

3.3.1 Gingin site

The Gingin site is located at about 100 km north of Perth CBD in Western Australia (see Figure 3.5). The site was originally a mining site for heavy minerals such as ilmenite, rutile and zircon, operated by Iluka Resources Limited. The disposal method, which is referred to ‘Modified Co-disposal process’, was used to encourage fast drainage and to accelerate the water loss achieved using the traditional solar drying technique employed in Western Australia (Iluka Resources Limited 2009). A series of in situ tests were performed in this project at one of the backfilled man-made tailings dams where a depth of the dam was about 6 to 10 m according to the construction record. Figure 3.6 shows a photograph of the UWA CPT rig set-up at the testing location at the Gingin site.

Figure 3.7 shows an overview of the testing area at the Gingin site. There was a pond 30 m away from the testing area (as seen at the rear in the picture) observed in winter 2010. This pond was created as water was drained from slurry tailings deposits and stored in the dam. The pond was virtually non-existent in summer 2011 because of significant evaporation.
3. Field test procedures and soil properties

3.3.2 Field test programme

Eight piezocone penetration tests were performed at the Gingin site within the working time allowed on site in February 2011 (summer). These eight tests are summarised in Table 3.3. Testing locations listed in Table 3.3 correspond to the locations indicated by those symbols ‘a’ to ‘h’ in Figure 3.7. A distance of at least 80 cm (22d, where d is the diameter of the CPTU) was kept between each test to make sure no interaction between tests while trying to obtain similar stratigraphy. A starter pit, extending to a depth of...
~15-20 cm, was excavated within the shallow crust before penetration. The various penetration rates (20, 10, 2, 1, 0.4, 0.2, and 0.02 mm/s) were used at the depth of between 0.7 and 1.7 m, where soft deposits were found. The standard rate of 20 mm/s was used at all other depths. Dissipation tests to measure pore pressure dissipation were also conducted at 1.5 m depth, while the rods were clamped.

**Table 3.3 Summary of cone penetration tests at the Gingin site**

<table>
<thead>
<tr>
<th>ID</th>
<th>Date (2011)</th>
<th>Location</th>
<th>Penetration rate mm/s</th>
<th>Testing depth (m)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>GCDT1</td>
<td>31/ Jan</td>
<td>a</td>
<td>20</td>
<td>1.5</td>
<td>dissipation</td>
</tr>
<tr>
<td>GCRT1</td>
<td>03/ Feb</td>
<td>b</td>
<td>10</td>
<td>0.68-1.68</td>
<td></td>
</tr>
<tr>
<td>GCRT2</td>
<td>03/ Feb</td>
<td>c</td>
<td>0.2</td>
<td>0.66-1.66</td>
<td></td>
</tr>
<tr>
<td>GCRT3</td>
<td>03/ Feb</td>
<td>d</td>
<td>2</td>
<td>0.73-1.73</td>
<td></td>
</tr>
<tr>
<td>GCST1</td>
<td>04/ Feb</td>
<td>e</td>
<td>20</td>
<td>0-2.5</td>
<td></td>
</tr>
<tr>
<td>GCRT4</td>
<td>04/ Feb</td>
<td>f</td>
<td>0.4</td>
<td>0.67-1.67</td>
<td></td>
</tr>
<tr>
<td>GCRT5</td>
<td>04/ Feb</td>
<td>g</td>
<td>1</td>
<td>0.67-1.67</td>
<td></td>
</tr>
<tr>
<td>GCRT6</td>
<td>08/ Feb</td>
<td>h</td>
<td>0.02</td>
<td>0.96-1.16</td>
<td></td>
</tr>
</tbody>
</table>

After all CPTUs were completed, tailings samples were collected by a hand auger and a scoop at the depths between 0.7 and 1.5 m beside the CPTU tests area (see Figure 3.7). Laboratory classification and compression tests have been carried out on these disturbed materials, as described below.

### 3.3.3 Classification test

Soil classification tests including particle size distribution (PSD) and Atterberg limit determinations were performed on the Gingin disturbed samples. Figure 3.8 shows the PSD curves from sieve analyses and hydrometer tests based on Australian Standard (AS 1289.3.6.3 1994, AS 1289.3.6.1 1995). The Gingin samples contain ~79% sand sized particles, ~6% silt sized, and ~15% clay sized particles (i.e. the fines content of about 21%). Therefore, the material may be described as Clayey SAND.

Table 3.4 summarises test results of the natural water content ($w_c$) and Atterberg limits, which show a mean $w_c$, liquid limit (LL) and plastic limit (PL) of 30%, 28% and 18% respectively. These results correspond to a plasticity index (PI) of 10 and a liquidity index (LI) of 1.2. The Casagrande plasticity chart classifies the material as a low plasticity clay (CL).
3. Field test procedures and soil properties

![Graph showing particle size distribution](#)

Figure 3.8 Particle size distribution of the Gingin soil

Table 3.4 Atterberg limits of the Gingin soil

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$w_c$ (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>LI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0-1.5</td>
<td>31.1</td>
<td>28.5</td>
<td>17.1</td>
<td>11.4</td>
<td>1.2</td>
</tr>
<tr>
<td>1.0-1.5</td>
<td>28.1</td>
<td>27.1</td>
<td>17.1</td>
<td>10.1</td>
<td>1.1</td>
</tr>
<tr>
<td>1.0-1.25</td>
<td>30.7</td>
<td>28.4</td>
<td>18.4</td>
<td>10.0</td>
<td>1.2</td>
</tr>
<tr>
<td>Average</td>
<td>30</td>
<td>28</td>
<td>18</td>
<td>10</td>
<td>1.2</td>
</tr>
</tbody>
</table>

3.3.4 Compression test

Rowe cell consolidation test

Two Rowe cell 1-D compression (RCC) tests were carried out using disturbed samples collected from the depth intervals that the field tests focused on. The samples were mixed with distilled water until they became slurry paste, and then placed under vacuum suction to remove any potential air bubbles trapped inside the slurry samples. The Rowe cell test was conducted by applying a staged increment of the cell pressure under the back pressure of 200 kPa.

Figure 3.9a shows void ratio ($e$) changes against vertical effective stress ($\sigma'_v$); Figure 3.9b and 3.9c show variations against $\sigma'_v$ of vertical coefficient of consolidation ($c_v$) and vertical permeability ($k_v$), respectively. The vertical coefficient of consolidation ($c_v$) for each consolidation increment was deduced by comparing the observed settlement and the pore pressure dissipation at the base of the sample using Terzaghi’s one dimensional
consolidation theory (up to 90% consolidation). The vertical permeability \( (k_v) \) was obtained by directly measuring water flow at the end of each consolidation stage under application of a constant head difference of 10 or 20 kPa.

The following observations were made:

- The compression index \( (C_c) \) ranged from 0.17 to 0.21 (average \( \sim 0.19 \)) at \( \sigma'_v \) values greater than 20 kPa; higher \( C_c \) values are in evidence between 10 and 20 kPa.

- The recompression index \( (C_r) \) ranged from 0.021 to 0.047 (average \( \sim 0.034 \)). The variations in \( C_c \) and \( C_r \) are possibly due to slight variations in the selected sample compositions and also in the initial water contents in the slurry paste for each test as the final water contents measured for Sample 1 and Sample 2 were 20% and 22% respectively.

- Burland (1990) proposed the following expression for the slope of the intrinsic compression line (ICL) \( (C_c^*) \)

\[
C_c^* = 0.256e_L - 0.043
\]  

(3-1)

where \( e_L \) is the void ratio at the liquid limit \( (e_L=w_L G_s) \). The \( C_c^* \) value of 0.15 is calculated for the Gingin material according to Equation (3-1); this is a little lower than the average \( C_c^* \) obtained in RCC tests on reconstituted soil of 0.19.

- The vertical coefficient of consolidation \( (c_v) \) is relatively low, but generally increases from \( 1.8\times10^{-8} \) to \( 1.4\times10^{-6} \) m²/s (0.6-45 m²/year) as the \( \sigma'_v \) increases from 10 to 240 kPa.

- The vertical permeability \( (k_v) \) ranges from \( 1.9\times10^{-9} \) m/s to \( 5.3\times10^{-10} \) m/s at \( \sigma'_v \) values between 20-160 kPa and generally decreases as \( \sigma'_v \) increases.

- The \( c_v \) and \( k_v \) values indicate that the soil has a very low permeability even though the PSD shows a majority of the Gingin soil comprises sand particles and is classified as Clayey SAND. Both \( c_v \) and \( k_v \) are evidently strongly affected by the clay fraction (of \( \sim 15\% \)).
3. Field test procedures and soil properties

Figure 3.9 Two Rowe cell test results on the Gingin remoulded samples: (a) $e-\sigma'_v$, (b) $c_v-\sigma'_v$ and (c) $k_v-\sigma'_v$
3. Field test procedures and soil properties

3.4 SOIL CONDITIONS AT BASSENDEAN

3.4.1 Bassendean site

A site at Bassendean was the second location selected for in situ variable penetration rate tests; these tests were conducted in April, May and July 2011. The Bassendean site is on a river bank located about 12 km from Perth CBD as shown in Figure 3.5 and about 10 km from the Burswood site (discussed in Section 3.5). Figure 3.10 shows a local area map in the vicinity of the Bassendean site. Both the Bassendean site and the Burswood site are categorised as ‘flood plain and river terrace’ generalised geomorphology where an alluvial complex is expected to be encountered (Gozzard 2007). Gozzard (2007) states the formation developed during the Holocene Period after a dramatic sea level change.

The testing area is located at the rear of a private residential property, which slopes down to the Swan River flood plain (see Figure 3.11). The current land owners commented that the testing area has been raised by 300 to 400 mm in the late 1980s for residential purposes (Van Kleef 2011). The area was loaded by a heavy tracked vehicle for short periods during this time but no other additional significant loading has since been applied.

All of the field tests were conducted within the area of approximately 20 m by 4 m, as indicated by the red box in Figure 3.11. The ground in this area is fairly flat and about 15 m from the Swan River water front (seen in the rear of the picture). The testing layout is illustrated in Figure 3.12 and the reference point (0,0) is indicated in Figure 3.11. The minimum distance between testing locations was restricted to 1 m to avoid any interaction.

As the testing area is very close to the water front, natural maximum tidal variations of about one metre are likely to cause the water table to fluctuate between depths of 1 and 2 m. Occasional puddles occurred during heavy rain in July 2011. Tidal and seasonal variations will lead to a degree of overconsolidation of the soils closer to ground level. A relatively uniform clayey soil was found below the surface crust down to about 10 m depth.
3. Field test procedures and soil properties

Figure 3.10 Map of Bassendean testing area in Western Australia (Map data © 2013 Google)

Figure 3.11 Overview of the testing area at the Bassendean site

Figure 3.12 Testing layout at the Bassendean site
3.4.2 Field test programme

Two series of field tests were carried out at the Bassendean site. Phase I, details of which are summarised in Table 3.5, was performed between April and May 2011 (at the end of summer in Western Australia). Phase I aimed primarily at investigating the soil stratigraphy and homogeneity and employed CPTUs with the standard rate of 20 mm/s. Variable penetration rate tests with rates varying from 20 mm/s down to 0.002 mm/s were also conducted in the layer between 3 and 4 m. Dissipation tests were completed at several depths.

Phase II, details of which are summarised in Table 3.6, was conducted at the beginning of winter season in July 2011. This second series employed ‘twitch’ tests of the form proposed by House et al. (2001) whereby the velocity is reduced incrementally when a new ‘depth interval’ is reached. Velocities and depth intervals used in each twitch test are presented in the Chapter 4. A new gearbox also used in Phase II allowed investigation of penetration velocities of as low as 0.0002 mm/s. The T-bar and Ball probe were also used for further investigation into rate effects. The location of each penetration test as indicated in alphabetical order in Table 3.5 and Table 3.6 corresponds to the testing location illustrated in Figure 3.12.

Table 3.5 Summary of field tests at the Bassendean site for Phase I

<table>
<thead>
<tr>
<th>ID</th>
<th>Date (2011)</th>
<th>Location</th>
<th>Penetration rate (mm/s)</th>
<th>Testing depth (m)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>BassCRT1</td>
<td>14/ Apr</td>
<td>a</td>
<td>20</td>
<td>~12</td>
<td></td>
</tr>
<tr>
<td>BassCRT2</td>
<td>14/ Apr</td>
<td>b</td>
<td>20</td>
<td>~12</td>
<td></td>
</tr>
<tr>
<td>BassCRT3</td>
<td>14/ Apr</td>
<td>c</td>
<td>20</td>
<td>~12</td>
<td></td>
</tr>
<tr>
<td>BassCRT4</td>
<td>23-25/ Apr</td>
<td>e</td>
<td>0.002</td>
<td>3.1-3.5</td>
<td>recording issue</td>
</tr>
<tr>
<td>BassCRT5</td>
<td>25-27/ Apr</td>
<td>f</td>
<td>0.004</td>
<td>3.1-3.5</td>
<td></td>
</tr>
<tr>
<td>BassCRT6</td>
<td>27-28/ Apr</td>
<td>g</td>
<td>0.01</td>
<td>3.1-3.5</td>
<td></td>
</tr>
<tr>
<td>BassCRT7</td>
<td>28-29/ Apr</td>
<td>h</td>
<td>0.02</td>
<td>3-4</td>
<td></td>
</tr>
<tr>
<td>BassCRT8</td>
<td>29/ Apr</td>
<td>i</td>
<td>2</td>
<td>3-4</td>
<td></td>
</tr>
<tr>
<td>BassCRT9</td>
<td>29-30/ Apr</td>
<td>j</td>
<td>0.02</td>
<td>3-4, 4-5</td>
<td></td>
</tr>
<tr>
<td>BassCRT10</td>
<td>03/ May</td>
<td>k</td>
<td>0.2</td>
<td>3-4</td>
<td></td>
</tr>
</tbody>
</table>
Table 3.6 Summary of field tests at the Bassendean site for Phase II

<table>
<thead>
<tr>
<th>ID</th>
<th>Date (2011)</th>
<th>Location</th>
<th>Penetration rate (mm/s)</th>
<th>Testing depth (m)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>BassCRT11</td>
<td>01/ Jul</td>
<td>m</td>
<td>20</td>
<td>~12</td>
<td></td>
</tr>
<tr>
<td>BassCRT12</td>
<td>02/ Jul</td>
<td>n</td>
<td>twitch</td>
<td>6-7</td>
<td></td>
</tr>
<tr>
<td>BassCDT2</td>
<td></td>
<td></td>
<td></td>
<td>4.5,6</td>
<td>dissipation</td>
</tr>
<tr>
<td>BassCRT13</td>
<td>03-06/ Jul</td>
<td>o</td>
<td>twitch</td>
<td>4-5</td>
<td></td>
</tr>
<tr>
<td>BassCDT3</td>
<td></td>
<td></td>
<td></td>
<td>4.5,6,7,8,9</td>
<td>dissipation</td>
</tr>
<tr>
<td>BassCRT14</td>
<td>06-10/ Jul</td>
<td>p</td>
<td>twitch</td>
<td>4-5</td>
<td></td>
</tr>
<tr>
<td>BassCRT15</td>
<td>10-12/ Jul</td>
<td>q</td>
<td>twitch</td>
<td>5-6</td>
<td></td>
</tr>
<tr>
<td>BassCRT16</td>
<td>12/ Jul</td>
<td>r</td>
<td>twitch</td>
<td>7.3-8.3</td>
<td></td>
</tr>
<tr>
<td>BassCRT17</td>
<td>12/ Jul</td>
<td>s</td>
<td>twitch</td>
<td>~11</td>
<td></td>
</tr>
<tr>
<td>BassTRT1</td>
<td>13/ Jul</td>
<td>t</td>
<td>20</td>
<td>~9</td>
<td>T-bar</td>
</tr>
<tr>
<td>BassTRT2</td>
<td>13-15/ Jul</td>
<td>u</td>
<td>twitch</td>
<td>3-4</td>
<td>T-bar</td>
</tr>
<tr>
<td>BassBRT1</td>
<td>16/ Jul</td>
<td>v</td>
<td>20</td>
<td>~9</td>
<td>Ball</td>
</tr>
<tr>
<td>BassBRT2</td>
<td>16-18/ Jul</td>
<td>w</td>
<td>twitch</td>
<td>3-4</td>
<td>Ball</td>
</tr>
<tr>
<td>BassCRT18</td>
<td>23-31/ Jul</td>
<td>x</td>
<td>twitch</td>
<td>3-3.5</td>
<td>new gearbox</td>
</tr>
</tbody>
</table>

The soil samples were collected using a steel tube pushed into the ground with the UWA CPT rig at the locations of ‘l’ and ‘y’ as indicated in Figure 3.12. This sampling method is likely to have some degree of disturbance to the samples. These disturbed materials were used for soil classification tests and Rowe cell consolidation tests on reconstituted samples. With the aid of Hagstrom Drilling, the hydraulic piston tube samples and the mechanically pushed steel tube samples were later obtained from the locations of ‘z’ in Figure 3.12. The piston tubes had a thin-wall (1.5mm) and an inside diameter of 60 mm, which gives an outside diameter to a wall thickness ratio \(d_o/t\) of 42, which is slightly less than the recommendation by Ladd & DeGroot (2003), where a \(d_o=76\) mm with \(d_o/t>45\) is suggested. The piston samples were used for the laboratory element tests as described below. Before extruding the samples, all the piston tube samples were examined in an X-ray machine to identify any potential cracks and shells.

3.4.3 Classification test

Figure 3.13 shows PSD curves of the Bassendean soils. The Bassendean samples contain 36 to 46% sand sized particles and 54 to 64% fines content including 7 to 15% clay fraction. Therefore, the material may be described as sandy and clayey SILT.
Table 3.7 summarises the Atterberg limits and the natural water content ($w_c$) results. The cone liquid limit (LL) and the plastic limit (PL) ranged from 42 to 55% and 21 to 22% respectively. The results suggest this material may be classified as an intermediate plasticity clay (CI) on the Casagrande plasticity chart. The $w_c$ values of 66 to 86% were higher than the LLs, resulting in liquidity indices (LI) ranging from 1.4 to 2.2.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$w_c$ (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>LI</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0-7.0</td>
<td>66.4</td>
<td>41.9</td>
<td>20.8</td>
<td>21.1</td>
<td>2.2</td>
<td>Phase I</td>
</tr>
<tr>
<td>3.1-3.25</td>
<td>68.0</td>
<td>55.4</td>
<td>22.8</td>
<td>32.6</td>
<td>1.4</td>
<td>Phase II</td>
</tr>
<tr>
<td>3.5-3.7</td>
<td>85.7</td>
<td>55.1</td>
<td>22.4</td>
<td>32.8</td>
<td>1.9</td>
<td>Phase II</td>
</tr>
</tbody>
</table>

### 3.4.4 Compression test

**Constant rate of strain consolidation test (Intact samples)**

Four constant rate of strain consolidation tests (CRSC) were performed on the piston samples. The inside diameter of the piston sampler was 70 mm and therefore a smaller cutting ring (with inside diameter of 43 mm and height of 24 mm) was used for the CRSC tests. The sample and ring were placed inside the CRSC apparatus and a piston shaft, which incorporated a load cell, was then placed on top of the sample. A linear variable differential transformer (LVDT) was used to measure settlement in a vertical direction.
The CRSC is controlled by a displacement rate, which ideally leads to a pore pressure ratio ($\Delta u_b/\sigma_v$) between 3 and 15%, where $\Delta u_b$ is the excess pore pressure measured at the sample base and $\sigma_v$ is the vertical total stress (ASTM D4186 2006). A displacement rate of 0.004 mm/min (equivalent to a strain rate of ~1%/h) was used for all the tests. This rate generated $\Delta u_b/\sigma_v$ ratios less than 10%, except for an extremely low $\Delta u_b/\sigma_v$ ratio obtained on the CRSBass2 specimen.

The change in void ratio ($e$) at any stage of compression was calculated based on the sample settlement measured by the LVDT. The average vertical effective stress ($\sigma'_v$) in the sample was estimated by accounting for non-uniformity of pore pressure distribution in the sample (i.e. the pore pressure is zero at the top of the sample) and determined after Wissa et al. (1971) as:

$$\sigma'_v = \sigma_v - \frac{2}{3} \Delta u_b$$  \hspace{1cm} (3-2)

where $\sigma_v$ is the total vertical stress measured at the top of the sample and $\Delta u_b$ is the pore pressure measured at the sample base. A vertical coefficient of consolidation ($c_v$) was then estimated according to Wissa et al. (1971):

$$c_v = \frac{H^2 \Delta \sigma'_v}{2 u_b \Delta t}$$  \hspace{1cm} (3-3)

where $H$ is the sample height, $\Delta \sigma'_v/\Delta t$ is the current rate of effective stress increase.

Figure 3.14 shows the CRSC test results on the Bassendean soil including void ratio versus vertical effective stress and vertical coefficient of consolidation versus vertical effective stress. Table 3.8 summaries all the CRSC test results. The vertical yield stress ($\sigma'_{vy}$) was determined using the Casagrande method. The compression index ($C_c$) was calculated as $\Delta e/\Delta \log \sigma'_v$. The maximum values ($C_{c,max}$) and the $C_c$ at three times $\sigma'_{vy}$ are summarised in Table 3.8. The recompression index ($C_r$) was taken as an average slope of the $e$-$\log \sigma'_v$ at the stresses between $\sigma'_{v0}$ and $\sigma'_{vy}$. The sample qualities indicated at least “good to fair” according to the $\Delta e/e_0$ method proposed by Lunne et al. (1997a), where $\Delta e$ is the change in void ratio when the sample is reconsolidated back to in situ stress relative to the initial void ratio ($e_0$).
The following observations are made:

- The Bassendean samples are very compressible except for one sample from 3.27 m (CRSBass2). The maximum compression index ($C_{c,\text{max}}$) ranged from 1.18 to 1.55 while the $C_c$ value at the $\sigma'_v=3\sigma'_{vy}$ ranges from 0.53 to 1.07. This difference reflects non-linearity in the $e$-$\log\sigma'_v$ relationship and highlights the influence of intact soil structure (Burland 1990).
- The $C_r$ ranged from 0.35 to 0.65 in the three samples with high compressibility.
- The result of CRSBass2 shown much lower compressibility with the $C_c$ of 0.49 and the $C_r$ of 0.04. It was observed that this sample contained a far greater proportion of coarse particles than typically observed.
- The yield stress ratio ($\text{YSR} = \sigma'_{vy}/\sigma'_{v0}$) ranged from 1.2 to 2.5, indicating the soil on site was lightly overconsolidated at the (shallow) depths investigated.
- The vertical coefficient of consolidation ($c_v$) generally decreased with an increase in $\sigma'_v$, but tended to stabilise or increase slightly in the higher stress (normally consolidated) region.
- The $c_v$ at the $\sigma'_{v0}$ and at the $\sigma'_{vy}$ ranged from 39 to 79 m$^2$/year ($1.2\times10^{-6}$ to $2.5\times10^{-6}$ m$^2$/s) and from 10 to 40 m$^2$/year ($3.2\times10^{-7}$ to $1.3\times10^{-6}$ m$^2$/s), respectively i.e. $c_v$ at the $\sigma'_{v0}$ tends to be 2 to 4.8 times higher than that at the $\sigma'_{vy}$.
- Note that the test on the 3.27 m specimen (CRSBass2) did not achieve the required $\Delta u_i/\sigma_i$ ratios and therefore an estimation of its $c_v$ value was not made.

Figure 3.14 CRSC test results on the intact Bassendean soil: (a) $e$-$\sigma'_v$ and (b) $c_v$-$\sigma'_v$. 

(a) void ratio - vertical effective stress 
(b) coeff. of consolidation - vertical effective stress
3. Field test procedures and soil properties

Table 3.8 Summary of CRSC test results on the intact Bassendean soil

<table>
<thead>
<tr>
<th>Test name</th>
<th>Depth (m)</th>
<th>( w_i ) (%)</th>
<th>( \sigma'_{v0} ) (kPa)</th>
<th>( \Delta e/e_0 )</th>
<th>( \sigma'_{vy} ) (kPa)</th>
<th>YSR</th>
<th>( C_{c,max} )</th>
<th>( C_r ) at ( 3\sigma'_{vy} )</th>
<th>( c_v ) at ( \sigma'_{v0} ) (m²/yr)</th>
<th>( c_v ) at ( \sigma'_{vy} ) (m²/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRSBass1</td>
<td>2.44</td>
<td>85.0</td>
<td>23.5</td>
<td>0.024</td>
<td>59.0</td>
<td>2.51</td>
<td>1.384</td>
<td>1.291</td>
<td>0.413</td>
<td>49.1</td>
</tr>
<tr>
<td>CRSBass2</td>
<td>3.27</td>
<td>43.8</td>
<td>27.8</td>
<td>0.045</td>
<td>40.4</td>
<td>1.45</td>
<td>0.496</td>
<td>0.493</td>
<td>0.039</td>
<td>N/A*</td>
</tr>
<tr>
<td>CRSBass3</td>
<td>3.43</td>
<td>85.7</td>
<td>28.6</td>
<td>0.007</td>
<td>33.5</td>
<td>1.17</td>
<td>1.183</td>
<td>1.065</td>
<td>0.648</td>
<td>79.1</td>
</tr>
<tr>
<td>CRSBass4</td>
<td>4.43</td>
<td>79.9</td>
<td>33.8</td>
<td>0.025</td>
<td>53.0</td>
<td>1.57</td>
<td>1.348</td>
<td>0.986</td>
<td>0.349</td>
<td>38.9</td>
</tr>
</tbody>
</table>

where \( w_i \) : initial water content

*The required pore pressure ratio (\( \Delta u_b/\sigma_v \)) not achieved

Rowe cell consolidation test (Reconstituted samples)

Two Rowe cell 1-D consolidation (RCC) tests were conducted on reconstituted Bassendean samples. The bulk samples collected from the site were remoulded in a mixer with distilled water to become slurry paste and then slowly poured into the Rowe cell apparatus. The interpretation of these tests followed that described for the Gingin soil in Section 3.3.4. Figure 3.15 shows the observed (a) \( e \)-log\( \sigma'_v \) relationships, (b) variations of \( c_v \) with \( \sigma'_v \) and (c) variations of \( k_v \) with \( \sigma'_v \), respectively.

The following observations can be made:

- The compression index (\( C_c \)) ranged from about 0.40 to 0.43 (average ~0.415) at the \( \sigma'_v >50 \) kPa. The recompression index (\( C_r \)) ranged from 0.047-0.058 (average ~0.053), for which the final water contents measured for Sample 1 and Sample 2 were 32% and 28% respectively.
- The \( C_c \) obtained in RCC was about 20% higher than the \( C_c* \) obtained using the liquid limit following as Equation (3-1). The \( C_{c,max} \) obtained on intact samples in CRSC were about three times higher than the \( C_c \) on remoulded samples in RCC. This suggests a significant influence of structure on the compressibility of the in-situ soil.
- The (normally consolidated) vertical coefficient of consolidation (\( c_{v,NC} \)) varied between 0.4 and 1.3 m²/year (1.2×10⁻⁸ and 4.1×10⁻⁸ m²/s) and increased slightly with an increase in \( \sigma'_v \).
- The \( c_{v,NC} \) values were two orders of magnitude lower than the intact \( c_v \) values at the in situ vertical effective stress (\( \sigma'_{v0} \)) and about 20 to 50 times lower than the intact \( c_v \) values at \( \sigma'_{vy} \) (see Figure 3.15b).
• $k_v$ values in the reconstituted Bassendean soils were very low, and generally decreased from $1.7 \times 10^{-9}$ to $1.4 \times 10^{-10}$ m/s as $\sigma_v'$ increased from 20 to 160 kPa.

![Graph showing void ratio vs. vertical effective stress](image1)

(a) void ratio - vertical effective stress

![Graph showing coefficient of consolidation vs. vertical effective stress](image2)

(b) coeff. of consolidation - vertical effective stress

![Graph showing permeability vs. vertical effective stress](image3)

(c) permeability - vertical effective stress

Figure 3.15 Rowe cell test results on two reconstituted Bassendean samples: (a) $e$-$\sigma_v'$, (b) $c_v$-$\sigma_v'$ and (c) $k_v$-$\sigma_v'$. 
3. Field test procedures and soil properties

3.4.5 Strength tests

CAU triaxial compression test

Anisotropically consolidated undrained (CAU) triaxial compression tests were performed on the Bassendean piston samples. The samples were trimmed to a diameter of 60 mm and length of 125 mm. Bender elements were attached to the top and bottom platens of the sample. Bender elements provide a useful and well established way to measure shear stiffness at very small strains in the laboratory (e.g. Atkinson 2000).

The specimen was firstly saturated under a backpressure of 500 kPa with a cell pressure of 510 kPa. A B-value calculated as $B = \Delta u / \Delta \sigma_3$ (i.e. an increase in the pore pressure divided by an increase in the cell pressure) was checked before consolidation by increasing cell pressure by 100 kPa. A minimum B-value of 0.96 was achieved. The sample was then anisotropically consolidated with a stress ratio ($K = \sigma'_h / \sigma'_v$) of ~0.7. The qualities of the samples were evaluated as “good to fair” according to the $\Delta e / e_0$ method proposed by Lunne et al. (1997a).

After consolidation was completed, a shear wave velocity ($V_s$) was measured using the bender elements with an electrical frequency of 4 Hz and 1.5 Hz for the specimens at 3.18 m (TXBAS01) and 5.39 m (TXBAS03), respectively. The shear modulus at very small strain ($G_0$) can be calculated as:

$$G_0 = \rho V_s^2$$  \hspace{1cm} (3-4)

where $\rho$ is the density of the soil and $V_s$ is the shear wave velocity, where the sample height (i.e. travelling distance) was divided by the time for the wave to travel. The travelling time was calculated as the time from a peak of the input sinusoidal wave to an initial major peak of the received wave (Lohani et al. 1999).

Additionally, the vertical permeability ($k_v$) was directly measured prior to shearing by applying a constant head difference of 10 kPa between the top and base of the sample. Loading was commenced under undrained condition at a constant displacement rate of 0.1 mm/min (i.e. a strain rate, $\dot{\varepsilon}_a$ of ~4.9-5.0 %/h).

Figure 3.16 summarise three CAU test results on the Bassendean soils. A diagonal failure place after shearing was observed in Figure 3.16a. Interestingly, there was a different coloured very sandy layer with a thickness of about 20 mm observed after the
Field test procedures and soil properties

A field test on the 3.18 m specimen was completed (see Figure 3.16b). Small shell fragments were occasionally evident in the 4.33 and 5.39 m specimens when the samples were cut after the tests. Figure 3.16c plots the measured deviator stress ($q$) versus axial strain ($\varepsilon_a$) for loading stage; Figure 3.16d shows stress paths in $q$-$p'$ space ($p'$ is the mean effective stress); and Figure 3.16e shows the normalised shear modulus degradation ($G/p'_0$) with axial strain, which includes the bender element data. Table 3.9 summarises all the triaxial test results on the Bassendean samples.

The following observations are made:

- Failure, defined at the peak deviator stress ($q$), occurred at the axial strain ($\varepsilon_a$) of about 2 to 5% and then softening behaviour was observed after the failure.

- The undrained shear strength ($s_u$) taken as a half of the peak $q$, varied from 17.4 to 18.6 kPa at the vertical consolidation effective stress ($\sigma'_{vc}$) of 30.1 to 36.1 kPa, resulting in the undrained strength ratio ($s_u/\sigma'_{vc}$) of between 0.52 and 0.61.

- The strain rate of ~5%/h used in this study is a little faster than the more typical strain rate of 1%/h employed in research quality undrained element tests. The effect of shear strain rate may be expressed in the form of:

$$\frac{s_u}{s_{u,\text{ref}}} = 1 + \mu \log \frac{\dot{\varepsilon}_a}{\dot{\varepsilon}_{a,\text{ref}}} \quad (3-5)$$

where $s_{u,\text{ref}}$ is the reference strength, $\dot{\varepsilon}_{a,\text{ref}}$ is the reference strain rate, and $\mu$ is the rate parameter. $s_u$ often increases ~10% per log cycle increase in $\dot{\varepsilon}_a$ used in laboratory element test (e.g. Kulhawy & Mayne 1990). Assuming $\mu=0.10$ with the $\dot{\varepsilon}_{a,\text{ref}}$ of 1%/h implies a modified $s_u/\sigma'_{vc}$ ratio in the range of between 0.46 and 0.55.

- The undrained strength ratio ($s_u/\sigma'_{vc}$) in clay commonly seen to be a related to the overconsolidation ratio (OCR) as follows:

$$\left(\frac{s_u}{\sigma'_{vc}}\right)_{\text{OC}} = \left(\frac{s_u}{\sigma'_{vc}}\right)_{\text{NC}} \cdot OCR^m \quad (3-6)$$

with $m=0.8$, although Ladd et al. (1977) suggest $m$ decreases from 0.85-0.75 with increasing OCR. The undrained strength ratio of 0.30 is typically measured on normally consolidated clays in CAU compression tests. Assuming
(s_u/σ'_{vc})_{NC}=0.3 implies an OCR range of 1.7 to 2.1. This range is higher than the YSRs of 1.2 to 1.6 inferred from the CRSC tests at the similar depths.

- Assuming c'=0, peak effective friction angles (ϕ_p) are estimated to be between 39° and 44°. These angles are somewhat higher than normally expected in natural clays, although a similarly high ϕ_p value has been reported on Burswood clay (Low et al. 2011) located 10 km downstream on the banks of the Swan River. The high friction angles may be partly due to the existence of shell fragments and influence by microstructure of the soils.

- The small strain G_0/p'_0 ratios (measured by the bender elements) varied between 300 and 600 and tended to decrease with depth.

- The mobilised secant shear moduli (G) calculated from the triaxial data were scattered at the small strain region (ε_a < 0.05%) but suggested the G at ε_a=0.1% were only about 0.33G_0, while G at a strain of 50% of the failure shear stress were about 0.2G_0.

- The vertical permeability (k_v) of intact samples is about 10 times higher than the permeability measured on the reconstituted normally consolidated Bassendean materials in RCC at same stress levels.

### Table 3.9 Summary of CAU triaxial tests for the Bassendean soil

<table>
<thead>
<tr>
<th>Test name</th>
<th>Depth (m)</th>
<th>w_i (%)</th>
<th>Δε/ε_0</th>
<th>σ'_{vc} (kPa)</th>
<th>σ'_{hc} (kPa)</th>
<th>k_v (m/s)</th>
<th>G_0 (MPa)</th>
<th>G_50 (MPa)</th>
<th>ε_a (%)</th>
<th>Δu_f (kPa)</th>
<th>p'_f (kPa)</th>
<th>s_u (kPa)</th>
<th>s_u* (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TXBAS01</td>
<td>3.18</td>
<td>67.9</td>
<td>0.029</td>
<td>30.1</td>
<td>20.5</td>
<td>1.6E-8</td>
<td>14.1</td>
<td>2.4</td>
<td>5.2</td>
<td>12.5</td>
<td>19.7</td>
<td>18.5</td>
<td>16.6</td>
</tr>
<tr>
<td>TXBAS02</td>
<td>4.33</td>
<td>72.4</td>
<td>0.037</td>
<td>33.6</td>
<td>23.7</td>
<td>7.7E-9</td>
<td>N/A</td>
<td>2.2</td>
<td>3.2</td>
<td>17.7</td>
<td>20.7</td>
<td>17.4</td>
<td>15.6</td>
</tr>
<tr>
<td>TXBAS03</td>
<td>5.39</td>
<td>67.3</td>
<td>0.039</td>
<td>36.1</td>
<td>23.8</td>
<td>3.4E-9</td>
<td>8.0</td>
<td>1.5</td>
<td>3.6</td>
<td>15.8</td>
<td>22.9</td>
<td>18.6</td>
<td>16.7</td>
</tr>
</tbody>
</table>

where
- σ'_{vc} : vertical effective stress after consolidation
- σ'_{hc} : horizontal effective stress after consolidation (K~0.7)
- k_v : vertical permeability after consolidation prior to shearing
- G_0 : very small strain shear modulus by bender element test prior to shearing
- G_50 : secant shear modulus at 50% of failure shear stress
- ε_a : axial strain at failure
- Δu_f : excess pore pressure at failure
- p'_f : mean effective stress at failure
- s_u : undrained shear strength measured (=q_f/2)
- q_f : peak deviator stress at failure
- s_u* : undrained shear strength corrected to axial strain rate of 1%/h
3. Field test procedures and soil properties

Figure 3.16 CAU triaxial test results for the Bassendean soil: (a) photograph of 5.39m specimen, (b) photograph of 3.18m specimen, (c) $q$-$\varepsilon_a$, (d) $q$-$p'$ and (e) $G/p'$-$\varepsilon_a$
3.5  SOIL CONDITIONS AT BURSWOOD

3.5.1  Burswood site

The Burswood site, in Western Australia was also selected for further in situ variable rate penetration tests. The site is located on the bank of the Swan River only a few kilometres from the centre of Perth and about 10 km downstream from the Bassendean site (see Figure 3.5). The Burswood site is believed to have a similar geology to the Bassendean site. Figure 3.17 shows a map of the Burswood area.

A number of researchers at the UWA have been involved in geotechnical site characterisation studies at Burswood since 1980. Low et al. (2011) summarise the results of two decades of site investigations, which included in situ penetration tests and laboratory soil element tests. Burswood clay is reported as being a very lightly overconsolidated sensitive silty clay with a high to extremely high plasticity index. Specifically, in situ penetration tests using different penetrometers have recently been conducted by Schneider et al. (2004) and Chung (2005). Schneider et al. (2004) conducted variable rate penetration tests in Burswood clay, using piezocones, T-bars and ball penetrometers over a velocity range between 0.01 and 20 mm/s. These authors observed viscous rate effects in undrained region but found the penetration resistance at 0.01 mm/s to be almost the same as that at the standard rate of 20 mm/s. The testing area previously being investigated was about 500 m away from the Burswood test area employed for this research (see Figure 3.17).

The aim of this project was to determine the rate dependence of penetrometer resistance over a much wider range of penetrometer velocities. This task was undertaken in January and February 2012. An overview of the testing area is shown in Figure 3.18. The ground surface was neatly mowed and the penetrometers were installed into 2.5 to 3 m predrilled boreholes. The testing locations of four tests were at a separation distance of 1m along a single line, with the anticipation being that ground conditions encountered in all tests would be closely comparable.

The water table at the test location was found at 1.3 m below the ground surface (as determined in the boreholes). Seasonal water table level changes and tidal effects may cause a light overconsolidation of the surface soil layer, noting that distance from the testing area to the waterfront was about 100 m.
3. Field test procedures and soil properties

3.5.2 Field test programme

Four cone penetration tests (CPTU) were performed at the Burswood site in January to February 2012, as summarised in Table 3.10. The standard cone penetration test at 20 mm/s was firstly completed, and then various penetration rates down to extremely slow velocity of 0.0002 mm/s were investigated. The specific velocities and travelling distances adopted in variable rate tests will be explained and discussed in Chapter 4. Dissipation tests were also completed to examine in situ horizontal coefficients of consolidation.
3. Field test procedures and soil properties

Table 3.10 Summary of field tests at the Burswood site

<table>
<thead>
<tr>
<th>ID</th>
<th>Date (2012)</th>
<th>Location</th>
<th>Various rate (mm/s)</th>
<th>Testing depth (m)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>BelmCRT1</td>
<td>24/Jan</td>
<td>a</td>
<td>20 twitch</td>
<td>~10</td>
<td></td>
</tr>
<tr>
<td>BelmCRT3</td>
<td>30/Jan-07/Feb</td>
<td>c</td>
<td>20 twitch</td>
<td>6-7</td>
<td>New gear box</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>7-8</td>
<td></td>
</tr>
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<td></td>
<td></td>
<td>0.2</td>
<td>8-9</td>
<td></td>
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<td>BelmCDT1</td>
<td>07-09/Feb</td>
<td>d</td>
<td>20</td>
<td>4,5,6,7,8,9,10,11</td>
<td>dissipation</td>
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</tbody>
</table>

Four piston tube samples for further investigation in the laboratory were provided by Golder Associates Pty. Ltd., (who had been managing a large site investigation in the area at the same time when the CPTUs were conducted in this study). The location of the borehole was within 5 m from the CPTU locations and so the similar stratigraphy can be expected. The thin-walled (1.5mm) piston sample tube was 450 mm long with an inside diameter of 60 mm, giving an outside diameter to a wall thickness ratio \(d/t\) of 42, which is slightly less than the recommendation by Ladd & DeGroot (2003) but is the same size as those used in the Bassendean site.

### 3.5.3 Classification test

Figure 3.19 shows three particle size distribution (PSD) test results. The Burswood soils consist of 8 to 14% sand sized particles (i.e. 86 to 92% fines content) with clay fraction content of 13 to 19%. The soil can be described as clayey SILT or silty CLAY and contains a higher proportion of fines than the Bassendean soil.

Atterberg limits test results are summarised in Table 3.11. The natural water contents \(w_c\), the cone liquid limits (LL) and the plastic limits (PL) were in the range 55 to 89%, 61 to 97% and 26 to 37%, respectively and tend to decrease with depth. The soil classifies as a high to very high plasticity clay (CH/CV) on the Casagrande plasticity chart. The classification test results for this study show a good agreement with the previous studies on the Burswood clay reported by Low et al. (2011). The plasticity of the Burswood soil is higher than that of the Bassendean soil, which is accord with its higher proportion of fines.
Field test procedures and soil properties

Figure 3.19 Particle size distribution of the Burswood soil

Table 3.11 Atterberg limits of the Burswood soil

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>(w_c) (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>LI</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.47-6.61</td>
<td>84.3</td>
<td>96.9</td>
<td>37.1</td>
<td>59.8</td>
<td>0.8</td>
</tr>
<tr>
<td>6.73-6.90</td>
<td>88.6</td>
<td>91.7</td>
<td>35.9</td>
<td>55.8</td>
<td>0.9</td>
</tr>
<tr>
<td>7.32-7.48</td>
<td>66.4</td>
<td>72.6</td>
<td>29.3</td>
<td>43.3</td>
<td>0.9</td>
</tr>
<tr>
<td>10.51-10.65</td>
<td>54.5</td>
<td>60.7</td>
<td>26.3</td>
<td>34.4</td>
<td>0.8</td>
</tr>
</tbody>
</table>

3.5.4 Compression test

Constant rate of strain consolidation test

Five constant rate of strain consolidation tests (CRSC) were performed on the Burswood piston samples. The displacement rate employed of 0.004 mm/min (i.e. the strain rate of \(~1%/h\)) resulted in pore pressure ratios \((\Delta u_b/\sigma_v)\) of 15% to 20% during the tests. The interpretation of these tests followed that described for the Bassendean soil in Section 3.4.4. Figure 3.20 plots the CRSC test results and Table 3.12 summarises soil properties obtained in these tests. The sample qualities were evaluated to be generally ‘good to fair’ except for the CRSBel5 test at 10.49m (i.e. ‘very poor’) based on the void ratio criteria \((\Delta e/e_0)\) proposed by Lunne et al. (1997a).

The following observations are made:

- The maximum inferred yield stress ratio (YSR) was 1.6 and there is an approximate tendency for this ratio to reduce with depth.
- The maximum compression index \( (C_{c,\text{max}}) \) varied between 0.57 and 1.13 whereas the \( C_c \) at \( 3\sigma'_{vy} \) was smaller and ranged from 0.57 to 1.12; compression indices show a tendency to decrease with depth.
- Apart from scatter (most likely due to unstable \( \Delta u \) measurements at low stresses), the coefficient of consolidation \( (c_v) \) decreased with an increase in \( \sigma'_v \). Sharpest drops in \( c_v \) occur around the value \( \sigma'_{vy} \) and stabilise at 0.6 to 0.8 \( m^2/\text{year} \) at \( \sigma'_v > 200 \text{kPa} \). The \( c_v \) values at \( \sigma'_{v0} \) and at \( \sigma'_{vy} \) ranged from 1.7 to 8.8 \( m^2/\text{year} \) and 1.5 to 8.9 \( m^2/\text{year} \), respectively.

![Figure 3.20 CRSC test results for the Burswood soil: (a) void ratio - vertical effective stress and (b) coeff. of consolidation - vertical effective stress](image-url)

Table 3.12 Summary of CRSC tests for the Burswood soil

<table>
<thead>
<tr>
<th>Test name</th>
<th>Depth (m)</th>
<th>( w_i ) (%)</th>
<th>( \sigma'_{v0} ) (kPa)</th>
<th>( \Delta e/e_0 )</th>
<th>( \sigma'_{vy} ) (kPa)</th>
<th>YSR</th>
<th>( C_{c,\text{max}} )</th>
<th>( C_v ) at ( 3\sigma'_{vy} )</th>
<th>( C_r )</th>
<th>( C_v ) at ( \sigma'_{v0} ) ( (m^2/\text{year}) )</th>
<th>( C_v ) at ( \sigma'_{vy} ) ( (m^2/\text{year}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRSBel1</td>
<td>5.75</td>
<td>87.1</td>
<td>38.8</td>
<td>0.061</td>
<td>62.0</td>
<td>1.60</td>
<td>1.129</td>
<td>1.123</td>
<td>0.524</td>
<td>2.3</td>
<td>1.5</td>
</tr>
<tr>
<td>CRSBel2</td>
<td>6.08</td>
<td>82.0</td>
<td>40.3</td>
<td>0.048</td>
<td>49.0</td>
<td>1.22</td>
<td>1.081</td>
<td>1.015</td>
<td>0.633</td>
<td>6.5</td>
<td>4.9</td>
</tr>
<tr>
<td>CRSBel3</td>
<td>6.71</td>
<td>78.0</td>
<td>43.2</td>
<td>0.059</td>
<td>43.2</td>
<td>1.00</td>
<td>1.003</td>
<td>0.966</td>
<td>0.669</td>
<td>8.8</td>
<td>8.9</td>
</tr>
<tr>
<td>CRSBel4</td>
<td>7.30</td>
<td>72.2</td>
<td>45.9</td>
<td>0.047</td>
<td>60.3</td>
<td>1.31</td>
<td>0.918</td>
<td>0.775</td>
<td>0.388</td>
<td>3.0</td>
<td>2.5</td>
</tr>
<tr>
<td>CRSBel5</td>
<td>10.49</td>
<td>55.8</td>
<td>62.1</td>
<td>0.143</td>
<td>65.0</td>
<td>1.05</td>
<td>0.570</td>
<td>0.570</td>
<td>0.461</td>
<td>1.7</td>
<td>1.7</td>
</tr>
</tbody>
</table>
3.5.5 Strength test

CAU triaxial test

Six triaxial compression tests (CAU) with a stress ratio ($K$) of ~0.7-0.8 were conducted on the Burswood piston samples. The qualities of the specimens ranged from “very good to excellent” to “good to fair” except for one “poor” specimen at 10.58 m, based on the $\Delta e/e_0$ criteria proposed by Lunne et al. (1997a). A displacement rate of 0.1 mm/min (i.e. a strain rate of ~4.9 to 5.0 %/h) was used for the undrained shearing stage, except for one specimen (TXBEL03 at 6.81 m depth) which employed a slower rate of 0.005 mm/min.

Figure 3.21 presents all of the CAU triaxial test results for the Burswood soils. Small shell fragments and large shells (up to 15mm in size) were occasionally encountered when the specimen was cut after the CAU test was completed (see Figure 3.21a). Figure 3.21 presents (b) $q$-$e_\Delta$ curves for the undrained loading phase, (c) stress paths in $p'$-$q$ space, and (d) $G/p'_0$ degradation curves. All of the CAU triaxial test results are summarised in Table 3.13.

The following observations can be made:

- The peak deviator stress ($q$) occurred at $e_\Delta$ of 2.6 to 6.2% after which softening was observed.
- The undrained shear strength ($s_u$) generally increases with vertical consolidation stress ($\sigma'_vc$) and varies from 24 to 36 kPa at $\sigma'_vc$ values between 37 and 67 kPa. These results correspond to an undrained strength ratio ($s_u/\sigma'_vc$) range of 0.51 to 0.67, except for TXBEL03 (at 6.81 m depth).
- The strain rate effect was examined using the strain rate of 0.25%/h for TXBEL03 (20 times slower than others). The $s_u$ value obtained was about 20% lower than the $s_u$ values at the strain rate of 5%/h at the similar $\sigma'_vc$.
- As discussed in Section 3.4.5, $s_u$ may be modified to a value corresponding to a more typical stain rate of 1%/h assuming the rate parameter ($\mu$) in Equation (3-5). The modified $s_u$ value, referred to as $s_u^*$, ranges from 22 to 33 kPa and gives undrained strength ratios of between 0.47 and 0.62. Using Equation (3-6) with $m=0.8$ implies a corresponding OCR range of between 1.8 and 2.5. This
OCR range is higher than the YSR range of 1.0 to 1.6 inferred from CRSC tests on samples from the similar depths.

- A peak effective friction angle ($\phi'_p$) of 33° to 40° (average of 38°) is indicated by the effective stress paths assuming $c'=0$. This range is very similar to that measured in a previous study on Burswood clay (Low et al. 2011).

- The secant shear modulus ratio at very small strains ($G_0/p'_0$) measured by bender elements ranges from 290 and 570, and tends to increase with depth. The secant shear modulus ($G$) calculated from triaxial compression at $\varepsilon_a = 0.1\%$ is only 0.25 to 0.33 of the $G_0$ values.

- The measured vertical permeabilities ($k_v$) are low and range from $1.9 \times 10^{-9}$ to $9.4 \times 10^{-9}$ m/s.

Table 3.13 Summary of CAU compression triaxial tests on the Burswood soil

<table>
<thead>
<tr>
<th>Test name</th>
<th>Depth (m)</th>
<th>$w_i$ (%)</th>
<th>$\Delta e/e_0$</th>
<th>$\sigma_{vc}'$ (kPa)</th>
<th>$\sigma_{hc}'$ (kPa)</th>
<th>$k_v$ (m/s)</th>
<th>$G_0$ (MPa)</th>
<th>$G_{10}$ (MPa)</th>
<th>$e_d$ (%)</th>
<th>$\Delta u_l$ (kPa)</th>
<th>$p'_f$ (kPa)</th>
<th>$s_u$ (kPa)</th>
<th>$s_u^*$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TXBEL01</td>
<td>5.86</td>
<td>86.7</td>
<td>0.033</td>
<td>36.8</td>
<td>29.0</td>
<td>3.7E-9</td>
<td>9.2</td>
<td>1.4</td>
<td>2.6</td>
<td>17.8</td>
<td>29.5</td>
<td>24.5</td>
<td>22.8</td>
</tr>
<tr>
<td>TXBEL02</td>
<td>6.54</td>
<td>84.3</td>
<td>0.069</td>
<td>47.5</td>
<td>29.4</td>
<td>N/A</td>
<td>9.2</td>
<td>0.9</td>
<td>6.2</td>
<td>15.0</td>
<td>32.9</td>
<td>26.2</td>
<td>24.3</td>
</tr>
<tr>
<td>TXBEL03</td>
<td>6.81*</td>
<td>88.5</td>
<td>0.052</td>
<td>46.2</td>
<td>30.8</td>
<td>N/A</td>
<td>10.3</td>
<td>1.1</td>
<td>4.4</td>
<td>12.2</td>
<td>32.2</td>
<td>20.6</td>
<td>21.9</td>
</tr>
<tr>
<td>TXBEL04</td>
<td>7.40</td>
<td>66.4</td>
<td>0.033</td>
<td>47.3</td>
<td>33.5</td>
<td>2.1E-9</td>
<td>17.7</td>
<td>2.4</td>
<td>4.0</td>
<td>18.5</td>
<td>33.5</td>
<td>25.5</td>
<td>23.8</td>
</tr>
<tr>
<td>TXBEL05</td>
<td>10.58</td>
<td>54.2</td>
<td>0.101</td>
<td>66.9</td>
<td>49.6</td>
<td>9.4E-9</td>
<td>27.4</td>
<td>4.6</td>
<td>4.7</td>
<td>31.5</td>
<td>44.7</td>
<td>33.9</td>
<td>31.5</td>
</tr>
<tr>
<td>TXBEL06</td>
<td>10.72</td>
<td>30.8</td>
<td>0.068</td>
<td>62.3</td>
<td>42.9</td>
<td>1.9E-9</td>
<td>28.2</td>
<td>4.9</td>
<td>3.4</td>
<td>23.7</td>
<td>45.5</td>
<td>35.6</td>
<td>33.1</td>
</tr>
</tbody>
</table>

* For TXBEL03 at 6.81m, a displacement rate was 20 times slower than other tests (Symbols refer to Table 3.9)
Figure 3.21 CAU triaxial test results for the Burswood soil: (a) photograph of 10.72m specimen, (b) $q-\varepsilon_a$, (c) $q-p'$ and (d) $G/p'_0-\varepsilon_a$.
3.6 SOIL PROPERTIES ON CLAYEY SANDS

3.6.1 Clayey sands

A series of classification and element tests was conducted on reconstituted clayey sands to assist interpretation of penetration tests performed in the same materials in the centrifuge and at 1g (see Chapters 5 and 6). Fines content, especially clay fraction, has significant effects on the degree of drainage, compressibility and strength of the soil (e.g. Leroueil & Hight 2003). This impact varies depending on the amount of clay fines in the host sands. Clayey sand mixtures prepared in the laboratory were examined in a series of soil classification tests, Rowe cell compression tests and simple shear strength tests as presented below.

3.6.2 Classification test

The clayey sands mixtures investigated comprise commercially available fine silica sands (with $D_{50}=0.19$ mm) with varying proportions of kaolin clays. The specific mixtures focused on in this project were those containing 5%, 10% and 25% (by dry weight) of kaolin soil mixed with 95%, 90% and 75% of dry sand. The mixtures are referred to as 5%, 10% and 25% kaolin mixtures in the following sections.

Figure 3.22 shows particle size distribution (PSD) curves for the three clayey sand mixtures as well as those for the constituent kaolin soil and clean fine sand available at the UWA. Fines contents (particle size <0.075 mm) and clay fractions (particle size <0.002 mm) are provided in percentages in the figure. Because of the small silt content of the kaolin soil, the 5%, 10% and 25% kaolin mixtures contain clay fractions of 4%, 8% and 20% respectively.

Table 3.14 summarises soil indices for the three clayey sand mixtures, fine sand and pure kaolin soil. The specific gravity ($G_s$) for each mixture was calculated based on the $G_s$ values for the fine sand and kaolin soil. Maximum and minimum void ratios ($e_{\text{max}}$ and $e_{\text{min}}$) were determined according to AS 1289.5.5.1 (1998) and the results are summarised in Table 3.14. These limiting void ratios are often useful to describe the relative density of a soil ($D_r$). However, there were difficulties associated with losing fines material, especially for testing the $e_{\text{min}}$. The standard recommendation in AS 1289.5.5.1 (1998) states that soils containing fines (i.e. particle size of <75 μm) of up to
5% by dry mass can be tested using the AS test method, and also silty sands with non-plastic fines up to 12% are also tested following the AS standard. On the other hand, the ASTM standards (ASTM D4253 2006, ASTM D4254 2006) suggest that their methods may be applicable to soils with fines up to 15% by dry mass. In fact, it was very difficult to continue these tests, in the presence of water for the $e_{\text{min}}$ test on the soils with the higher clay fractions, especially for the 25% kaolin mixture due to the loss of fine material. Figure 3.23a shows the $e_{\text{max}}$ and $e_{\text{min}}$ test results on the three clayey sands as well as fine sand. The $e_{\text{max}}$ values tend to increase with increasing clay fractions. On the other hand, $e_{\text{min}}$ decreased slightly with increasing clay fraction (up to 8%) when compared to clean sand.

Table 3.14 summarises liquid (LL) and plastic limits (PL) determined for the three clayey sand mixtures and the kaolin soil. The Australian Standards require soils with particle size of <425 μm to conduct these tests but the clayey sand mixtures were used without dry sieving process to remove very small portion (<1.7%) of particles greater in size than 425 μm. The test results are also plotted on the plasticity chart in Figure 3.23b. With a large amount of fine sand particles, the LLs drop and lead to the low plasticity of the clayey sands. The three clayey sand mixtures have relatively low plasticity indices (PI) of 1%, 4%, and 6% for the 5%, 10%, and 25% kaolin mixtures when compared to the PI of about 30% for the kaolin soil.

![Figure 3.22 Particle size distributions of clayey sands](image_url)
Table 3.14 Limiting void ratios and Atterberg limits results on clayey sands

<table>
<thead>
<tr>
<th>Property</th>
<th>fine sand</th>
<th>5%K + 95%S</th>
<th>10%K + 90%S</th>
<th>25%K + 75%S</th>
<th>kaolin</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity</td>
<td>$G_s$</td>
<td>2.65&lt;sup&gt;1&lt;/sup&gt;</td>
<td>2.648</td>
<td>2.645</td>
<td>2.638</td>
</tr>
<tr>
<td>Maximum void ratio $e_{\text{max}}$</td>
<td>0.77</td>
<td>0.94</td>
<td>1.04</td>
<td>1.42</td>
<td>-</td>
</tr>
<tr>
<td>Minimum void ratio $e_{\text{min}}$</td>
<td>0.48</td>
<td>0.44</td>
<td>0.43</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Liquid limit&lt;sup&gt;4&lt;/sup&gt; LL (%)</td>
<td>-</td>
<td>13</td>
<td>18</td>
<td>20</td>
<td>55</td>
</tr>
<tr>
<td>Plastic limit&lt;sup&gt;4&lt;/sup&gt; PL (%)</td>
<td>-</td>
<td>12</td>
<td>14</td>
<td>14</td>
<td>27</td>
</tr>
<tr>
<td>Plasticity index PI (%)</td>
<td>-</td>
<td>1</td>
<td>4</td>
<td>6</td>
<td>29</td>
</tr>
</tbody>
</table>

<sup>1</sup> K and S denote kaolin and fine sand respectively
<sup>2</sup> Xu (2007): $e_{\text{max}} = 0.78$, $e_{\text{min}} = 0.49$
<sup>3</sup> Stewart (1990): LL = 61%, PL = 27%
<sup>4</sup> after Chong (2012)

Figure 3.23 Soil properties on clayey soils: (a) limiting void ratios and (b) plasticity chart

The concept of a relative density ($D_r$) derived using the void ratio limits ($e_{\text{max}}$ and $e_{\text{min}}$) and in-situ void ratio ($e$) is often used to characterise sands. In sands containing fines, Mitchell (1976) suggested use of the intergranular void ratio ($e_g$), defined as:

$$e_g = \frac{V_{\text{void}} + V_{\text{clay}}}{V_{\text{sand}}} = G_s \frac{w_c + \frac{FC_C}{100G_{sC}}}{\frac{FC_C}{100}}$$

(3-7)

where $V_{\text{void}}$ is the volume of voids, $V_{\text{clay}}$ is the volume of clay fraction, $V_{\text{sand}}$ is the volume of sands, $G_s$ is the specific gravity of sands, $w_c$ is the water content of the soil, $FC_C$ is the ratio of clay fraction weight to total soil weight by dry weight, $G_{sC}$ is the specific gravity of clays. $e_g$ treats the volume occupied by the fines (i.e. the clay
fractions may be used for clayey sands) as a void space assuming implicitly that the shear strength is controlled by the force chains in the sand. When the $G_{sS}$ and the $G_{sC}$ are the same, the intergranular void ratio is referred to as the skeleton void ratio ($e_{sk}$) (Kuerbis et al. 1988).

In sands and sands with a small portion of fines such as clayey sands, specimen reconstitution methods may influence soil behaviour due to the creation of different densities and to the development of different fabrics associated with formation of clay within the sand matrix. Carraro et al. (2009) compare different specimen preparation methods employed by previous research and conclude that the slurry deposition (SD) method is most appropriate for the development of the in situ fabric of silty sands. This study employed the SD method to create sample for laboratory element tests, which was the same preparation method used for samples tested in the centrifuge and in the 1g penetration tests (described in Chapters 5 and 6). One exception was the 5% kaolin mixture, which (due to experimental difficulties) used the wet pluviation method (WP) for the simple shear tests.

3.6.3 Compression test

Rowe cell consolidation test

A series of Rowe cell 1-D consolidation (RCC) tests were carried out on the clayey sand samples and pure kaolin soil samples. The sand and clay dry particles were mixed with distilled water into a slurry state and then poured into the RCC cell. Consolidation time for each loading increment depended on soil component and stress level and therefore full consolidation was determined from fully dissipation of excess pore pressures.

Figure 3.24 shows the RCC test results on: (a) $e$ variations with log $\sigma'$, (b) $e_g$ variations with log $\sigma'$, (c) variations in coefficient of consolidation ($c_v$) with $\sigma'$, (d) variations in permeability ($k_v$) with $\sigma'$, and (e) compressibility index against clay fraction content. The $c_v$ values were deduced by comparing the pore pressure dissipation measured at the base of the sample to the Terzaghi’s one dimensional consolidation theory (up to 90% consolidation). The following observations are made:

- The $e$ and the $e_g$ reductions with effective stress level are more significant for soils containing a larger proportion of kaolin.
3. Field test procedures and soil properties

- The compression index ($C_c$) and the recompression ($C_r$) increase with an increase in clay fraction. The 5% and 10% kaolin mixtures are much more incompressible than the 25% kaolin mixture. The $C_c$ values for the kaolin soil, the 25%, 10% and 5% kaolin mixtures are 0.555, 0.153 (averaged), 0.033 and 0.035 respectively; the corresponding respective $C_r$ values are 0.089, 0.016, 0.006 and 0.009.

- The $C_c$ indices are higher than $C_c^*$ values determined using Equation (3-1) when clay fraction is relatively high (CF>20%); see Figure 3.24e. The $C_c$ and $C_s$ values differ slightly from kaolin parameters reported in literature (Lehane et al. 2009, Stewart 1990).

- $c_v$ generally increases with $\sigma'_v$ whereas $k_v$ slightly decreases with $\sigma'_v$. The relationships between $c_v$ and $\sigma'_v$ were approximately expressed by the power law function (as shown in Figure 3.24c and 3.24d). The $c_v$ values in the $\sigma'_v$ range of between 25 and 150 kPa are summarised in Table 3.15.

- The $c_v$ values for the 25% kaolin mixture was about one order of magnitude higher than that of pure kaolin clay whereas $c_v$ for the 10% and 5% kaolin mixtures are about 1,000 times values of the 25% kaolin. $c_v$ cannot be represented as a simple linear function of kaolin content in these clayey soils.

- The $k_v$ values in the 20-160 kPa stress range for the 25%, 10% and 5% kaolin mixtures were on average about 5.7×10$^{-9}$, 8.4×10$^{-7}$, and 4.9×10$^{-6}$ m/s, respectively. $k_v$ for the 25% kaolin mixture was over 100 times smaller than that of the 10% kaolin mixture, which was about 5 times smaller than that of the 5% kaolin mixture. Schneider et al. (2007) reported $k_v$ of 1×10$^{-9}$ m/s at $\sigma'_v$=100 kPa in kaolin, which is about 5 times smaller than that of the 25% kaolin mixture.

<table>
<thead>
<tr>
<th>$\sigma'_v$ (kPa)</th>
<th>Coefficient of consolidation, $c_v$ (m$^2$/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kaolin 25%K+75%S 10%K+90%S 5%K+95%S</td>
</tr>
<tr>
<td>25</td>
<td>2.1 11 1.3×10$^4$ 2.5×10$^4$</td>
</tr>
<tr>
<td>50</td>
<td>2.8 22 2.0×10$^4$ 3.3×10$^4$</td>
</tr>
<tr>
<td>100</td>
<td>3.7 43 3.1×10$^4$ 4.3×10$^4$</td>
</tr>
<tr>
<td>150</td>
<td>4.4 64 3.9×10$^4$ 5.1×10$^4$</td>
</tr>
</tbody>
</table>
3. Field test procedures and soil properties

(a) void ratio - vertical effective stress
(b) intergranular void ratio - vertical effective stress
(c) coeff. of consolidation - vertical effective stress
(d) permeability - vertical effective stress
(e) compressibility indices - clay content

Figure 3.24 Rowe cell test results for clayey sands and kaolin clay: (a) $e$-log$\sigma'_v$, (b) $e_v$-log$\sigma'_v$, (c) $c_v$-$\sigma'_v$, (d) $k_v$-$\sigma'_v$, and (e) compressibility indices
3. Field test procedures and soil properties

3.6.4 Strength test

A series of simple shear (SS) tests was conducted on the reconstituted clayey sands and kaolin soil using the (Berkeley type) UWA apparatus. Specimens in this apparatus are enclosed in an unreinforced latex membrane, back-pressure saturated, and consolidated prior to constant volume shearing. The specimen height (and hence volume) and total vertical stress on top of the specimen are kept constant during undrained shearing by locking the vertical actuator and adjusting the cell pressure, respectively. The apparatus is described in more detail in Mao (2000) and Mao & Fahey (2003).

The UWA apparatus uses a rubber membrane to apply a confining pressure ($\sigma_3$) in a similar way to a triaxial apparatus. The shear stress applied on the horizontal plane ($\tau_{xy}$) and its complimentary stress on the vertical planes together with the vertical effective stress ($\sigma'_{v} = \sigma_{v} - u$) is often used to interpret a simple shear test. However, it has been observed in a large number of tests that a typical failure plane tends to be diagonally developed in the UWA ‘Berkeley’ type simple shear test (Joer et al. 2010). In order to account for this failure mechanism, Joer et al. (2010) proposed new interpretation method considering a normal stress ($\sigma'_n$) and a shear stress ($\tau_{fail}$) acting on a diagonal failure plane through the sample. In this study, both interpretation methods are employed for data analysis and are referred to as the ‘traditional’ method and ‘new’ method. It is noted that shear strain ($\gamma$) during simple shear test is calculated as the ratio of sample horizontal displacement to sample height for both interpretation methods.

In this series of simple shear tests, specimens were generally slurry deposited (SD) as this preparation method was also used to prepare the centrifuge and 1g samples for penetration tests. The 10% and 25% kaolin mixtures and the 100% kaolin soil were mixed with distilled water and pre-consolidated under 20 kPa in stainless tubes with a diameter of ~72 mm. The 5% kaolin mixture specimens were prepared directly in a mould by wet pluviation (WP) as it was found when applying the SD method that suctions large enough to keep the samples together were not achievable after their extrusion from the stainless steel tubes. The specimens had a diameter of ~71 mm and a height between 38 mm and 41 mm depending on the preparation methods.

The specimen was firstly saturated under a backpressure of 400 kPa with a cell pressure of 410 kPa prior to consolidation. A minimum B-value of 0.96 was achieved. The specimen was then anisotropically consolidated with a stress ratio ($K = \sigma'_{h}/\sigma'_{v}$) of about
0.5. Once consolidation was completed (i.e. when no significant volumetric change was observed), shearing was commenced at a constant rate of 0.1 mm/min (equivalent to a shear strain rate of between 0.005 and 0.008 %/s) in undrained shearing, and at 0.01 mm/min (equivalent to a shear strain rate of 0.0005 to 0.0006 %/s) in drained shearing.

Table 3.16 summarises the results of the 15 simple shear tests performed. Figure 3.25 plots the test results obtained (using the ‘traditional’ and ‘new’ interpretation methods) during an undrained tests under a consolidation stress, $\sigma'_{vc}$ of about 100 kPa All other test results are provided in Appendix A.

The following observations can be made:

- Shearing behaviour is strongly influenced by the clay (kaolin) fraction; such a strong influence has also been observed in undrained triaxial compression tests on clayey sands by Georgiannou et al. (1990) and others.
- Strong contractant behaviour was observed after the peak shear stress ($\tau_{xy}$) was reached at a relatively small shear strain ($\gamma$) in the 25% kaolin mixture. Shear induced excess pore pressures ($\Delta u$) were much higher in the 25% kaolin mixture than in the 100% kaolin soil.
- Contractant behaviour was also exhibited by the 5% and 10% kaolin mixtures, but only up to $\gamma$=5% after which there was significant strain hardening (and particularly so for the 5% kaolin mixture). A steady state was not achieved for these dilative materials within the shearing range investigated ($\gamma$ ~ 20%).
- The ‘new’ interpretation method, for which the initial shear stress ($\tau_{fail}$) was not zero because of anisotropic consolidation, indicated that peak shear stresses ($\tau_{fail}$) were mobilised at the smaller shear strains, compared to the traditional interpretation method.
- Stress paths ($\tau_{xy}$ vs. $\sigma'_v$ and $\tau_{fail}$ vs. $\sigma'_n$) show strong contraction and some undrained brittleness for the 5%, 10% and 25% kaolin mixtures.
- Differences between the undrained response of samples prepared using the SD and WP sample formation methods are compared for the 10% kaolin mixture in Appendix A (see tests SS10K1 and SS10K4); no significant difference was observed.
In drained monotonic shearing, the following observations can be made:

- The volumetric strain ($\varepsilon_v$) induced by drained shearing of the 25% kaolin mixture is much greater than measured for the 100% kaolin soil; this observation is consistent with higher value of excess pore pressure generated during undrained shearing of the 25% kaolin mixture.
- The 5% and the 10% kaolin mixtures exhibit dilation after contraction. A much greater level of dilation was indicated by the 5% kaolin mixture.

Table 3.16 Summary of simple shear tests on clayey sands

<table>
<thead>
<tr>
<th>Test name</th>
<th>Prep</th>
<th>$\rho_{d_{ini}}$ (t/m$^3$)</th>
<th>$\sigma'_v$ (kPa)</th>
<th>$e$</th>
<th>$e_g$</th>
<th>D or UD</th>
<th>$\dot{\gamma}$ (%/s)</th>
<th>$\tau_{xy_{20}}$ (kPa)</th>
<th>$\tau_{fail_{20}}$ (kPa)</th>
</tr>
</thead>
</table>
| kaolin clay
| SS100K1 | SD   | 1.03 | 49.0 | 1.52 | -     | UD     | 0.0072          | 10.7            | 12.2            |
| SS100K2 | SD   | 1.03 | 98.0 | 1.48 | -     | UD     | 0.0082          | 19.6            | 23.3            |
| SS100K3 | SD   | 1.07 | 51.4 | 1.43 | -     | D      | 0.0006          | 13.5            | 20.4            |
| 25%K + 75%S
| SS25K1  | SD   | 1.71 | 47.8 | 0.52 | 0.91  | UD     | 0.0068          | 9.6             | 9.1             |
| SS25K2  | SD   | 1.73 | 48.4 | 0.51 | 0.89  | UD     | 0.0068          | 10.6            | 9.1             |
| SS25K3  | SD   | 1.73 | 97.5 | 0.49 | 0.87  | UD     | 0.0071          | 15.5            | 14.9            |
| SS25K4  | SD   | 1.73 | 99.6 | 0.49 | 0.87  | UD     | 0.0064          | 15.4            | 13.8            |
| SS25K5  | SD   | 1.72 | 48.2 | 0.52 | 0.91  | D      | 0.0006          | 20.0            | 26.1            |
| 10%K + 90%S
| SS10K1  | SD   | 1.63 | 51.5 | 0.60 | 0.74  | UD     | 0.0054          | 16.7            | 15.4            |
| SS10K2  | SD   | 1.64 | 100  | 0.59 | 0.73  | UD     | 0.0055          | 26.5            | 24.7            |
| SS10K5  | SD   | 1.65 | 51.1 | 0.60 | 0.74  | D      | 0.0005          | 21.8            | 28.0            |
| 5%K + 95%S
| SSS2K2  | WP   | 1.67 | 99.8 | 0.56 | 0.63  | UD     | 0.0055          | 69.5            | 84.9            |
| SSS3K3  | WP   | 1.67 | 50.8 | 0.57 | 0.64  | UD     | 0.0054          | 56.5            | 63.5            |
| SSS4K4  | WP   | 1.69 | 48.9 | 0.55 | 0.62  | D      | 0.0005          | 21.2            | 38.0            |

where Prep: preparation method (SD: slurry deposition, WP: wet pulviation)

- $\rho_{d_{ini}}$: initial dry density
- $\sigma'_v$: vertical effective stress after consolidation
- $e$: void ratio
- $e_g$: intergranular void ratio
- D or UD: Drained or Undrained shearing
- $\dot{\gamma}$: shear strain rate
- $\tau_{xy_{20}}$: shear stress at $\dot{\gamma} = 20\%$ (Traditional method)
- $\tau_{fail_{20}}$: shear stress at $\dot{\gamma} = 20\%$ (New method)
3. Field test procedures and soil properties

Figure 3.25 Undrained simple shear test results of clayey sands and kaolin clay: (a) $\tau_{xy}-\gamma$, (b) $\tau_{fail}-\gamma$, (c) $\Delta u-\gamma$, (d) $\tau_{xy}-\sigma'_v$ and (e) $\tau_{fail}-\sigma'_n$
The simple shear undrained strengths, assuming both the $\tau_{xy}$ and the $\tau_{fail}$ values at $\gamma=20\%$, are listed above in Table 3.16. Respective undrained shear strength ratios ($s_{uss}/\sigma'_{vc}$) are plotted against the vertical effective stress ($\sigma'_{vc}$) in Figure 3.26a, and against clay fraction in Figure 3.26b.

The undrained strength ratio measured in the 25% kaolin mixture was lower ($s_{uss}/\sigma'_{vc}$ average of ~0.18) than the 100% kaolin clay ($s_{uss}/\sigma'_{vc}$ average of ~0.23). This lower ratio cannot be explained in terms of a lower friction angle and is likely due to more compressible structure in the 25% kaolin mixture than that of the kaolin soil itself. The $s_{uss}/\sigma'_{vc}$ ratios in the 10% and the 5% kaolin mixtures averaged at 0.29 and 1.02 respectively. Dilative behaviour may be expected to be more in evidence at the lower stress level ($\sigma'_{vc}=50$ kPa). This tendency is seen to give slightly higher $s_{uss}/\sigma'_{vc}$ ratios at the low stress level for the 5% kaolin mixture, although the same trend was not clear for the other mixtures (see Figure 3.26a). Clayey sands with clay fractions in excess of 20% (the 25% kaolin mixture) generally appear to behave as clays. $s_{uss}/\sigma'_{vc}$ ratios have been reported in the literature; for example, Ladd (1991) found that a $s_{uss}/\sigma'_{vc}$ ratio varies between 0.20 and 0.25 in normally consolidated clays and silts, and tends to decrease with lower plasticity in direct simple shear tests. Similar $s_{uss}/\sigma'_{vc}$ ratios can be found in others (e.g. Bjerrum & Landva 1966).

The findings in the simple shear tests agree with some observations on clayey sands in triaxial compression tests by Georgiannou et al. (1990) and Hight et al. (1994). Hight et al. (1994) presented that a clay content of 10% shown the maximum undrained brittleness but undrained brittleness reduced for higher clay contents. The particular material with a clay content of 30% in the paper might be suggested to exhibit the behaviour of normally consolidated clay. Although dilatancy generally reduces with increasing fines content, the more contractant response to undrained shear of the clayey sand with a kaolin content of 25% compared with 100% kaolin soil highlights the importance of fabric on soil response.

Apparent peak friction angles ($\phi'_p$) against clay fraction were estimated from the undrained simple shear tests (see Figure 3.27). Three methods were considered assuming no cohesion: (i) $\phi'_p=\tan^{-1}(\tau_{xy}/\sigma'_{v})$, (ii) $\phi'_p=\sin^{-1}(t/s')$ in the traditional method, where $t$ is the plane strain shear stress and $s'$ is the mean stress, and (iii) $\phi'_p=\tan^{-1}(\tau_{fail}/\sigma'_{n})$ in the new method. Although interpreted $\phi'_p$ values for the 25% kaolin
mixture seems to be slightly lower than the two other clayey sands, $\phi'_p$ for the 5%, 10% and 25% kaolin mixtures are broadly similar with an range of 28° to 34° and average of 32°. This average angle is about 8° higher than the interpreted $\phi'_p$ values of 24° for kaolin clay (which contains clay fractions of 80%; see Figure 3.22). The $\phi'_p$ obtained in the simple shear tests are in reasonably agreement with the constant volume (critical state) friction angle ($\phi'_c$) of 31° in fine silica sand (White et al. 2008) and the $\phi'_c$ of 24 ±2° in normally consolidated kaolin clay (Lehane et al. 2009).

(a) Undrained shear strength ratios (Traditional)  
(b) Undrained shear strength ratios - clay fraction

Figure 3.26 Undrained shear strength ratios in simple shear tests: (a) $s_{uw}/\sigma'_{vc}$ and (b) $s_{uw}/\sigma'_{vc}$-CF

(c) Apparent peak friction angles

Figure 3.27 Apparent peak friction angles
CHAPTER 4  FIELD VARIABLE RATE PENETRATION TEST RESULTS

4.1 INTRODUCTION

Three sets of variable rate piezocone tests (CPTUs) were conducted at three Western Australian sites, namely Gingin, Bassendean and Burswood. The primary purpose of these field tests was to extend the currently sparse database of variable rate piezocone tests in the field, using the UWA CPT rig that has the capability of varying penetration rate by six orders of magnitude. Site information and the soil properties obtained in the laboratory tests are presented in Chapter 3. The CPTUs were initially conducted at the standard rate of 20 mm/s and subsequently penetration velocities were varied between 20 mm/s and 0.0002 mm/s. Piezocone dissipation tests were also conducted at each site. At the Bassendean site, T-bar and ball penetrometers were also used to further investigate penetration rate effects.

Some of the field test results presented in this Chapter have been published in Suzuki et al. (2012) and Suzuki & Lehane (2014a); Appendix D.

4.2 GINGIN SITE

4.2.1 Standard rate piezocone penetration test

A corrected total cone resistance \( q_t \) was calculated to take account of the unequal pore pressure effect using the following relationship (Lunne et al. 1997b):

\[
q_t = q_c + u_2 (1 - \alpha)
\]  (4-1)

where \( q_c \) is the measured cone resistance, \( u_2 \) is the pore pressure measured at the cone shoulder and \( \alpha \) is the unequal area ratio (=0.73 for the CPTU). The net cone resistance \( q_{net} \) was then calculated as:
Field variable rate penetration test results

\[ q_{\text{net}} = q_t - \sigma_{v0} \]  \hspace{1cm} (4-2)

where \( \sigma_{v0} \) is the total vertical stress.

Normalised CPTU parameters were then used for soil profiling and soil behaviour type (SBT). Normalised cone resistance \( (Q_t) \) is often used to take account of the in-situ vertical effective stress \( (\sigma'_{v0}) \):

\[ Q_t = \frac{q_{\text{net}}}{\sigma'_{v0}} = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \]  \hspace{1cm} (4-3)

The ratio of excess pore pressure \( (\Delta u = u_2 - u_0) \) to \( q_{\text{net}}, \) referred to as the pore pressure ratio \( (B_q) \), can be determined as:

\[ B_q = \frac{\Delta u}{q_{\text{net}}} = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \]  \hspace{1cm} (4-4)

The profiles of the corrected tip resistance \( (q_t) \) and the pore pressure \( (u_2) \) measured in a standard CPTU at the Gingin site are shown in Figure 4.1. A relatively soft/loose soil layer can be seen between 0.6 and 1.3 m depth, where the \( q_t \) value lies between 200 and 300 kPa. Beneath this layer, the \( q_t \) increases with depth until again reducing in a 200 mm thick layer at 2.2 m. The pore pressures are zero throughout except in the 0.6-1.3 m and the 2.2-2.4 m soft layers where the \( u_2 \) values are between 10 and 30 kPa. The seasonal variation of the water level in the tailings pond (~30 m away) coupled with the presence of layers with relatively low permeability provide an explanation for the observed \( u_2 \) values. It appears that after the winter rain has drained from the sandy layers, residual pore pressures may be maintained in the less free draining layers. The observed \( u_2 \) pore pressure profiles suggest there may have been a perched water table above this layer during the winter season. Therefore, two-way drainage can be expected in the 0.6-1.3 m soft layer.

Prior to installation of the CPTU, the very hard surface crust (about 12 to 18 cm deep) was predrilled by hand for all penetration locations. This is unlikely to have had any influence on the penetration measurements at the focused depths below 0.9 m as the \( q_t \) profiles began to stabilise at 0.7 m.
The normalised cone resistance ($Q_t$) and the pore pressure ratio ($B_q$) as defined in Equations (4-3) and (4-4) are about 20 and 0.1 respectively in the 0.6-1.3 m layer. This layer is classified as a silty sand to sandy silt according to the soil classification chart proposed by Robertson (1990). The other pore pressure ratio calculated ($\Delta u / \sigma'_{v0}$) was about 1.8, which classifies this layer as a silt and low rigidity clay according to Schneider et al. (2008).

Undrained shear strength ($s_u$) may be estimated from the cone net resistance and the cone factor ($N_{kt}$):

$$s_u = \frac{q_{net}}{N} = \frac{q_t - \sigma'_{v0}}{N_{kt}}$$  \hspace{1cm} (4-5)

The $s_u$ of the 0.6-1.3 m soft layer was estimated to be approximately 20 kPa, assuming a nominal $N_{kt}$ value of 14 and fully undrained conditions during piezocone penetration.
4.2.2 Variable rate piezocone penetration tests

The soft layer was investigated at various cone penetration rates (see the test programme in Section 3.3.2). Figure 4.2 shows the $q_t$ and $u_2$ profiles from the seven piezocone penetration tests performed in the soft layer. Although some local variations at the site interfered with interpretation of the rate effect, the $q_t$ was seen to be roughly independent of the penetration rate when the rate was between 0.2 and 20 mm/s ($q_t$ about 280 to 330 kPa) but higher when the rate fell below 0.02 mm/s. The $u_2$ measurements mirrored the $q_t$ data with similar values being recorded at penetration rates between 0.2 to 20 mm/s with much lower values at a penetration rate of 0.02 mm/s.

![Figure 4.2 Comparison for the rate tests at the Gingin site: (a) $q_t$ and (b) $u_2$](image-url)
Figure 4.3 summaries the $q_{\text{net}}$ and $u_2$ measurements over ~100 mm depth intervals for each penetration velocity within the soft layer as indicated to be ‘zone of interest’ in Figure 4.1a. The each measured values were the averaged values over ~100 mm depth intervals and correspond to each data symbolled in Figure 4.2. The values are further averaged for each velocity to examine the tendency of rate dependence. It is evident in Figure 4.3 that the $q_{\text{net}}$ value at 0.02 mm/s was 60±20% higher than that at 20 mm/s but the $u_2$ value at 0.02 mm/s was only 30% lower than at 20 mm/s.

Figure 4.3 Penetration rate effects at the Gingin site: (a) $q_{\text{net}}$ and (b) $\Delta u$
4. Field variable rate penetration test results

4.2.3 Dissipation test

A dissipation test was also performed to estimate the in situ horizontal coefficient of consolidation \(c_h\). Figure 4.4a shows the penetration profiles of \(q_t\) and \(u_2\) at the penetration velocity of 20 mm/s for the dissipation test at 1.5 m depth (GCDT1; see Table 3.3) with a comparison of the standard rate CPTU (GCST1) presented in Figure 4.1. It appears that the similar soft layer is present at slightly variable depths between the testing locations.

Figure 4.4b shows the dissipation of the pore pressure at 1.5 m depth. The observed delay in reaching peak pore pressure is likely to be related to the slow response time of the pore pressure transducer. After the peak pore pressure was reached, the pore pressure then dissipated. The excess pore pressures (\(\Delta u\), assuming the perched water pressure for this soft layer as discussed above), were normalised by the initial excess pore pressure (\(\Delta u_{\text{pred}}\)), which was predicted by assuming a linear reduction in \(u_2\) with the square root of time (as illustrated in Figure 4.4c). In Figure 4.4c, the \(\Delta u/\Delta u_{\text{pred}}\) values are plotted against the normalised time factor \((T^*)\) proposed by Teh & Houlsby (1991) to assess \(c_h\) values using the following relationship:

\[
c_h = \frac{T^*}{t} r^2 \sqrt{I_r}
\]  

(4-6)

where \(t\) is the time, \(a\) is the cone radius and \(I_r\) is the soil rigidity index.

The time to reach 50\% dissipation (\(t_{50}\)) was about 480 seconds (7.9 mins) after penetration ceased. Assuming a soil rigidity index (\(I_r\)) of 150, a \(c_h\) value of 2 mm\(^2\)/s (63 m\(^2\)/year) was derived at 1.5 m depth. This \(c_h\) value may vary between 51-115 m\(^2\)/year depending on whether the \(I_r\) is considered to be 100 or 500. The in situ \(c_h\) value of 63 m\(^2\)/year estimated using \(I_r\) of 150 is about 60 times higher than the vertical coefficient of consolidation \((c_v\text{NC})\) of 1.1 m\(^2\)/year measured at the same vertical effective stress on a reconstituted normally consolidated sample; see Section 3.3.4. This difference can be explained by (i) acknowledgement that, unlike the normally consolidated state in the laboratory, soil largely follows a (stiff) re-loading path during a piezocone dissipation test (ii) anisotropic permeability and (iii) the dependence of the coefficient of consolidation on the soil structure.
Figure 4.4 Dissipation test result at 1.5 m depth at Gingin site: (a) penetration profiles, (b) $u_2$ dissipation and (c) the Teh and Houlsby (1991) solution
4.3 BASSENDEAN SITE

4.3.1 Standard rate piezocone penetration tests

All in situ penetration tests at the Bassendean site, including the Phase I (summer) and Phase II (winter) tests are described in this section. As the depth to the water table at this site fluctuates between 1 m and 2 m due to seasonal and tidal effects, water table depths of 1.5 m and 1.2 m were used to analyse the test data in Phase I and Phase II, respectively. The CPTU penetrations were installed within hand augered, 0.8m to 1.0m deep pre-drilled boreholes in Phase I and within 2.5 m boreholes created by a dummy cone in Phase II (as explained in Section 3.2.3). The penetration test results down to 3 m from the ground surface are not considered representative due to the potential influence of the predrilling.

Figure 4.5 shows the $q_{net}$ and $u_2$ results from four standard rate CPTUs. The $q_{net}$, as defined in Equation (4-2), generally increased linearly as the depth below the upper 2.5 m thick crust increased from ~150 kPa at 3 m to ~370 kPa at 10 m with a $q_{net}$ gradient of ~31 kPa/m. Apart from occasional drops, $u_2$ generally increased from ~90 kPa at 3 m to ~275 kPa at 10 m with a gradient of ~26 kPa/m. Spikes in $q_{net}$ were observed, especially for BassCRT2 between 5.5 and 7 m, which corresponded to drops in the $u_2$ profiles. BassCRT2 was close to a tree but more than 5 m away from the other penetrations and the results suggest that more free draining sandy horizons might exist at this location. The $u_2$ values measured in Phase II (BassCRT16) show a rapid recovery in pore pressure after the drops in sandy layers, particularly at shallower depths.

Figure 4.6 presents the piezocone parameters obtained at the standard penetration rate of 20 mm/s. The normalised cone resistance ($Q_t$) calculated from Equation (4-3) ranged from 4.5 to 6 (excluding the sandy spikes). The pore pressure ratio ($B_q$) defined by Equation (4-4) and the pore pressure ratio ($\Delta u/\sigma'_{vo}$) were 0.50±0.05 and 2.6±0.2, respectively. In the Robertson (1990) chart, this material can be classified as a clay to silty clay; it classifies as a clay in the Schneider et al. (2008) chart. The normalised friction ratio ($F_r$) can also be used for the soil classification in the Robertson (1990) chart as defined as:
where $f_s$ is the measured sleeve friction and $q_{net}$ is the net cone resistance defined by Equation (4-2). In the Bassendean material, the $F_r$ ranges from 3 to 5% in one of the standard rate tests (BassCRT16) but greater variations in $f_s$ measurements were found throughout the standard rate CPTU tests compared to the other CPTU measurements of $q_c$ and $u_2$. According to soil behaviour type ($I_c$) classification proposed by Robertson & Wride (1998), the Bassendean material can be categorised as a silty clay to clay.

(a)  
\[ q_{net} = 31.4z + 55.7 \]
(b)  
\[ u_2 = 26.4z + 10.7 \]

Figure 4.5 Standard rate CPTU results at the Bassendean site: (a) $q_{net}$ and (b) $u_2$.
4. Field variable rate penetration test results

Figure 4.6 CPTU parameters at the Bassendean site: (a) $Q_t$, (b) $B_q$, (c) $\Delta u/\sigma'_{vo}$, (d) $F_r$ and (e) $I_c$
Effect of various filter elements

Figure 4.7 shows the CPTU test results at the standard penetration rate of 20 mm/s for \( q_{\text{net}} \) and \( u_2 \) using various types of filter elements. Three different filter elements were used in these tests: one sintered bronze element (BassCRT11), one porous polypropylene element (BassCRT16) and one sintered stainless element (BassCRT17). The porous polypropylene filter elements were used at all other in situ test sites discussed in this Thesis, except for BassCRT11 and BassCRT17. A porous polypropylene element was also used when the pore pressure transducer was calibrated.

The \( q_{\text{net}} \) and \( u_2 \) results for the three CPTUs are in reasonably good agreement between 3 and 7 m depth. There are some variations in \( q_{\text{net}} \) peaks and corresponding drops in \( u_2 \) below 7.5 m, especially for BassCRT11. However, the disparity in the \( q_{\text{net}} \) and \( u_2 \) results among the three tests is believed to be mainly due to the sandy layers frequently encountered at BassCRT11. Overall, the type of filter employed had no significant effect on the CPTU measurements.

![Diagram showing CPTU test results with different filter elements](image-url)

Figure 4.7 Effect of porous filters on CTPU results: (a) \( q_{\text{net}} \) and (b) \( u_2 \)
4.3.2 Standard rate full-flow penetrometer tests

To further investigate penetration rate effects at the Bassendean site, the cone probe was replaced with T-bar and ball probes. The T-bar probe had a 250 mm×40 mm bar and the ball had a diameter of 75.35 mm. The T-bar net tip resistance (\( q_{\text{T-bar}} \)), and the ball net tip resistance (\( q_{\text{ball}} \)) were corrected for unequal pore pressure effects (\( \alpha \)) and overburden pressure effects (\( \sigma_{v0} \)) using the following relationship (Chung & Randolph 2004):

\[
q_{\text{T-bar}} \text{ or } q_{\text{ball}} = q_m - [\sigma_{v0} - u_0(1 - \alpha)] \cdot \frac{A_s}{A_p} \quad (4-8)
\]

where \( q_m \) is the tip resistance measured by the T-bar or the ball probe (which is the total load divided by \( A_p \)), \( A_s \) is the cross-sectional area of connection shaft in plane normal to shaft, and \( A_p \) is the projected area of the probe in a plane normal to shaft. The \( A_s \) value was 10 cm\(^2\) which is the same as the CPTU while \( A_p \) for the T-bar was 100 cm\(^2\) and \( A_p \) for the ball was 44.6 cm\(^2\). These dimensions result in low \( A_s/A_p \) ratios of 0.1 for the T-bar and 0.22 for the ball penetrometer and hence any errors associated with correction for overburden pressure and unequal area ratios are not significant compared to those for the piezocone penetrometer.

Figure 4.8a compares the net resistances measured using the piezocone, T-bar and ball penetrometers at the standard penetration rate of 20 mm/s. 1.5m deep boreholes were pre-drilled prior to the T-bar installations. Predrilled boreholes were not needed for the ball installations. The T-bar and the ball net resistance (\( q_{\text{T-bar}} \) and \( q_{\text{ball}} \)) are very similar for both penetration and extraction. Penetration resistances increase linearly from ~140 kPa at 3 m to ~250 kPa at 9 m with a gradient of ~18 kPa/m, which is lower than the gradient for \( q_{\text{net}} \) recorded by the piezocone penetrometer. The \( q_{\text{T-bar}}/q_{\text{net}} \) and \( q_{\text{ball}}/q_{\text{net}} \) for penetration decrease from 0.85 at 3 m depth to about to 0.75 at 9 m depth (see Figure 4.8b). Figure 4.8c shows the ratios of extraction resistance divided by penetration resistance (\( q_{\text{out}}/q_{\text{in}} \)) for the T-bar and ball penetrometers. The \( q_{\text{out}}/q_{\text{in}} \) ratios for the T-bar were ~0.5 between 3 and 9 m depth but the \( q_{\text{out}}/q_{\text{in}} \) ratios for the ball varied from 0.5 to 0.7 at the same depth since the ball penetration measured higher \( q_{\text{out}} \) values between 3 and 4 m between 7 and 9 m during extraction. Figure 4.8d compares the pore pressure measurements recorded at the \( u_2 \) position in a CPTU test with pore pressures recorded by the same sensor during T-bar and ball penetration tests (see Section 3.2.1). The pore
pressures recorded with the full-flow probes are evidently much lower than the cone $u_2$ values but are slightly higher than the hydrostatic pressures ($u_0$).

![Graphs showing comparisons of various penetrometers on penetration profiles.](image)

Figure 4.8 Comparisons of various penetrometers on penetration profiles: (a) net resistance, (b) net resistance ratio, (c) net resistance ratio of penetration to extraction and (d) $u_2$. 
4.3.3 Undrained shear strength

Undrained shear strengths ($s_u$) can be assessed using in situ standard rate penetration test results in clayey soils as defined as Equation (4-5) in the case of CPTUs. $s_u$ can be estimated in a similar fashion from the T-bar and ball penetrometers by dividing the net resistances by a T-bar factor ($N_{T-bar}$) and a ball factor ($N_{ball}$).

Figure 4.9 shows undrained shear strength profiles assessed by the three different penetrometers, using $N_{kt}$ of 12, and $N_{T-bar}$ and $N_{ball}$ of 10.5. Using these $N$ values, the $s_u$ profiles for the three penetrometers are in agreement below 3 m. Note that the ground surface layer is not the focus here due to the use of 3m deep predrilled boreholes to facilitate the penetrometer installations. The $s_u$ values generally increased from 12 to 25 kPa between depths of 3 to 9 m, resulting in an undrained shear strength gradient of about 2.2 kPa/m. The sharp peaks in the $s_u$ profiles are believed to arise because of the presence of thin sandy layers; these are clearly not representative $s_u$ values.

The $s_u$ values obtained from CAU triaxial compression tests in the laboratory (see Section 3.4.5) are also plotted in Figure 4.9. Using the $N$ values specified above, the $s_u$ estimations from the in situ penetration tests are reasonably comparable to the $s_u$ values.
obtained after strain rate correction in the triaxial compression tests (see Section 3.4.5). The \( N \) values, which gave the best fit to the \( s_u \) values obtained in the triaxial undrained compression tests, were within the \( N \) factors recommended in Low et al. (2010) and fairly agree with the theoretical studies (e.g. Randolph 2004). The \( N \) values (defined as the excess pore pressure, \( \Delta u \), over the \( s_u \) measured in the CAU triaxial compression) are found to be \(~6.5\) (on average), which is also similarly reported in Low et al. (2010).

### 4.3.4 Variable rate penetration tests

Comprehensive variable rate penetration tests were undertaken in both Phase I and Phase II at the Bassendean site. Two testing methods were used: a constant rate test at a penetration rate between 20 and 0.002 mm/s in Phase I, and a ‘twitch’ test format (as originally proposed by House et al. (2001)) with the velocity varying from 0.0002 to 20 mm/s in Phase II. Table 4.1 summarises all of the variable penetration rate tests details including the velocities and travelling distance used for each variable rate test. The penetration traveling distance for each penetration rate was set at either 72 or 180 mm (equivalent to two or five times the cone diameter).

<table>
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<th>Travel distance (mm)</th>
<th>Figure</th>
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<td>B-3</td>
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</tr>
<tr>
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<td>180</td>
<td>B-4</td>
</tr>
<tr>
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<td>72</td>
<td>B-5</td>
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<tr>
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<td>BassCRT18</td>
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<td>72</td>
<td>B-8</td>
</tr>
<tr>
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<td>72</td>
<td>B-10</td>
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<tr>
<td>BassBRT2</td>
<td>4-5</td>
<td>20, 2, 0.2, 0.02, 0.002, 0.001</td>
<td>180</td>
<td>B-11</td>
</tr>
</tbody>
</table>

*1 Field test ID corresponds to the testing details (see Section 3.4.2)
*2 Both penetration rate and travelling distance are indicated in the figure
*3 Figures B-1 to B-11 are provided in Appendix B
The $q_{\text{net}}$ and $u_2$ profiles for the Phase I constant penetration rate tests, for which velocities ranged from 0.002 to 20 mm/s, reveal the following trends (see Figure 4.10):

- In general, $q_{\text{net}}$ increased and $u_2$ decreased when the cone velocity ($v$) reduced. At ~3.3 m depth, $q_{\text{net}}$ values at $v<0.02$ mm/s (~400±50 kPa) are about twice $q_{\text{net}}$ values at $v>2$ mm/s (~180±20 kPa).
- The $u_2$ values at $v<0.02$ mm/s are essentially equivalent to the ambient pore pressure ($u_0$) of ~20 kPa, suggesting that the CPTUs at these slower penetration rates are essentially drained in this material.
- Partially drained conditions are evident at $v=0.2$ mm/s with for which $q_{\text{net}}$ are higher and $u_2$ values are lower compared to those at $v=2$ mm/s.
- Interestingly, a ‘wavy’ characteristic of the $q_{\text{net}}$ and $u_2$ profiles is noticeable at $v=0.2$ and 0.02 mm/s. The ‘wave length’ of these waves was typically about 150 mm, which may be indicative of the amount of sediments deposited annually by the Swan River in that area.
- Within these wavy profiles, the peaks in $q_{\text{net}}$ and troughs in $u_2$ might are likely to indicate slightly more free draining material than the soil immediately above and below the layers. In the same way, the local troughs in $q_{\text{net}}$ would correspond to the finest material present within the stratigraphy.
- Negative excess pore pressures were measured at $v=20$ mm/s after drained penetrations at much lower velocities were completed e.g. between 3.5 and 4 m depth after the penetrations at $v=0.01$ and 0.004 mm/s were completed.
4. Field variable rate penetration test results

Figure 4.10 Constant rate tests with the velocity varied from 0.002 to 20 mm/s at 3-4m depth in BassCRT1-10: (a) $q_{\text{net}}$ and (b) $u_2$

Figure 4.11 shows a typical ‘twitch’ variable penetration rate test result for the Bassendean site. The standard test result ($v=20\text{mm/s}$) of BassCRT16 is also plotted for comparison purposes. Averaged values of $v$, $q_{\text{net}}$, $u_2$ for each penetration rate required filtering prior to examining the penetration rate effects. In the averaging process, potential sandy layers were avoided and only data recorded after a penetration of at least $\sim2d$ (i.e. twice of the cone diameter) were considered. The other ‘twitch’ variable rate test results are provided in Appendix B. The following observations can be made from a series of the ‘twitch’ variable penetration rate tests:

- The $q_{\text{net}}$ values, in general, start to increase when the velocities reduce from 2 to 0.2 mm/s; for example, this trend was observed between 5.4 m and 5.6 m apart from within a potential sandy layer (see Figure 4.11). On the other hand, penetration rate effects are not evident within the velocity range of 20 to 0.4 mm/s used between 4 m and 5 m in BassCRT14 (see Appendix B, Figure B-3).
• The $u_2$ reductions, as expected, show a good correspondence with the increases in $q_{net}$ (reflecting partial consolidation around the cone tip). The $u_2$ values measured at $v=0.2$ mm/s tend to be less than those at $v=2$ mm/s (e.g. see between 5.4 m and 5.6 m depth in Figure 4.11).

• The $u_2$ values recorded at $v$ equal to or less than 0.02 mm/s are almost equivalent to the hydrostatic pressures ($u_0$) (e.g. see at 5.8 m depth in Figure 4.11). At much slower penetration rates of $v=0.002$ mm/s and 0.0002 mm/s, no excess pore pressures are measured suggesting that the CPTUs at $v<0.002$ mm/s were drained in the Bassendean material (e.g. see Appendix B, Figure B-7).

• Local consolidation occurred while the penetration was halted to add the next 1 m cone rod. This pause period resulted in higher peak resistances and lower pore pressures when penetration resumed and was advanced (e.g. see at ~6.1 m depth in Appendix B, Figure B-1). Similarly, McNeilan & Bugno (1985) reported that the increase in soil strength (called as set-up) occurred within a short stopping time in silts as fast dissipation of pore pressure was expected.

• Negative pore pressures were measured when the penetration rate was returned back to 20 mm/s after drained penetrations were achieved at $v<0.002$ mm/s (e.g. see at around 6 m in Figure 4.11); this feature is also observed in Figure 4.10. Negative pore pressures are potentially associated with undrained shearing (at the standard CPTU rate) in clay that has been overconsolidated by the action of drained penetration.

• Penetration rates varying from slower to faster (from 0.002 to 20 mm/s), were used between 7.3 m and 8.3 m in BassCRT1 (in contrast to the other tests for which penetration rates reduced (see Appendix B, Figure B-6). $q_{net}$ values generally reduced as pore pressures increased in response to the increasing penetration rate. These observations were complicated by the presence of sandy layers within the clay at these depths.

• Similar penetration rate effects using the T-bar and the ball probes were observed (see Appendix B, Figures B-8 to B-11). The $q_{T-bar}$ and the $q_{ball}$ resistances started to increase with decreasing velocity when the velocity fell below 0.2 mm/s.
Figure 4.11 Example of variable rate CPTU test result: BassCRT14 at 5 to 6 m depth

Figure 4.12 to 4.14 summarise penetration rate effects observed at the Bassendean site. The $q_{net}$ and $\Delta u$ values, which were selected after at least $2d$ (i.e. twice of the cone diameter) penetration at constant velocity are plotted on Figure 4.12. Variations in piezocone parameters: $Q_0$, $B_q$ and $\Delta u/\sigma'_{v0}$ with the penetration velocity are shown using the averaged data (see Figure 4.13). Figure 4.14 shows the $q_{T-bar}$ and $q_{ball}$ variations with the probes’ velocities using the averaged data.

The following observations can be made:

- Sandy layers within the clay stratum led to large increases in the $q_{net}$ values. These are not included in the averaged data to avoid misinterpretation of the actual penetration rate effects.
- Greatest increases in $q_{net}$ occurred between $v=0.2$ and 0.002 mm/s, whereas large reductions in $u_2$ took place as $v$ reduced from 0.2 mm/s to 0.02 mm/s. Similar $q_{net}$ and $\Delta u$ values were measured at penetration rates faster than 2 mm/s.
- The penetrations at $v=0.002$ mm/s confirmed an absence of any excess pore pressure generation ($\Delta u \sim 0$). In some cases $\Delta u$ were approximately zero at
v=0.02 mm/s. Negative $\Delta u$ values measured at v=0.02 mm/s may be due to suctions developing in the sandy layers.

- As expected, $q_{\text{net}}$ and $\Delta u$ values increased with depth at the standard cone rate of 20 mm/s. Stress level dependency in $q_{\text{net}}$ and $\Delta u$ measurements was taken into account by using the piezocone parameters $Q_t$, $B_q$ and $\Delta u/\sigma'_{v0}$.

- A transition from undrained to partially drained conditions tended to occur at $v\sim2$ mm/s based on the $B_q$ and $\Delta u/\sigma'_{v0}$ reductions. However, the $Q_t$ values increased as the velocity reduced down to a $v$ value of about 1 mm/s.

- On the other hand, a transition from partially drained to fully drained conditions can be seen in the velocity range between 0.02 and 0.002 mm/s.

- The undrained normalised resistances ($Q_{UD}$) at $v=20$ mm/s ranged from 4 to 6 as seen before (see Figure 4.6). Some cases with $v=20$ mm/s in the variable rate tests generated lower excess pore pressures. This may be partly due to short travelling distances after the penetrations were halted and resumed.

- The drained normalised resistances ($Q_D$) ranged from 9 to 13, which resulted in the $Q_D/Q_{UD}$ ratios of about 1.6 to 2.3 (average of ~2.0) when the minimum $Q_{UD}$ was used instead of the $Q_t$ value at $v=20$ mm/s.

- The T-bar and the ball penetration indicated a transition from undrained to partially drained conditions at $v\sim0.2$ mm/s. The resistances increased up to about 1.9 to 2.7 (average of ~2.2) times the minimum undrained resistances. These ratios are slightly higher than those seen in the variable rate CPTUs and this is seen to be due to the higher undrained penetration resistance measured in the standard rate CPTUs than the $q_{T\text{-bar}}$ and $q_{\text{ball}}$ values at the standard rate of 20 mm/s.
Figure 4.12 Penetration rate effects at the Bassendean site: (a) $q_{\text{net}}$ and (b) $\Delta u$
Figure 4.13 Penetration rate effects on piezocone parameters at the Bassendean site: (a) $Q_t$, (b) $B_q$ and (c) $\Delta u/\sigma'_{v0}$.
4. Field variable rate penetration test results

4.3.5 Dissipation tests

The horizontal coefficients of consolidation ($c_h$) were assessed from the standard rate piezocone dissipation tests, using the analytical solutions proposed by Teh & Houlsby (1991) and Mayne (2001). The initial maximum excess pore pressures ($\Delta u_{\text{pred}}$) were estimated by assuming a linear reduction in $u_2$ against the square root time after the penetration was halted (see in Figure 4.4b). The excess pore pressures were then normalised by the predicted initial pore pressures ($\Delta u/\Delta u_{\text{pred}}$) and plotted in Figure 4.15a. against the Teh & Houlsby (1991) time factor ($T^*$). Figure 4.15b shows two examples of the pore pressure dissipations and two estimated dissipation profiles based on the Mayne (2001) analytical solution. Mayne (2001) takes into account both shear and octahedral pore pressures using the spherical cavity expansion theory. The solution requires four input parameters: an overconsolidation ratio (OCR), a rigidity index ($I_r$), a friction angle ($\phi'$) and a plastic volumetric strain ratio ($A$). The measured 50% excess pore pressure dissipations were compared with these two analytical solutions to estimate $c_h$ values.

The short delay in reaching the peak pore pressures (~50 seconds) may be related to a slow response of the pore pressure transducer. An assumption of $I_r=150$ was considered to be fairly reasonable in this Bassendean material, since the $I_r$ values between 90 and 150 could be estimated from CAU triaxial compression tests (see Section 3.4.5), where...
\( I_r \) was calculated as the secant shear modulus at 50\% of failure shear stress (\( G_{50} \)) divided by the undrained shear strength (\( s_u \)). Assuming \( I_r=150 \), the \( c_h \) estimates based on Teh & Houlsby (1991) ranged from 66 to 249 m\(^2\)year\(^{-1}\) at depths between 4 m to 9 m. Assuming \( I_r=150 \), OCR=1.5, \( \phi'=28 \) and \( \lambda=0.85 \), the pore pressure dissipations predicted by the Mayne (2001) solution were reasonably fitted to the measured dissipations after the pore pressure peaks apart from the initial rise in \( u_2 \). \( c_h \) values of 56 and 16 m\(^2\)year\(^{-1}\) were estimated at depths of 4 m and 7 m, respectively.

![Graph](image)

**Figure 4.15** Dissipation test results based on (a) Teh & Houlsby (1991) and (b) Mayne (2001) at the Bassendean site

Table 4.2 summarises the dissipation test results using the interpreted time for 50\% dissipation (\( t_{50} \)) and the \( c_h \) estimates based on the Teh & Houlsby (1991) and Mayne (2001) solutions as described above. The \( c_h \) estimates are plotted against depth and compared to the vertical coefficients of consolidation (\( c_v \)) measured in the constant rate

---

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<th>Depth (m)</th>
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<th>Teh and Houlsby (1991)</th>
<th>Mayne (2001)</th>
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<td>249</td>
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<tr>
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<tr>
<td>7.0</td>
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<td>8.0</td>
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\( \Delta u/\Delta u_{pred} \)

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<th>Depth (m)</th>
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<th>Teh and Houlsby (1991)</th>
<th>Mayne (2001)</th>
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<td>9.0</td>
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of strain consolidation (CRSC) tests (see Section 3.4.4) in Figure 4.16. The $t_{50}$ values ranged from 120 to 453 seconds (i.e. about 2 to 8 minutes), which are relatively fast for clays which can often take up to about 100 minutes for 50% dissipation. The $c_h$ estimates based on Teh & Houlsby (1991) were 2.7 to 4.4 times (average of ~3.3 times) higher than that based on Mayne (2001). Interestingly, the $c_h$ values derived using Mayne (2001) were similar in range to the $c_v$ measurements at similar depths (see Figure 4.16). The coefficients of consolidation appear to decrease with depth from 3 m to 7 m and then increase from 7 m to 10 m at the Bassendean site.

Table 4.2 Summary of dissipation tests at the Bassendean site (OCR was set at 1.5)

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<th>$c_h$ Teh and Houlsby (1991) (mm$^2$/s)</th>
<th>$c_h$ Teh and Houlsby (1991) (m$^2$/yr)</th>
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<td>249</td>
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<td>57</td>
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<td>6.2</td>
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<td>63</td>
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</table>

*1 Field test ID corresponds to the testing details (see Section 3.4.2)
Figure 4.16 The coefficients of consolidation estimated in the field and in laboratory.
4.4 BURSWOOD SITE

4.4.1 Standard rate piezocone penetration test

The net cone resistance ($q_{net}$) and the pore pressure ($u_2$) profiles at the standard penetration of 20 mm/s in the Burswood site are shown in Figure 4.17. The average water table at the Burswood site was about 1.3 m below the ground surface but a tidal effect leads to fluctuations and this may have caused light overconsolidation of the near surface layers. The CPTUs at the Burswood site were installed via 2.5 m to 3 m deep predrilled boreholes. Only deep penetrations (below ~6 m) were assessed in this study to avoid any potential influence of predrilling.

Apart from spikes in $q_{net}$ due to either sandy/silty lenses or the existence of shell fragments (as observed in the laboratory tests), $q_{net}$ increased from 220 to 320 kPa from

![Figure 4.17 Standard rate CPTU result at the Burswood site: (a) $q_{net}$, and (b) $u_2$](image-url)
6 m to 10 m while $u_2$ increased from 160 kPa to 250 kPa over the same depth interval (Figure 4.17). One piezocone test conducted by Schneider et al. (2004) about 500 m from this study is also plotted for comparative purposes. The $q_{int}$ values obtained in this study between 6 m and 10 m were about 20 to 70 kPa higher than those reported in Schneider et al. (2004) while the two sets of $u_2$ measurements were in a relatively good agreement. Apart from a potential ground variation between the two testing locations at the Burswood site, experience of the author at this location suggests that tip resistance could be resolved to ±30 kPa provided that the appropriate temperature corrections were made.

The CPTU parameters including normalised tip resistance ($Q_t$), normalised friction ratio ($F_r$), and pore pressure ratios ($B_q$ and $\Delta u/\sigma_{v0}'$), obtained from the standard rate CPTU are shown in Figure 4.18. The $Q_t$ values down to ~5.5 m tended to be higher than those below 6 m possibly due to light overconsolidation caused by predrilling. $Q_t$ ratios of 5 to 5.5, $B_q$ of 0.5±0.05 and $\Delta u/\sigma_{v0}'$ of 2.5±0.2 were found at depths below about 6 m. The $F_r$ ratios of about 2 to 4 were also measured and these indicate that the Burswood material classifies as a silty clay to clay using soil behaviour type index ($I_c$) proposed by Robertson & Wride (1998).

Figure 4.18 CPTU parameters at the Burswood site: (a) $Q_t$ and $F_r$, (b) $B_q$ and $\Delta u/\sigma_{v0}'$

4. Field variable rate penetration test results
Figure 4.19 shows the undrained strength ($s_u$) profile estimated from $q_{net}$ at $v=20$ mm/s and a cone factor ($N_{kt}$), as defined in Equation (4-5). Using the $N_{kt}$ factor of 10.5, the $s_u$ profiles estimated from the CPTU are in good agreement with the $s_u$ measured in the CAU triaxial compression (see Section 3.5.5). $s_u$ increases from 21 to 32.5 kPa between 5.5 m and 11 m depth with a undrained shear strength gradient of about 2.1 kPa/m (using $N_{kt} = 10.5$); 10.5 is within the $N$ factors recommended in Low et al. (2010). The $s_u$ values tend to be higher at the shallower depths above 5.5 m. The $N$ values (defined as the excess pore pressure, $\Delta u$, over the $s_u$ measured in the CAU triaxial compression) are found to be ~5, which is also similarly reported in Low et al. (2010).

![Figure 4.19 Undrained shear strength profile](image)

### 4.4.2 Variable rate piezocone penetration tests

Three ‘twitch’ format variable penetration rate tests were conducted at the Burswood site and summarised in Table 4.3. The employed penetration distances were 72 mm ($2d$), 180mm ($5d$), 500 mm ($14d$) or 1000 mm ($28d$), where $d$ is the cone diameter, as listed in Table 4.3.
Table 4.3 Summary of variable rate tests at the Burswood site

<table>
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<tr>
<th>Field test ID</th>
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<th>Travel distance (mm)</th>
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<td>1000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10.1-10.6</td>
<td>4</td>
<td>500</td>
<td></td>
</tr>
</tbody>
</table>

*1 Field test ID corresponds to the testing details (see Section 3.5.2)

Figure 4.20, 4.21 and 4.22 compare the profiles of $v$, $q_{\text{net}}$ and $u_2$ between the variable penetration rate tests and the standard penetration rate test results. The standard test profile was not measured between 10.5 m and 11.0 m and had to be estimated (Figure 4.20). No measurements were obtained due to the recording problem at around 7 m and the figure shows a dashed line at this location (Figure 4.21).

The following observations can be made:

- It is evident that the $q_{\text{net}}$ reduces by 20% when $v$ reduces from 20 to 0.4 mm/s while the $u_2$ values resulted in a very small rate dependence over the same velocity range between 10.4 m and 11.0 m depth (see Figure 4.20).
- The $u_2$ values reduce significantly below $v=0.2$ mm/s and the $u_2$ values are similar to hydrostatic pressures ($u_0$) when $v$ dropped to 0.002 mm/s at 6 to 7 m. The $q_{\text{net}}$ values accordingly increased due to local consolidation around the tip (see Figure 4.21).
- As seen at the Bassendean site, after an extremely slow test such as one with $v=0.0002$ mm/s, the subsequent rapid acceleration of the cone to $v=2$ mm/s evidently led to the development of suction (negative $\Delta u$).
- Penetration resistances at $v=0.2$ mm/s and 0.02 mm/s are about 25% lower than those at $v=20$ mm/s at depths between 8 m and 10 m (see Figure 4.22).
- The $u_2$ values at $v=0.02$ mm/s between 9m and 10m are only 75% from those at $v=20$ mm/s while $u_2$ values with $v=0.02$ mm/s are only 50% of those at $v=20$ mm/s between 6 m and 7 m depth. This discrepancy may be due to variations in soil permeability and differences in soil composition.
Figure 4.20 Variable rate CPTU test result: BelmCRT1 at 10 to 11 m depth

Figure 4.21 Variable rate CPTU test result: BelmCRT3 at 6 to 7 m depth
As for the other test sites, all of the variable rate CPTUs data at the Burswood site required filtering prior to examination of general trends. The method used to select and average the data was the same as used for the other field tests (see Section 4.3.4). The in situ penetration rate effects at Burswood site (Figure 4.23) showed:

- A relatively large increase in $Q_t$ and a sharp drop in both $B_q$ and $\Delta u/\sigma'_0$ between 0.2 and 0.002 mm/s. Partial consolidation effects are likely to be very significant in this velocity range.
- Undrained penetration conditions may exist at $v>1$ mm/s at 6 to 7 m depth but may be present at $v>0.2$ mm/s at 10 to 11 m depth (according to changes in $B_q$ and $\Delta u/\sigma'_0$).
- As $v$ reduced to 0.002 m/s, $\Delta u/\sigma'_0$ approached zero (i.e. no excess pore pressures generated) and the rate of change in $Q_t$ became smaller. Penetrations can be considered fully drained at $v<0.0002$ mm/s.
- $Q_t$ values tended to increase above $v=0.1$ mm/s especially at the 10 to 11 m depth. This increase was similar to that observed by Schneider et al. (2004) and may be associated with viscosity the in undrained condition. The $Q_t$ data
between 6 and 7 m were more scattered possibly reflecting non-uniform ground conditions.

- A drained normalised penetration resistance \((Q_D)\) of 7.3 was measured at \(v=0.0002\) mm/s. This implies a drained to undrained resistance ratio \((Q_D/Q_{UD})\) of \(~1.6\), when the minimum resistance of \(~4.5\) is used for the \(Q_{UD}\).

- This \(Q_D/Q_{UD}\) ratio is considerably lower than expected based on previous research into rate effects performed in the centrifuge. For example, DeJong & Randolph (2012) suggest the \(Q_D/Q_{UD}\) value of 2.5 for normally consolidated kaolin clay based on experimental (centrifuge) and numerical investigations of rate effects.
Figure 4.23 Penetration rate effects on piezocone parameters at the Burswood site: (a) $Q_t$, (b) $B_q$, (c) $\Delta u/\sigma_{v0}$
4.4.3 Dissipation tests

Eight piezocone dissipation tests were performed at 1 m depth intervals between 3.8-10.9 m to assess the in situ horizontal coefficient of consolidation ($c_h$) based on two proposed analytical solutions. The Teh & Houlsby (1991) solution was employed (see Figure 4.24a) and the initial maximum pore pressures ($\Delta u_{\text{pred}}$) were estimated using the same method used for the other field tests (see Section 4.2.3). The Mayne (2001) solution was also used as employed in the Bassendean site (see Section 4.3.5) and two analysis examples are shown in Figure 4.24b. Table 4.4 summarises all the dissipation test results including $t_{50}$ and corresponding $c_h$ estimates based on Teh & Houlsby (1991) and Mayne (2001).

After penetration ceased for the dissipation tests, there was a delay of ~80 seconds before peak pore pressure was reached. This is possibly due to the slow response of the pore pressure transducer as observed and discussed for the other test sites. It took about 21 to 132 minutes (average of ~53 minutes) to reach 50% dissipation at this site, which suggests that the Burswood soil is more impermeable than the Bassendean soil. The soils at between 10 m and 11 m tended to be more impermeable than shallower soils. The $c_h$ estimates by Teh & Houlsby (1991) ranged from 3.8 to 22 m$^2$/year (average of ~13 m$^2$/year) whereas the $c_h$ estimates by Mayne (2001) ranged from 0.9 to 4.7 m$^2$/year with the input parameters ($I_r=150$, $\phi'=25$, $A=0.8$ and OCR listed in Table 4.4). Slightly higher OCRs were expected at depths shallower than 6 m based on the higher $Q_t$ profile (see Figure 4.18). The Mayne (2001) solution resulted in $c_h$ values that were 4.3 times smaller than those estimated by the Teh & Houlsby (1991).

The $I_r$ value of 150 was used to interpret dissipation test results, whereas the $I_r (=G_{50}/s_u)$ values of between 37 and 148 were obtained on the intact samples in the CAU triaxial compression tests (see Section 3.5.5). Variations depending on $I_r$ of the $c_h$ estimates by Teh & Houlsby (1991) may be expected to vary by less than a factor of 2 since a $c_h$ value calculated with $I_r=35$ is about 0.48 times that calculated with $I_r=150$ (according to Equation (4-6)).
4. Field variable rate penetration test results

Figure 4.24 Dissipation test results based on (a) Teh & Houlsby (1991) and (b) Mayne (2001) at the Burswood site

Table 4.4 Summary of the dissipation test results at the Burswood site (BelmCDT1*)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$t_{50}$ (sec)</th>
<th>$c_h$ Teh and Houlsby (1991) (m²/s)</th>
<th>$c_h$ Mayne (2001) (m²/s)</th>
<th>$c_h$ Mayne (2001) (m²/s)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.82</td>
<td>1730</td>
<td>0.54</td>
<td>17</td>
<td>0.12</td>
<td>3.8</td>
</tr>
<tr>
<td>4.85</td>
<td>4106</td>
<td>0.23</td>
<td>7.3</td>
<td>0.06</td>
<td>1.9</td>
</tr>
<tr>
<td>5.86</td>
<td>2247</td>
<td>0.42</td>
<td>13</td>
<td>0.09</td>
<td>2.8</td>
</tr>
<tr>
<td>6.88</td>
<td>1886</td>
<td>0.50</td>
<td>16</td>
<td>0.11</td>
<td>3.5</td>
</tr>
<tr>
<td>7.89</td>
<td>1264</td>
<td>0.70</td>
<td>22</td>
<td>0.15</td>
<td>4.7</td>
</tr>
<tr>
<td>8.88</td>
<td>1833</td>
<td>0.52</td>
<td>16</td>
<td>0.12</td>
<td>3.8</td>
</tr>
<tr>
<td>9.92</td>
<td>7923</td>
<td>0.12</td>
<td>3.8</td>
<td>0.03</td>
<td>0.9</td>
</tr>
<tr>
<td>10.94</td>
<td>4469</td>
<td>0.21</td>
<td>6.6</td>
<td>0.05</td>
<td>1.6</td>
</tr>
</tbody>
</table>

* Field test ID of BelmCDT1 corresponds to the testing details (see Section 3.5.2)
Figure 4.25 shows profiles of the coefficients of consolidation estimated from the CPTU dissipation tests and those measured in the laboratory (see Section 3.5.4). The vertical coefficient of consolidation ($c_v$) values measured at the in-situ vertical stresses on intact samples are fairly similar to or slightly higher than the $c_h$ estimates by Mayne (2001) and are about 4 times (on average) smaller than the $c_h$ estimates by Teh & Houlsby (1991). The $c_h$ estimates by Teh & Houlsby (1991) at the Burswood site are generally ~10 times lower than those estimated at the Bassendean site (see Section 4.3.5).
4.5 SUMMARY

This Chapter have presented the results of an extensive series of penetration tests at three sites: Gingin, Bassendean and Burswood. Results from piezocene tests conducted at various rates are reported along with data from dissipation tests. The main observations (relevant to these 3 sites) are summarised below:

- Field CPTU parameters (with the standard 10 cm$^2$ cone) were highly dependent on the penetration velocity, especially at velocities that are 10 to 1000 times slower than the standard penetration rate of 20 mm/s.

- Transitions from undrained to partially drained or partially drained to fully drained conditions occurred at different penetration rates, which reflected varying drainage characteristics in the field. Partially drained penetration occurred over two to three orders of penetration velocities.

- Drained normalised penetration resistances (with almost zero excess pore pressures) were achieved even in the clayey deposits. The ratios of drained to undrained penetration resistances ($Q_D/Q_{UD}$) are lower than those expected based on experimental (centrifuge) investigations into penetration rate effects.

- The T-bar and ball penetration test results are comparable to those of the cone and both penetrometers exhibit similar rate dependence. One main difference between the CPTU and the full-flow test results was attributed to their different geometries – as indicated, for example, by the different theoretical $N$ factors in undrained conditions.

- Dissipation test results offered insights into the in situ drainage characteristics and can assist further evaluate of penetration rate effects. The horizontal coefficients of consolidation differed by a factor of 10 in the Bassendean and Burswood sites, despite their comparable geological histories and locations. The effect of drainage characteristics on penetration rate dependency is further discussed in Chapter 8.
CHAPTER 5  VARIABLE RATE  
MINIATURE CPT IN CLAYEY  
SANDS USING DRUM  
CENTRIFUGE

5.1  INTRODUCTION

This Chapter presents results from a series of cone penetration tests in clayey sand mixtures conducted in the drum centrifuge facility at the University of Western Australia (UWA). A centrifuge facility is a major physical modelling tool and widely used for geotechnical research purposes. The main advantage of centrifuge testing is the ability to model the in situ stress field, and hence obtain results which are representative of full scale tests. The investigation of cone penetration rate effects in centrifuge facilities has increased since 1990s (as reviewed in Chapter 2). In particular, recent research including that Schneider et al. (2007) and Jaeger et al. (2010) reported unique characteristics of penetration rate effects in intermediate soils.

This Chapter explains the experimental apparatus and sample preparation procedure and then describes the CPT test results. Penetration rate effects in the clayey sand mixtures are discussed in terms of fines content, drainage characteristics and soil density. The test results are compared with the CPT test results in clean sand and kaolin clay from the previous centrifuge studies reported in the literature.

Some of the centrifuge test results presented in this Chapter have been published in Suzuki & Lehane (2014b); Appendix D.
5. Variable rate miniature CPT in clayey sands using drum centrifuge

5.2 TEST APPARATUS

5.2.1 Drum centrifuge at UWA

The centrifuge is known as very useful physical modelling apparatus as centrifuge modelling replicates in situ stress of field scale problems. Stress conditions of the soil sample in the centrifuge are factored by the gravitational acceleration ($N$ times the normal gravity, $g$). Accordingly, a stress condition at $z$ depth in the centrifuge corresponds to a stress condition at $N \times z$ depth in the field. The details of the scaling laws between prototype and centrifuge model scale are explained by Taylor (1995) and summarised in Table 5.1. In fact, the gravitational acceleration factor $N$ varies depending on a radius in the soil model. The inertial acceleration is given by $\omega^2 r$ defined as Equation (5-1), where $\omega$ is the angular rotational speed of the centrifuge and $r$ is the radius to any element in the soil model.

$$N \cdot g = \omega^2 \cdot r$$  \hspace{1cm} (5-1)

It is important to consider this $N$ variation with depth, especially for a comparison with the prototype model, where the normal earth gravity ($g$) produces linear increase in stress condition with depth.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Prototype</th>
<th>Model scale</th>
<th>Scaling factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravity</td>
<td>1g</td>
<td>$Ng$</td>
<td>$N$</td>
</tr>
<tr>
<td>Length</td>
<td>1</td>
<td>1/$N$</td>
<td>1/$N$</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Force</td>
<td>1</td>
<td>1/$N^2$</td>
<td>1/$N^2$</td>
</tr>
<tr>
<td>Mass</td>
<td>1</td>
<td>1/$N^3$</td>
<td>1/$N^3$</td>
</tr>
<tr>
<td>Density</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 5.1a is a photograph of the drum centrifuge installed at UWA. The drum centrifuge has a sample containment channel which is 300 mm in height and 200 mm in radial depth. The channel has a diameter of 1.2 m. A full description of this centrifuge is provided by Stewart et al. (1998).
This drum centrifuge has a smaller radius than many beam centrifuge facilities, although the plan area of sample available for testing is significantly larger than that of beam facilities. For this centrifuge study, an effective centrifuge radius \( R_e \) of 522 mm was set as a reference radius for ‘\( N \)’ acceleration during spinning for full consolidation. This meant that for a sample height of 140 mm under \( N \) of 100, actual gravitational acceleration varied between about 83 and 110 between the surface and base of the sample. This \( N \) variation was taken into account when the test data were processed.

The drum centrifuge allows samples to be prepared by pouring slurry directly into the channel, which allows multiple testings in 360 degrees. However, in this study, rather than using the full available channel space, six small rectangular strong boxes were located within the channel. Each of these housed a different type of sample that had been prepared outside of the centrifuge. The box dimensions and locations of the boxes in the centrifuge are explained later in Section 5.3.2.

![UWA drum centrifuge](image1.png)  ![Mini epoxy cone penetrometer](image2.png)

Figure 5.1 Photographs of (a) UWA drum centrifuge without protective cover and (b) Miniature epoxy cone penetrometer mounted in position

### 5.2.2 Miniature epoxy cone penetrometer

The miniature epoxy cone penetrometer, previously built in house, was used for this study (see Figure 5.1b). This CPT is made of epoxy in order to minimise the diameter of the penetrometer rod whilst retaining rigidity of the penetrometer when it is tested in sand. The fabrication method has been presented in detail by Lee et al. (2012), which also reports the design method for an epoxy ball penetrometer. The miniature CPT in
this project has a diameter of 6 mm and a cone apex angle of 60°. Only tip resistance ($q_c$) is measured i.e. there is no friction sleeve. The device was fabricated in one piece so that $q_c = q_t$ i.e. no correction for pore pressure is required. The internal load cell strain gauge for the tip resistance measurement was calibrated against an external reference load cell. Figure 5.2 shows the calibration result, in which a factor of 265.25 N/volt was determined by a first order linear regression method.

![Calibration result for CPT tip resistance](image)

Figure 5.2 Calibration of the strain gauge for CPT tip resistance

### 5.3 SAMPLE PREPARATION

#### 5.3.1 Clayey sand mixtures

Mixtures of commercially available fine silica sand and kaolin clay were used to prepare the test samples. Varying proportions of kaolin clay (by dry weight) to the host sand were prepared; including 25% kaolin & 75% sand, 10% kaolin & 90% sand, and 5% kaolin & 95% sand mixtures. The mixtures are referred to as 25%, 10% and 5% kaolin mixtures in the following sections.

Particle size distribution curves (PSD), limiting void ratios ($e_{\text{max}}$ and $e_{\text{min}}$), and Atterberg limits (PL, LL, PI) on the three clayey sand mixtures were reported in Section 3.6.2. In addition, a series of 1D Rowe cell consolidation tests and simple shear tests were also conducted and described in Sections 3.6.3 and 3.6.4.
5.3.2 Pre-consolidation

The three clayey sand mixtures were prepared at a water content of twice the liquid limit of each mixture, and then poured into the six small strongboxes. The slurry samples in the boxes were pre-consolidated at 1g prior to consolidation under high acceleration in the centrifuge. There were two main reasons for selection of this preparation method; (i) to save consolidation time in the centrifuge and (ii) to minimise the potential risk of clay and sand particle segregation under high acceleration.

The test samples were prepared at 1g as follows:

(i) Filter fabrics were placed inside the box to assist upwards drainage (Figure 5.3a). A drainage hole at the base of the box was kept closed during pre-consolidation to maintain saturated conditions.

(ii) The slurry samples were poured slowly into the box up to about 5 mm below the top of the box (Figure 5.3b).

(iii) A filter paper and a filter fabric were then placed onto the soil surface before applying any surcharge.

(iv) A wrapped and greased timber was used to distribute the weight evenly over the sample. The weights were gradually increased to achieve a pre-consolidation stress of about 20 kPa (Figure 5.3c). Due to this pre-consolidation pressure, it was expected that the surface of the samples had been lightly over consolidated.

During pre-consolidation, the slurry samples were topped up accordingly when significant settlement occurred (depending on compressibility of the samples). As a result, no topping-up was required in the centrifuge. Figure 5.3d shows the six samples at completion of the pre-consolidation stage. Table 5.2 summarises the relevant information on sample preparation at 1g.
5. Variable rate miniature CPT in clayey sands using drum centrifuge

Table 5.2 Summary of clayey sand samples prepared in six boxes after pre-consolidation

<table>
<thead>
<tr>
<th>Box number</th>
<th>Box effective inside dimension w/h/d (mm)*1</th>
<th>Sample composition*2</th>
<th>Pre-consolidation pressure (kPa)</th>
<th>Sample weight (kg)</th>
<th>Sample height (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Box1</td>
<td>80/250/157</td>
<td>5%K+95%S</td>
<td>22.5</td>
<td>5.91</td>
<td>142</td>
</tr>
<tr>
<td>Box2</td>
<td>80/245/157</td>
<td>5%K+95%S</td>
<td>27.9</td>
<td>5.71</td>
<td>142</td>
</tr>
<tr>
<td>Box3</td>
<td>80/250/157</td>
<td>10%K+90%S</td>
<td>22.4</td>
<td>6.22</td>
<td>147</td>
</tr>
<tr>
<td>Box4</td>
<td>80/250/157</td>
<td>10%K+90%S</td>
<td>22.5</td>
<td>6.20</td>
<td>148</td>
</tr>
<tr>
<td>Box5</td>
<td>80/250/157</td>
<td>25%K+75%S</td>
<td>22.5</td>
<td>6.30</td>
<td>149</td>
</tr>
<tr>
<td>Box6</td>
<td>80/250/157</td>
<td>25%K+75%S</td>
<td>22.5</td>
<td>6.26</td>
<td>150</td>
</tr>
</tbody>
</table>

*1 width/height/depth
*2 K and S stand for kaolin and fine sand respectively

Figure 5.3 Samples preparation in pre-consolidation stage at 1g: (a) assembled box, (b) pouring of sample, (c) surcharge application and (d) samples after pre-consolidation

5.3.3 Consolidation in the centrifuge

After pre-consolidation at 1g, all six boxes were placed into the drum centrifuge channel. This required the sample to self-stand vertically with its surface facing the centre of the centrifuge. Suction was applied to minimise the potential for collapse of the samples in this vertical position. This was achieved by opening the bottom drainage hole and then
Variable rate miniature CPT in clayey sands using drum centrifuge

Applying suction at the base using a low pressure vacuum pump; see Figure 5.4a. The boxes were then carefully placed in the channel (Figure 5.4b). The locations of the six boxes inside the drum channel were as shown in Figure 5.4c. After placement of the six boxes, the central tool table with the actuator was positioned inside the drum channel, and finally a protective enclosure was placed over the centrifuge for safety purpose.

Unfortunately, it was observed that the Box4 sample (one of the 10% kaolin mixture samples) had slumped on one side of the box before the g-forces could be applied; see Figure 5.4b. To avoid any influence from this disturbance, CPT test locations were moved well away from this zone.

An acceleration of 20g was used while the drum channel was being filled with water. The whole channel was filled with water to fully saturate all samples and to improve the balance of the centrifuge when spinning. In addition, the balance was improved by
adding some water (as weights) into four water containers, which are attached underneath the channel. Once all samples were fully saturated, the centrifuge was ramped up to 100g for full consolidation and a reference radius of 522 mm was set (see Section 5.2.1). In general, observed settlements after 100g was applied were relatively small. Settlements were more significant in the 25% kaolin mixtures (Box5 and Box6) because of their higher clay compositions (and thus higher compressibility) but settlements were virtually completed after one day of consolidation.

Consolidation under 100g was undertaken for about 70 hours before CPT testing began. The time required for 90% degree of consolidation \( (t_{90}) \) can be estimated as:

\[
t_{90} = \frac{T_{90}d^2}{c_v}
\]

where \( T_{90} \) is the time factor (=0.848) for 90% degree of consolidation and \( d \) is the drainage length. Assuming a one-way drainage length of 150 mm in the small strongbox, and a \( c_v \) of 22 m²/year (measured at the effective vertical stress of 50 kPa in the 25% kaolin mixture in 1D Rowe cell consolidation, see Section 3.6.3), a \( t_{90} \) of 7.6 hours is calculated. Pre-consolidation at 1g also assisted in reducing the time for consolidation in the centrifuge. Based on the foregoing, it could be inferred that all samples were fully consolidated after the period of 70 hours allowed before testing.

5.3.4 Testing details

The CPT testing schedule is listed in Table 5.3, which also gives the corresponding penetration velocity and target depth of each test. A total of 17 CPTs were conducted; 13 tests were performed at an acceleration of 100g and 4 tests were at 200g in one of the 5% kaolin mixture (Box2). Most of the tests were conducted to a maximum penetration of 110 mm, which is about 5\( d \) (i.e. 5 times of the cone diameter) away from the bottom of the box. Penetration rates were varied between 0.001 and 3 mm/s at the focus depth range between 70 and 100 mm.

Testing locations for the CPTs in each box are indicated in Figure 5.5. All the CPTs were conducted along the centre line of each box, giving a distance of 37 mm (i.e. about 6\( d \)) to the side walls of the box. In general, 4 CPTs were conducted in each box with a distance of 44 mm (i.e. about 7\( d \)) between the CPTs. In some cases, CPTs were located at shorter distances (for Box4-a to 4-d and Box6-e) as illustrated in the figure. Notations
of the CPT locations in Figure 5.5 correspond to ‘Box-Test location’ as listed in Table 5.3.

### Table 5.3 Schedule of cone penetration testings

<table>
<thead>
<tr>
<th>Time (hours)</th>
<th>Box-Test location</th>
<th>Velocity (mm/s)</th>
<th>Input travel distance (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>100g achieved</td>
<td></td>
<td></td>
</tr>
<tr>
<td>70.0</td>
<td>1-a</td>
<td>3</td>
<td>112</td>
</tr>
<tr>
<td>70.9</td>
<td>1-b</td>
<td>0.3</td>
<td>112</td>
</tr>
<tr>
<td>71.3</td>
<td>3-a</td>
<td>3</td>
<td>120</td>
</tr>
<tr>
<td>71.9</td>
<td>3-b</td>
<td>0.1</td>
<td>120</td>
</tr>
<tr>
<td>72.2</td>
<td>3-c</td>
<td>0.3-0.006</td>
<td>80-110</td>
</tr>
<tr>
<td>73.9</td>
<td>1-c</td>
<td>0.3-0.006</td>
<td>60-90</td>
</tr>
<tr>
<td>75.6</td>
<td>4-a</td>
<td>0.3</td>
<td>110</td>
</tr>
<tr>
<td>75.8</td>
<td>4-b</td>
<td>3-0.03</td>
<td>40-100</td>
</tr>
<tr>
<td>76.5</td>
<td>4-c</td>
<td>1</td>
<td>110</td>
</tr>
<tr>
<td>76.7</td>
<td>5-a</td>
<td>3</td>
<td>118</td>
</tr>
<tr>
<td>76.8</td>
<td>5-b</td>
<td>0.3</td>
<td>115</td>
</tr>
<tr>
<td>77.1</td>
<td>6-a</td>
<td>0.1</td>
<td>118</td>
</tr>
<tr>
<td>77.6</td>
<td>6-b</td>
<td>0.3-0.003</td>
<td>40-110</td>
</tr>
<tr>
<td>93.4</td>
<td>5-c</td>
<td>3-0.03</td>
<td>50-110</td>
</tr>
</tbody>
</table>

### Table 5.3 Schedule of cone penetration testings (Cont.)

<table>
<thead>
<tr>
<th>Time (hours)</th>
<th>Box-Test location</th>
<th>Velocity (mm/s)</th>
<th>Input travel distance (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>94.3-97.6</td>
<td>ramping down, draining water, flipping over Box1-6 (not Box2), ramping up to 50g, saturating samples, ramping up to 100g</td>
<td></td>
<td></td>
</tr>
<tr>
<td>97.6</td>
<td>1-d</td>
<td>3-0.03</td>
<td>50-110</td>
</tr>
<tr>
<td>98.3</td>
<td>3-d</td>
<td>3</td>
<td>120</td>
</tr>
<tr>
<td>98.6</td>
<td>4-d</td>
<td>3-0.01</td>
<td>50-110</td>
</tr>
<tr>
<td>100.4</td>
<td>6-c</td>
<td>1</td>
<td>114</td>
</tr>
<tr>
<td>100.6</td>
<td>6-e</td>
<td>3</td>
<td>114</td>
</tr>
<tr>
<td>100.7</td>
<td>5-d</td>
<td>0.3-0.001</td>
<td>50-110</td>
</tr>
<tr>
<td>117.6</td>
<td>6-d</td>
<td>3-0.01</td>
<td>50-110</td>
</tr>
<tr>
<td>121.4-124.0</td>
<td>ramping down, draining water, removing Box3-6, ramping up to 50g, saturating samples, ramping up to 200g (left for 1.5 hrs)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>124.2</td>
<td>2-a</td>
<td>3</td>
<td>113</td>
</tr>
<tr>
<td>124.7</td>
<td>2-b</td>
<td>3-0.03</td>
<td>50-110</td>
</tr>
<tr>
<td>125.4</td>
<td>2-c</td>
<td>0.3</td>
<td>113</td>
</tr>
<tr>
<td>125.6</td>
<td>2-d</td>
<td>0.3-0.001</td>
<td>50-110</td>
</tr>
</tbody>
</table>

Figure 5.5 Locations of CPTs in the strongboxes (plan view)
5. Variable rate miniature CPT in clayey sands using drum centrifuge

5.3.5 Processing data

Figure 5.6 shows one typical example of the CPT penetration and extraction profile (Box5-d), where the penetration velocity was changed from 3 mm/s to 0.001 mm/s at a depth of about 45 mm. In some cases after very slow penetrations were used, it was noticed that the ‘zero’ tip resistance had shifted after extracting the cone penetrometer from the soil surface. This zero shift was not evident in quick testing at relatively fast penetration and extraction rates. Further examination revealed a higher than anticipated temperature dependency of the strain gauge measuring the tip resistance. As no measurements of temperatures were obtained, two correction methods were proposed to minimise errors (Figure 5.6). One method, referred to as a linear correction, is to simply assume that the ‘zero’ shifting of the strain gauge was proportional to elapsed time during slow penetration. The second method, called the square root correction, was adopted as it led to far more consistent tip resistance and was more in keeping with anticipated rates of temperature change. An illustration of the assume time dependency of the strain gauge shift is shown on Figure 5.6. Both the linear and square root methods are compared on Figure 5.6.

All penetration data were recorded at 10 Hz and then averaged over about 1 mm model depth intervals for presentation purposes.

![Correction for zero shifting of tip resistance measurement](image.png)
5.4 PENETRATION TEST RESULTS AND ANALYSIS

5.4.1 Penetration test profiles

The cone penetration profiles for the 5%, 10% and 25% kaolin mixtures are presented in Figure 5.7, Figure 5.8 and Figure 5.9 respectively. The imposed velocity ($v$) profiles are given the symbols (a) to (d) in the figures and corresponding cone tip resistance ($q_t$) and normalised tip resistance ($Q=(q_t-\sigma_v)/\sigma'_{v0}$) profiles are annotated accordingly. The CPT data recorded in the tests are summarised in Appendix C. The vertical coefficients of consolidation ($c_v$) values for the normally consolidated soils obtained in the Rowe cell tests are also included in the table.

The observations from the tests are summarised as follows:

The 5% kaolin mixture

- The $q_t$ values almost followed a linearly increasing trend with depth under both 100g (Box1) and 200g (Box2). The $q_t$ values increased up to about 2.5±0.3 MPa under 100g at the model depth of ~100 mm whereas the $q_t$ values increased up to about 6.0±0.7 MPa under 200g at the same depth.
- The soil layer at depths between 40 and 60 mm in Box1 was apparently denser as the rate of change of $q_t$ with depth is greater (and hence $Q$ is greater) than that at the other depths.
- The first penetration test at $v$=3 mm/s in Box2 (Box2-a) shows slightly lower resistances than others. The sample may not have reached full consolidation at this stage immediately after the spinning was ramped up to 200g (see Table 5.3).
- A variation in soil density is inferred for the test at $v$= 0.3 mm/s in Box2 (Box2-c) at the depth below 50 mm, where $Q$ values are slightly higher than in other tests.
- The $q_t$ values tend to drop immediately after the penetration velocities are changed. For example, at $v$=0.03 mm/s (Box1-d), the $q_t$ reduced over 15 mm of advancement after the velocity changed at the 50 mm depth. However, the $q_t$ values recorded at $v$=0.001 mm/s (Box2-d) at the deeper depths increased back to the same $Q$ values as those measured at $v$=0.3 mm/s at the shallower depths.
No clear penetration rate dependency can be concluded. In general, the $Q$ values of 33±5 (with slightly higher values for 200g) were obtained for all penetration velocities tested at the 70-100 mm depth.

Figure 5.7 CPT penetration profiles for the 5% kaolin mixtures: (a) Box1 and (b) Box2
The 10% kaolin mixture

- The penetration test results in Box4 vary significantly even at the shallower depths when compared to Box3. This is believed to be caused by different densities and complex stress conditions arising from the collapse referred to previously (see Figure 5.4b). No further data analysis for Box4 was made for the rate effects investigation.
- In Box3, the $q_t$ values increased rapidly with depth down to about 40 mm and then increased again below 70 mm, while a constant $q_t$ value of about 170 kPa was obtained at the depths between 40 and 70 mm.

![Figure 5.8 CPT penetration profiles for the 10% kaolin mixtures: (a) Box3 and (b) Box4](image-url)
• The trend at the shallow depths was expected as a result of the applied preconsolidation pressure of about 20 kPa prior to consolidation in the centrifuge.

• There was no obvious rate dependency of \( q_t \) in the velocity range tested except for \( v=0.006 \text{ mm/s} \) case (Box3-c), which showed a slightly lower resistance than that recorded at \( v=0.1 \) and 3 mm/s.

• The \( Q \) values in the normally consolidated region varied between about 20 and 30, which were slightly lower than those obtained in the 5% kaolin mixture.

The 25% kaolin mixture

• The \( q_t \) values at shallow depths for the 25% kaolin mixtures were almost 10 times less than those recorded in the 10% kaolin mixtures. It was noted that the CPT result of Box5-a measured extremely low \( Q \) values (even less than zero) and are not considered reasonable (see Figure 5.9a).

• The tendency for \( Q \) to reduce from the soil surface is consistent with the reducing OCR of the samples associated with the pre-consolidation pressure of 20 kPa in both Box5 and Box6.

• Below the depths of 50 mm where different penetration rates were imposed, a clear rate effect is evident for both \( q_t \) and \( Q \) profiles. For example, in the Box6-d profile (see Figure 5.9b), the \( q_t \) value recorded at \( z=100 \text{ mm} \) and \( v=0.01 \text{ mm/s} \) is about 4 times the \( q_t \) value recorded at the corresponding depth at \( v=1 \text{ mm/s} \) in the Box6-c profile.

• One additional penetration test at \( v=3 \text{ mm/s} \) (Box6-e) was conducted at the location between Box6-b and Box6-c (as illustrated in Figure 5.5). The \( q_t \) and \( Q \) values were generally similar to those at \( v=1 \text{ mm/s} \) (Box6-c), but were not same as those at the same rate (Box5-a).
5. Variable rate miniature CPT in clayey sands using drum centrifuge

Figure 5.9 CPT penetration profiles for the 25% kaolin mixtures: (a) Box5 and (b) Box6
5.4.2 Rate dependency of cone resistance

Figure 5.10 presents a summary of the variable rate penetration test data (in terms of $q_t$ and $Q$) for the three clayey sands investigated. The data at the depths between about 70 and 100 mm were selected (as indicated by symbols in the penetration profiles), and these are considered representative of steady state penetration resistance values in normally consolidated soils. The effective vertical stresses ($\sigma''_v$) at these depths were between 60 and 95 kPa under 100g and between 125 to 185 kPa under 200g.

The resistances measured by the 5% and 10% kaolin mixtures were clearly not dependent on the penetration rate over the investigated velocity range of 0.001 to 3 mm/s. In the 5% kaolin mixture, the $Q$ values varied by about 15% (i.e. $Q \sim 33 \pm 5$) but no clear systematic trend of rate dependency can be inferred. The same inference is made at both stress levels induced by 100g and 200g, for which $Q$ values differ on average by less than about 10%.

The normalised resistances in the 10% kaolin mixture were about $Q \sim 25 \pm 5$, which are slightly less than those of the 5% kaolin mixture ($Q \sim 33 \pm 5$). The $Q$ values decreased by about 20% when the penetration rate was reduced from 0.1 to 0.006 mm/s in the 10% kaolin mixture. Although this trend is somewhat surprising, a similar trend has been reported for clean saturated silica sand in centrifuge tests by Xu (2007).

In contrast to these two lower clay content mixtures, the 25% kaolin mixture showed a dramatic increase in $q_t$ when the penetration rate was reduced below 1 mm/s. The $Q$ values remained almost constant at about 12 at velocities below 0.01 mm/s. This $Q$ value of 12 is about 8.6 times higher than the average $Q$ value recorded at $v>1$ mm/s. It is also evident that the variation of $q_t$ in the 25% kaolin mixture at the various stress levels in the normally consolidated region can be unified using the normalised $Q$ term (although the scatter is larger at $v=0.1$ and 0.3 mm/s).
Figure 5.10 Effect of penetration rate on penetration resistance in three clayey sands: (a) $q_t$ and (b) $Q$

Normally consolidated at about 70 to 100 mm depth

\[ Q = \frac{q_{\text{cent}}}{\sigma' v_0} \]
5. Variable rate miniature CPT in clayey sands using drum centrifuge

5.4.3 Use of normalised velocity

The penetration rate dependency for the three clayey sand mixtures are now compared in Figure 5.11, using a normalised velocity ($V_v$) term as proposed by Finnie & Randolph (1994), defined as:

$$V_v = \frac{v d}{c_v}$$  \hspace{1cm} (5-3)

where $v$ is the cone penetration rate, $d$ is the cone diameter (=6 mm for this miniature CPT) and $c_v$ is the vertical coefficient of consolidation measured in a 1-D consolidation test at the same overconsolidation ratio.

The $c_v$ values for each clayey sand mixture at stress levels similar to those induced in the centrifuge are summarised in Table 5.4. The Rowe cell consolidation test results have been reported in Section 3.6.3.

Figure 5.11 also shows the CPT results previously reported from other experimental studies into rate dependence on penetration resistance, including the studies in kaolin clay by Randolph & Hope (2004) and Schneider et al. (2007), and those in two similar clayey sands (i.e. sands with 25% kaolin) by Kim et al. (2010) and Jaeger et al. (2010). In addition, the CPT results in dry silica sand in the UWA drum centrifuge as reported in Lehane & White (2005) and Schneider & Lehane (2006) are also included for comparative purposes. The normally consolidated $c_v$ values used for normalising velocities were obtained from Rowe cell consolidation tests, as reported in Randolph & Hope (2004), Schneider et al. (2007) and Jaeger et al. (2010) or obtained from a flexible-wall permeameter tests in Kim et al. (2010).

Table 5.4 Summary of $c_v$ values for three clayey sands mixtures

<table>
<thead>
<tr>
<th>$\sigma^\prime_v$ (kPa)</th>
<th>Coefficient of consolidation, $c_v$ (m$^2$/year [m$^2$/s])</th>
<th>5% K+95%S</th>
<th>10% K+90%S</th>
<th>25% K+75%S</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>5% K+95%S</td>
<td>10% K+90%S</td>
<td>25% K+75%S</td>
</tr>
<tr>
<td>60</td>
<td>35173 [1.1x10$^{-3}$]</td>
<td>22520 [7.1x10$^{-3}$]</td>
<td>30 [9.5x10$^{-7}$]</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>41282 [1.3x10$^{-3}$]</td>
<td>28736 [9.1x10$^{-3}$]</td>
<td>56 [1.8x10$^{-6}$]</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>46249 [1.5x10$^{-3}$]</td>
<td>34162 [1.1x10$^{-3}$]</td>
<td>87 [2.7x10$^{-6}$]</td>
<td></td>
</tr>
<tr>
<td>180</td>
<td>54281 [1.7x10$^{-3}$]</td>
<td>43593 [1.4x10$^{-3}$]</td>
<td>161 [5.1x10$^{-6}$]</td>
<td></td>
</tr>
</tbody>
</table>
The following observations may be made:

- It appears from Figure 5.11 that, for all data sets, the transition from undrained to partially drained condition occurs at about $V_v \sim 10$ and from partially drained to drained condition is at about $V_v \sim 0.06$.
- The magnitude of the resistances recorded in the 25% kaolin mixture in this study are in relatively good agreement with those reported by Jaeger et al. (2010) at $V_v < 5$ (noting that Jaeger et al. (2010) reported some $Q$ values of less than 1 in the undrained region).
- The corresponding results from Kim et al. (2010) are also similar except for $V_v > 1$ (i.e. close to or in the undrained region) where the $Q$ values are higher than other 25% kaolin mixtures in this study and those in Jaeger et al. (2010). This discrepancy may have resulted from different soil compositions (e.g. host sands) and sample preparation methods - noting that the centrifuge was used for this study and for that of Jaeger et al. (2010), whereas a pressure chamber was used by Kim et al. (2010).
- The undrained strength ratios ($s_u / \sigma'_v$) for the 25% kaolin mixture equate to 0.08 to 0.10 with a $N_{kt}$ cone factor of 15 or equal to 0.13 to 0.16 with $N_{kt}=10$; the latter values are more comparable to expectations for ($s_{uds}/\sigma'_vc$)$_{NC}$ of about 0.18 obtained in the simple shear tests (see Section 3.6.4)
- A notable feature of all the measurements obtained for the 25% kaolin mixtures is the tendency for lower undrained normalised tip resistance ($Q_{UD}$) values than those reported in kaolin clay. This study recorded an average $Q_{UD}$ value of 1.4 compared to $Q_{UD}$ values for kaolin clay of between 2.6 and 3.4 reported by Randolph & Hope (2004) and Schneider et al. (2007).
- This lower $Q_{UD}$ in the 25% kaolin mixture is consistent with the more contractant fabric than that of 100% kaolin clay under undrained sheared (see Section 3.6.4).
- The drained $Q$ values ($Q_D$) of ~12 observed in the 25% kaolin mixture are higher than the maximum $Q$ values of 6.3 and 7.1 reported in kaolin clay by Randolph & Hope (2004) and Schneider et al. (2007) respectively. The higher friction angle and the low compressibility of the 25% kaolin mixture compared to pure kaolin are likely to have contributed to the higher drained resistance.
The average drained to undrained ratio \( (Q_D/Q_{UD}) \) of ~8.6 in the 25% kaolin mixture contrasts with the ratio of ~17 suggested by Jaeger et al. (2010) and the ratio of ~6 reported by Kim et al. (2010). Differences in these ratios are largely associated with the different \( Q_{UD} \) values observed.

Penetration tests at the velocity range conducted in this study for the 5% and 10% kaolin mixtures appear to be fully drained.

The average \( Q_D \) values of ~33 and ~25 for the 5% and 10% kaolin mixtures respectively are likely to be associated with their higher stiffness and strength compared to the 25% kaolin mixture.

The \( Q_D \) values obtained in the 5% and 10% kaolin mixtures are lower than those measured by Lehane & White (2005) and Schneider & Lehane (2006) in dry silica sands at relative densities \( (D_r) \) ranging between 0.45 and 0.9. Schneider & Lehane (2006) proposed the following expression to derive \( D_r \) in silica sand through UWA drum centrifuge experience. The proposed expression is given as:

\[
D_r = \sqrt{\frac{Q}{250}}, \quad \text{where } Q = \frac{q_{net}}{\sigma' v_0}.
\]

The densities of the clayey sand mixtures in this study are discussed in the next section.

Figure 5.11 Rate dependency of normalised cone resistance versus normalised velocity
5.4.4 Soil density

Tube steel samplers with a diameter of 38 mm were used to extract soil samples after the strong boxes were removed from the centrifuge. Using the extracted samples, average (along the whole depth) effective saturated unit weights ($\gamma'$) of 10.1 and 10.5 kN/m$^3$ (i.e. average void ratios ($e$) of \(~0.60\) and \(~0.54\)) were derived in the 5% and 10% kaolin mixtures, respectively. These values are expected to be approximate values because the soil samples were collected after the spinning had been stopped i.e. the measurements were not recorded during spinning under high acceleration levels.

Withstanding obvious complications associated with characterisation of clayey sands, based on the inferred limiting void ratios given in Section 3.6.2, ($e_{\text{max}}$ and $e_{\text{min}}$), the measured unit weights imply relative densities ($D_r$) of \(~0.68\) and \(~0.82\) for the 5% and 10% kaolin mixtures respectively. Although the use of $D_r$ for these materials is contentious, this calculation indicates that the sand in the 5% and 10% kaolin mixtures is in a medium dense to dense state. These apparent $D_r$ are similar to the $D_r$ values of 74 to 76% (calculated in the same way) of the specimens tested in the simple shear tests; see Section 3.6.4.

Using the relationship of Schneider & Lehane (2006), the $Q_D$ values in the ‘medium dense to dense’ 5% and 10% kaolin mixtures are generally similar to those in ‘loose’ state fine silica sand with a relative density of 35%. Evidently a small amount of kaolin content strongly influences the penetration resistance. It is inferred that, although fines (i.e. kaolin clay fraction) in the voids can lead to a higher density, this increased density does not translate to increased penetration resistance as the sand-sand skeleton is stronger than that which can be produced from a combination of fine and coarse particles.

The moisture contents were measured in the 25% kaolin mixture and the corresponding $\gamma'$ values were calculated showing an increase with depth from about 10.72 to 11.03 kN/m$^3$, (i.e. void ratios ($e$) of between 0.46 and 0.50). This $e$ range for the 25% kaolin mixture is comparable to that of the 25% kaolin specimens in the simple shear tests but is much smaller than kaolin clay. The lower $Q_{UD}$ values measured in the 25% kaolin mixture compared to the kaolin clay cannot be explained in terms of soil density. It is more likely due to a more contractant structure in the 25% kaolin mixture, which is
supported by the response observed in the undrained simple shear tests (see Section 3.6.4).

### 5.4.5 Evaluation of effective strength

The effective friction angle ($\phi'$) was evaluated for the three clayey sand mixtures using the tip resistance data following Kulhawy & Mayne (1990). These authors proposed a relationship between the peak triaxial compression friction angle ($\phi'_p$) and a normalised tip resistance ($Q_{tn}$) from CPT data for unaged, uncemnted, clean quartz to siliceous sand, defined as:

$$\phi'_p = 17.6 + 11 \log(Q_{tn}) \quad (5-4)$$

The normalised tip resistance ($Q_{tn}$) takes into account of the stress exponent ($n$) which varies with soil type and stress level, defined as (Robertson 2009):

$$Q_{tn} = \left( \frac{q_t - \sigma'_{vo}}{p_a} \right) \left( \frac{p_a}{\sigma'_{vo}} \right)^n \quad (5-5)$$

where $p_a$ is the atmospheric pressure in the same unit as $q_t$ and $\sigma'_{vo}$, and $n$ can be estimated by an iteration process based on the soil behaviour type index ($I_c$) proposed by Robertson & Wride (1998).

In the 5% kaolin mixture, assuming $n=0.5$ (as no friction sleeve stresses were measured by the miniature CPT device), the estimated $Q_{tn}$ values generally range between 22 and 46 (average of ~33) implying $\phi'_p$ angles of 32 to 36° (average of ~34°). These angles are slightly higher than the apparent $\phi'_p$ values of between about 28 and 34° (average of ~32°) obtained in the simple shear tests (see Section 3.6.4).

In the 10% kaolin mixture, the average $Q_{tn}$ of ~19 and the average $\phi'_p$ of ~32° was inferred from Equations (5-4) and (5-5) by assuming $n=0.5$. Using $n=1.0$ for the 25% kaolin mixture, since it behaves more like a natural clay, the $Q_{tn} (=Q_{D} \sim 12)$ implies a $\phi'_p$ value of ~29°. In the clayey sand mixtures tested in this study, these estimated $\phi'_p$ values are comparable to the apparent $\phi'_p$ values from the simple shear tests. It should be noted that no fines content correction was applied in the interpretation above because no friction sleeve measurements were available from the CPT device.
5.4.6 Potential undrained penetration behaviour in clayey sands

In the centrifuge test, undrained conditions were not achieved in the relatively dense clayey sands i.e. the 5% and 10% kaolin mixtures tend to be drained based on the normalised velocities ($V$) (see Figure 5.11). Based on the observations of Silva (2005), due to strong effects in the partially drained and undrained regions, it would appear that penetration resistances could increase when extremely fast penetration velocities are used in relatively dense sands with a small clay fraction. This trend is evident in the stress-strain paths from the simple shear tests, where the 5% and 10% kaolin mixture specimens experience dilative behaviour following contraction but to a much higher degree in the 5% kaolin mixture. Potentially undrained shear resistances can be higher than drained shear resistances.

5.4.7 Field scale standard CPT

This centrifuge study used the miniature cone penetrometer with a diameter ($d$) of 6 mm with the maximum penetration rate ($v$) of 3 mm/s. Therefore, at maximum velocity, the ratio of $vd$ in the centrifuge to the standard field $vd$ value (of 20 mm/s times 36 mm) is about 0.025. Assuming that coefficients of consolidation are the same in the centrifuge model and prototype, corresponding $V_v$ in the field are about 40 times higher than those in the centrifuge. Therefore the onset of partially drained conditions in the field would arise when the coefficient of consolidation is about 60 times greater than that of the 25% kaolin mixture i.e. greater than about 1800 m$^2$/year (60 times 30 m$^2$/year).

The centrifuge data also indicate that partially drained conditions would arise in the 5% and 10% kaolin mixtures in a field scale standard CPT. The $V_v$ values of ~0.015 and ~0.021 at $v=3$ mm/s in the centrifuge correspond to $V_v$ values of about ~0.60 and ~0.84 at field scale. The data suggests that partially drained conditions would be encountered in soils containing $c_v$ values of between about $2.2 \times 10^4$ and $5.4 \times 10^4$ m$^2$/year.
5. Variable rate miniature CPT in clayey sands using drum centrifuge

5.5 SUMMARY

This chapter presented the CPT results at variable penetration rates in three clayey sand mixtures performed in the drum centrifuge. These centrifuge tests, when compared to previous physical modelling studies, show the following:

- Using coefficients of consolidation \(c_v\) measured at normally consolidated conditions from Rowe cell consolidation tests, undrained conditions were encountered at the normalised velocities \(V_v\) greater than 10; drained conditions appeared at \(V_v\) values less than 0.06.
- The undrained resistance \(Q_{UD}\) measured in the 25% kaolin mixture was lower than that of pure kaolin clay, which is consistent with observations in the undrained simple shear element tests.
- The drained resistance \(Q_D\) measured in the 25% kaolin mixture was higher than that of kaolin clay, which can be associated with higher friction angle and stiffness.
- The \(Q_D\) in the 5% and 10% kaolin mixtures were even higher than that in the 25% kaolin mixture but lower than those in clean dry silica sands reported in previous studies.
- Although the clayey sands have a higher density than equivalent mixtures with no fines content, their penetration resistance is lower than clean sands.
- Effective peak friction angles \(\phi'_p\) estimated from \(Q_D\) were comparable to the apparent \(\phi'_p\) estimates from simple shear tests (with no fines content applied).
- A partially drained or undrained condition during penetration was not achieved in sands with a small portion of clays (i.e. the 5% and 10% kaolin mixtures) in the centrifuge tests as the maximum \(V_v\) value achieved was 0.021.
- A standard field CPT would lead to partially drained conditions in the 5% and 10% kaolin mixtures and an undrained condition in the 25% kaolin mixture.
CHAPTER 6 VARIABLE RATE MINIATURE CPTU AT 1G IN CLAYEY SOILS

6.1 INTRODUCTION

This Chapter presents an experimental study of the rate dependence of piezocone penetration in clayey soils in pressurised chambers at 1g. Clayey soil samples were consolidated in simple containers in the laboratory and a series of miniature piezocone tests were carried out at variable penetration rates. The soil samples tested here include kaolin clay and a mixture of 25% kaolin and 75% sand. The test set-up is firstly described followed by a description of sample preparation techniques before analysing the penetration test results. Repeatability between penetrations within one container and also between the sample containers were confirmed at shallow depths. Effects of the variable rate CPTUs were investigated at depths corresponding to steady state penetration and the results are then compared with the centrifuge tests presented in Chapter 5 and the similar studies from the literature.

6.2 TEST APPRATUS

6.2.1 Miniature piezocone

Figure 6.1 shows the miniature piezocone penetrometer used in this experiment. The device can measure cone tip resistance \( q_c \), sleeve friction \( f_s \) and pore pressure at the cone shoulder \( u_2 \). The piezocone has a diameter \( d \) of 10 mm with a cone apex angle of 60° (i.e. a cross section area of 78.54 mm\(^2\)) and a friction sleeve area of 1162.39 mm\(^2\). A high-density polypropylene filter element was attached to the device at the \( u_2 \) position to measure pore pressures. Calibration of the device was conducted prior to testing and the calibration results are summarised in Table 6.1. An unequal area ratio \( \alpha \) was assessed in the calibration chamber by applying external pressures in stages and found to be 0.80.
6. Variable rate miniature CPTU at 1g in clayey soils

![Image of miniature piezocone penetrometer and testing rig](image)

Figure 6.1 Photographs of (a) miniature piezocone penetrometer and (b) testing rig

<table>
<thead>
<tr>
<th>Item</th>
<th>Calibration factor</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load cell (tip)</td>
<td>193.526</td>
<td>N/Volt</td>
</tr>
<tr>
<td>Load cell (friction sleeve)</td>
<td>158.942</td>
<td>N/Volt</td>
</tr>
<tr>
<td>Pore pressure transducer</td>
<td>240.461</td>
<td>kPa/Volt</td>
</tr>
<tr>
<td>Unequal area ratio, (\alpha)</td>
<td>0.80</td>
<td>-</td>
</tr>
<tr>
<td>Displacement transducer</td>
<td>408.449</td>
<td>mm/Volt</td>
</tr>
</tbody>
</table>

Table 6.1 Calibration details

6.2.2 Testing rig

In this 1g laboratory experiment, the UWA field penetration testing rig was used (see Figure 6.1b). The rig was originally manufactured for UWA research projects such as that reported in Schneider et al. (2004); see Chapter 3. The testing frame without the vehicle had enough capacity to provide sufficient penetration resistance for all of the 1g experiments. The miniature piezocone was tightly clamped to the ‘driving part’ to allow vertical movements (as shown in Figure 6.1b). Penetration was performed through a ‘cut-out’ in the top plate as will be described in Section 6.3.3. The penetration velocity was varied between 0.0002 and 20 mm/s.
6. Variable rate miniature CPTU at 1g in clayey soils

6.2.3 PVC container and pressure system

A sample container with a pressure system for sample consolidation is shown in Figure 6.2a. The container consisted of a PVC pipe and PVC plates to cover the top and bottom of the container. The PVC container had an inside diameter of 150 mm and an effective height of 390 mm. A 200 mm long detachable PVC pipe for an extra upper section was used at the beginning of consolidation of slurry samples, and then removed after the sample had settled. Two-way drainage condition was expected via narrow holes in the top and bottom PVC plates.

The inside of the PVC containers were greased with petroleum jelly before pouring the slurry samples in. However, some of the containers were not greased and so test results will be discussed comparably on the effect of greasing.

A pressure system was built using small metal bar pieces and threaded bars. Dead weights were placed at the bottom of the frame, held by one contact point in the middle of the top plate (see Figure 6.2a). This helped surcharge pressures to be applied evenly to the samples. The sample container was placed on the testing bench for penetration tests (see Figure 6.2b) and the surcharge pressure was maintained on top of the sample to preserve normally consolidated conditions before penetration testing.

Figure 6.2 Photographs of PVC container and pressure system set-up for: (a) consolidation phase and (b) testing phase
6.3 SAMPLE PREPARATION

6.3.1 Soil samples

Pure kaolin clay samples and clayey sand mixtures, comprising 75% fine silica sand and 25% of kaolin clay (by dry weight), were prepared for this experiment. The soil properties including the particle size distributions and Atterberg limits were presented in Section 3.6.2. In addition, a series of the Rowe cell test results and the simple shear test results on kaolin clay (100% kaolin) and clayey sand mixture (25% kaolin mixture) were reported in Sections 3.6.3 and 3.6.4.

6.3.2 Consolidation of sample

Each sample was mixed at a water content of twice its liquid limit. The mixed slurry sample was then slowly poured into the PVC container to minimise the entrapment of air bubbles. Geofabrics were used between the PVC top/bottom plates and the soil slurry sample to allow even drainage and to prevent potential sample extrusion through the drainage holes. To prevent its extrusion in pure kaolin samples, a fine sand layer of about 30 mm was placed on top of the kaolin clay.

Using the pressure system as explained above, dead weights were gradually placed to apply a surcharge to the top of samples while sample settlements were monitored on a periodic basis. A final total weight of about 96 kg (equivalent to a surcharge pressure of 54 kPa) was applied to all samples. Fully saturated conditions were ensured by keeping the water level above the top plate during consolidation.

Figure 6.3 and Figure 6.4 summarise loading schedules and corresponding sample settlement results in five 100% kaolin samples and eight 25% kaolin mixture samples. Loading was applied in increments. The 100% kaolin samples were left to consolidate under a surcharge pressure of 54 kPa for about three months, while the 25% kaolin mixture samples were left for about two months. At the end of consolidation, the 100% kaolin samples had settled by about 45% (on average) of their initial sample height (see Figure 6.3b). Settlements for the 25% kaolin mixture samples were lower at about 30-35% the initial sample heights. This difference reflects (i) the initial water contents used to mix samples into slurry conditions and (ii) the different compression indices \( C_c \) of \(-0.555\) and \(-0.153\) in 100% kaolin and the 25% kaolin mixture respectively, as obtained
Figure 6.3 Consolidation results for 100% kaolin samples: (a) loading schedule and (b) sample settlement in Rowe cell consolidation tests (see Section 3.6.3).

The time required for 90% consolidation (assuming the total surcharge of 55 kPa is applied at once) can be estimated based on Taylor’s root time method with the vertical coefficient of consolidation ($c_v$) obtained in the Rowe cell tests (see Section 3.6.3). Assuming a case with a sample height of 600 mm with one-way drainage, kaolin clay and the 25% kaolin mixture require about 45 days and 6 days respectively for 90% consolidation under the vertical stress of 55 kPa. Taylor’s method does not provide a direct indication due to the loading pattern adopted. However, full consolidation could be expected after 2 to 3 months of consolidation time.
After consolidation was completed, the sample container was moved onto the testing bench (as shown in Figure 6.2b) and left for at least 24 hours to ensure that the surcharge pressure was fully applied with the set-up prior to any penetration tests.
6.3.3 Testing details

Four penetration tests were conducted in each sample through a 14 mm diameter hole cut-out in the top plate. The plate with two cut-outs was replaced with the consolidation top plate (without cut-outs) by unloading all the dead weights, positioned and then the surcharge was reapplied to maintain normally consolidated condition. The CPTU testing locations are illustrated in Figure 6.5. Four penetration tests were conducted with time intervals of about 24 hours between the extraction and the next penetration test, in order to minimise any potential complicating effects associated with excess pore pressure generation during the preceding penetrometer installation. The distance between the outside face of the penetrometer to the inside wall of the container was about 35 mm, which is about seven times the cone radius ($r$).

Most penetration tests involved two penetrometer velocities although, in some cases, only one velocity was employed. The two-rate penetration tests involved changing the penetration rate at a specified penetration depth. At shallow depths, a relatively fast penetration rate was used to check if similar CPTU measurements can be obtained between penetrations, thereby proving that effects of interaction between penetrations were minor. At greater depths, variable penetration rates ranging from 0.0002 to 20 mm/s were used.

![Figure 6.5 Testing layout for four penetrations in one container (plain view)](image-url)
6.4 PENETRATION TEST RESULTS AND ANALYSIS

6.4.1 Penetration test profiles

Figure 6.6a shows a typical penetration test result in the 100% kaolin sample (100E sample). The penetration was performed to a depth of about 200 mm, at which stage the distance to the bottom boundary was about 100 mm (~10d, 10 times the cone diameter). The velocity (v) was changed at a specified depth for each sample; for example it was at ~120 mm in sample 100E; see Figure 6.6a. For further discussion on rate effects, the CPTU measurements were extracted from a zone of interest where steady values of these measurements were obtained after the velocity was changed. In sample 100E, the selected zone of interest was between depths of 150 and 200 mm (see Figure 6.6a).

A relatively constant corrected cone tip resistance (qt) of about 200 kPa was recorded at v=1 mm/s in a 100% kaolin sample (sample 100E) at depths greater than about 30 mm below the sand surface layer. The qt values measured at v=0.4 mm/s were similar to those at v=1 mm/s, whereas qt at v=0.04 and 0.0004 mm/s were higher (~290 and ~370 kPa respectively) than the qt at v=1 mm/s. The measured pore pressure (u2) was zero in the sand layer from the surface down to about 30 mm. u2 increased between about 40 and 70 mm depth, followed by a fairly constant value of ~100 kPa at v=1 mm/s. As expected, lower u2 values were measured at slower velocities. At v=0.0004 mm/s, fully drained condition was observed as zero u2 were measured in the zone of interest in the 100% kaolin sample.

Figure 6.6b shows a typical test result in the 25% kaolin mixture sample (sample 25H). In this case, the selected zone of interest was between 160 and 210 mm depth, where there was a distance of about 120 mm (~12d) to the bottom boundary. At v=20 mm/s, a steady qt value of about 150 kPa and u2 value of 80 kPa were obtained at depths below 50 mm from the soil surface. The qt and u2 values measured at v=1 mm/s were similar to those measured at v=20 mm/s. A rate dependence on qt and u2 was evident at v=0.02 and 0.001 mm/s, where qt increased but u2 decreased as v was reduced.
6. Variable rate miniature CPTU at 1g in clayey soils

Figure 6.6 Typical CPTU penetration profiles in: (a) 100% kaolin sample and (b) 25% kaolin sample
6. Variable rate miniature CPTU at 1g in clayey soils

6.4.2 Effects of penetration rate

The penetration test results in the five 100% kaolin samples are summarised against penetration velocities in Figure 6.7. All the data points were extracted from the zone of interest determined individually in each sample as explained above.

The effect of grease on the inside wall of the chamber is evident especially in $q_t$ variations against $v$, where the $q_t$ values are generally lower at any velocity in the non-greased containers (samples 100A and 100B) than those in the greased containers (samples 100C, 100D and 100E). The effect of grease is not so significant in $u_2$ as similar $u_2$ values are obtained except for slightly smaller $u_2$ values at $v<1$ mm/s in the non-greased containers. The smaller $q_t$ values obtained in the non-greased containers are likely to be associated with reduced stress levels imposed from the surcharge because of higher frictional forces between the soil and non-greased inside wall. Therefore, further analysis on rate dependency is focuses only on the samples in the greased containers.

The penetrations at velocities greater than 1 mm/s tend to be undrained. Almost constant $u_2$ values of about 110 kPa and the $q_t$ values of 200±15 kPa are measured for $v \geq 1$ mm/s. On the other hand, fully drained conditions are evident at $v<0.001$ mm/s where almost zero excess pore pressure ($\Delta u = u_2 - u_0$, where $u_0$ is the hydrostatic pore pressure ≈1 to 2 kPa) is obtained even though the $q_t$ values at the same velocity range seems to vary between 310 and 370 kPa (average of 340 kPa). These drained $q_t$ values are 1.6 to 1.9 times the $q_t$ values measured in the undrained condition. Partially drained conditions are seen to operate over three orders of magnitude of velocity (between 0.001 and 1 mm/s in this experiment). The greatest increase in $q_t$ occurred when $v$ reduced from 0.1 to 0.01 mm/s whereas the greatest reductions in $u_2$ occurred at $v$ between 0.01 and 1 mm/s.
Following the same method, the penetration test results in the eight 25% kaolin mixture samples are summarised in Figure 6.8. As observed in the 100% kaolin samples, effects of greasing the inside of the containers had a significant effect on $q_t$ variations at any velocities. In the greased containers (samples 25D to 25H), the penetrations at $v>0.4$ mm/s seems to be undrained with $q_t$ and $u_2$ values of 165±15 kPa and about 80 kPa respectively. Drained conditions appear at $v<0.001$ mm/s where $\Delta u$ is essentially zero and $q_t$ is about 900 kPa; this drained $q_t$ value is about 5.5 times higher than the undrained $q_t$ value. Partially drained conditions operate between 0.002 and 0.4 mm/s, which is fairly similar to the partially drained range shown by the 100% kaolin samples.
The effect of the penetration rate in the two soils (100% kaolin and 25% kaolin mixture) are compared in Figure 6.9 using standard piezocone parameters including the normalised cone resistance \( Q = q_{\text{net}}/\sigma'_{v0} \) and two pore pressure ratios \( \Delta u/\sigma'_{v0} \) and \( B_q = \Delta u/q_{\text{net}} \). The \( Q \) values at \( v > 0.4 \) mm/s are relatively constant in each mixture, but slightly lower \( Q \) values of \( \sim 2.0 \) can be found in the 25% kaolin mixtures, compared to \( Q \) of \( \sim 2.5 \) in the 100% kaolin. The \( Q \) values at \( v < 0.4 \) mm/s are rather different between the two soil types. For example, \( Q \) values at \( v < 0.001 \) mm/s of 4.6 to 5.8 are seen in the 100% kaolin but much higher values of about 15 are developed in the 25% kaolin.

The transitions between undrained and partially drained conditions and between partially drained and fully drained conditions are relatively similar for the two soil types.
in this experiment. According to $\Delta u/\sigma'_{v0}$ variations in Figure 6.9b, partially drained conditions tend to be encountered between $v=0.001$ and 1 mm/s. As the $q_t$ values at $v=0.4$ mm/s are similar to those at $v>1$ mm/s, the transition from undrained to partially drained based on the $q_t$ variations does not coincide exactly with the $u_2$ variations; this has been referred to as the offsetting effect by Kim (2005). This offset presumably occurs due to two factors: (i) a dissipation of pore pressure is measured immediately at the cone shoulder but a gain in strength at the cone tip reflects overall stress field change in both plastic and elastic zones surrounding the cone tip, and (ii) a gain in cone resistance due to strain rate dominates when partial consolidation effect is relatively small. The generation of $\Delta u/\sigma'_{v0}$ in undrained condition is about 1.4 in the 25% kaolin mixtures, which is about 30% less than that measured in 100% kaolin ($\Delta u/\sigma'_{v0}\sim2.0$). The $B_q$ values are relatively similar for the two soil types in undrained condition, but zero $B_q$ values tend to appear at higher $v$ in the 25% kaolin mixtures due to much higher $q_t$ values obtained.
Figure 6.9 Effects of penetration rate on piezocone parameters: (a) $Q$, (b) $\Delta u/\sigma'_v$ and (c) $B_q$.
6.4.3 Testing after completion of penetration tests

After the penetration tests, small portions of bulk samples were collected at various depths to measure water contents ($w_c$). The $w_c$ values obtained are fairly comparable between the containers for each soil type (see Figure 6.10). However, the $w_c$ values in the non-greased containers are slightly higher than those in the greased containers, which may imply lower stress levels being imposed in the non-greased containers.

On average in the greased containers, $w_c$ values of 50% to 54% (100% kaolin) and 16% to 18% (25% kaolin mixture) were obtained at depths between 50 and 250 mm. These values are slightly less than the respective liquid limits of 55% and 20% in each soil type (see Section 3.6.2). In particular, $w_c$ values of ~52% (100% kaolin) and ~17% (25% kaolin mixture) were measured within the primary zone of interest (see Section 6.4.1).

![Figure 6.10 Results on moisture content test in: (a) 100% kaolin and (b) 25% kaolin mixture](image)

Figure 6.11 show the void ratio variations with depth. The void ratios ($e$) of ~1.32 to 1.37 (100% kaolin) and ~0.43 to 0.46 (25% kaolin mixture) can be calculated at the zones of interest in the greased containers, using the relationship of $e = w_c \times G_s$ (assuming fully saturated condition). The differences between the greased containers in terms of the water content and the void ratio are relatively small, which suggests that the 1g experiment had good repeatability and allowed comparisons between the containers.
6. Variable rate miniature CPTU at 1g in clayey soils

6.4.4 Validation of 1g test

The penetration test results obtained in this 1g simple set-up experiment are compared with centrifuge and calibration chamber studies reported in the literature (Randolph & Hope 2004, Schneider et al. 2007, Jaeger et al. 2010, Kim et al. 2010). The data from previous research were digitally extracted from available figures. Figure 6.12 and Figure 6.13 plots $Q$ and $\Delta u/\sigma_{v0}$ variations in kaolin clay and the 25% kaolin mixture against the normalised velocity ($V$), initially proposed by Finnie & Randolph (1994), which is defined as:

$$V_v = \frac{v d}{c_v}$$  \hspace{1cm} (6-1)

where $v$ is the penetrometer velocity, $d$ is the cone diameter and $c_v$ is the vertical coefficient of consolidation. In this analysis, the vertical coefficient of consolidation ($c_{vNC}$) measured in one dimensional consolidation tests in normally consolidated soils is used to allow direct comparisons with the other previous studies. The $c_{vNC}$ values of about 0.088 mm$^2$/s (2.8 m$^2$/year) and 0.70 mm$^2$/s (22 m$^2$/year) were obtained at the stress level of 50 kPa in kaolin clay and the 25% kaolin mixture (see Section 3.6.3), and used for velocity normalisation.
The 1g variable rate CPTU results in kaolin clay are compared in Figure 6.12 with two centrifuge studies conducted at 100g and 160g by Randolph & Hope (2004) and Schneider et al. (2007). As can be seen in Figure 6.12, the $Q$ variations obtained at 1g are generally similar to those reported in the centrifuge studies. Almost doubling of resistance occurs as the velocity slows from undrained to drained conditions, although the 1g test shows slightly lower drained resistance. The $\Delta u/\sigma'_v$ trends for all three studies are very similar. Figure 6.12a also confirms the suitability of the $V_v$ normalisation (noting that the centrifuge studies produced a maximum about twice the effective stress field of that used in the 1g tests). With reference to the $\Delta u/\sigma'_v$ kaolin data on Figure 6.12b, a transition from undrained to partially drained occurs at $V_v \sim 100$ and a transition from partially drained to fully drained occurs at $V_v \sim 0.1$.

Figure 6.12 Comparisons of penetration test results in kaolin: (a) $Q$ and (b) $\Delta u/\sigma'_v$
Figure 6.13 compares the 1g test results in the 25% kaolin and 75% sand mixture with previous studies by Jaeger et al. (2010) and Kim et al. (2010). The variable rate CPT results obtained in the drum centrifuge are also plotted for comparison purpose (see Chapter 5). The 1g test results generally follow a similar trend to that reported in the centrifuge experiments reported in Chapter 5 and by Jaeger et al. (2010) and in the calibration chamber tests reported by Kim et al. (2010). As the normalised velocity reduces by 2 to 3 order of magnitude, all studies show significant increases in $Q$ values with corresponding large reductions in $\Delta u/\sigma'_{v0}$. However, inferred transitions between undrained and partially drained conditions varies between $V_v$ values between 10 and 30 from $\Delta u/\sigma'_{v0}$ profiles while transitions from partially drained to fully drained conditions tend to be at $V_v$ between 0.03 and 0.06. The small variations are believed to be partly due to the different fabrics and sand densities associated with the sample preparation techniques employed in the respective studies. Following the same technique in the centrifuge (Chapter 5) and at 1g (this Chapter), the similar drainage transitions were available in the two different testing methods, although there were slight differences in the measured drained and undrained resistances as follows.

The drained and undrained resistances ($Q_D$ and $Q_{UD}$) obtained in all studies are listed in Table 6.2. Although there are some variations in $Q_D$ and $Q_{UD}$ measurements between the studies, it is clear that, in all experiments, the 25% kaolin mixture has lower $Q_{UD}$ and higher $Q_D$ values than the 100% kaolin clay.

Table 6.2 Comparisons of the drained and undrained resistances in kaolin and 25% kaolin + 75% sand mixtures

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Study</th>
<th>$Q_D$</th>
<th>$Q_{UD}$</th>
<th>$Q_D/Q_{UD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>kaolin</td>
<td>This study (1g)</td>
<td>5.2</td>
<td>2.5</td>
<td>2.1</td>
</tr>
<tr>
<td>kaolin</td>
<td>Randolph and Hope (2004) (100g)</td>
<td>6.3*</td>
<td>2.7</td>
<td>2.3</td>
</tr>
<tr>
<td>kaolin</td>
<td>Schneider et al. (2007) (160g)</td>
<td>7.1</td>
<td>3.4</td>
<td>2.1</td>
</tr>
<tr>
<td>25% kaolin + 75% sand</td>
<td>This study (1g)</td>
<td>15</td>
<td>2.0</td>
<td>7.5</td>
</tr>
<tr>
<td>25% kaolin + 75% sand</td>
<td>Chapter 5 (100g)</td>
<td>12</td>
<td>1.4</td>
<td>8.6</td>
</tr>
<tr>
<td>25% kaolin + 75% sand</td>
<td>Kim et al. (2008) (1g)**</td>
<td>12.6</td>
<td>2.0</td>
<td>6.3</td>
</tr>
<tr>
<td>25% kaolin + 75% sand</td>
<td>Jaeger et al. (2010) (100g)</td>
<td>15</td>
<td>-0.9</td>
<td>-17</td>
</tr>
</tbody>
</table>

* Maximum $Q$ value  
** The work followed by Kim et al. (2010)
The ratio of drained to undrained resistance \( \frac{Q_D}{Q_{UD}} \) is often evaluated in penetration rate effects studies in clayey soils. Numerical simulation of variable rate CPTU using large deformation finite element analysis presented by Yi et al. (2012) suggests that the \( \frac{Q_D}{Q_{UD}} \) ratio is essentially affected by a stiffness ratio \( \frac{G}{p'} \) in fine-grained soils. These authors proposed the following relationship:

\[
\frac{Q_D}{Q_{UD}} = 0.022 \frac{G}{p'} + 1.331
\]  
(6-2)

where \( G \) is the equivalent linear elastic shear modulus and \( p' \) is the mean effective stress.
Using an effective Poisson’s ratio ($\nu'$), a void ratio ($e$), and a recompression index ($\kappa$), a $G/p'$ can be expressed as:

$$\frac{G}{p'} = \frac{3(1 - 2\nu')(1 + e)}{2(1 + \nu')\kappa}$$

(6-3)

Following Equation (6-3), $G/p'$ of ~30 can be obtained for kaolin clay with $e=1.35$, $\kappa=0.039$ (noting $C_t=0.089$ from Rowe cell test in Section 3.6.3) and $\nu'=0.3$. According to Equation (6-2), a $G/p'=30$ leads to a $Q_D/Q_{UD}$ of 2.0, which agrees reasonably well with the experimental $Q_D/Q_{UD}$ values in kaolin clay (including the 1g tests).

The 25% kaolin mixtures had $e=0.45$ and $\kappa=0.006$ (i.e. $C_t=0.016$, Section 3.6.3). Using the same procedure, a $G/p'$ of 112 is calculated for this soil which leads to a predicted $Q_D/Q_{UD}$ ratio of ~3.8 according to Equation (6-2). This predicted ratio is much lower than the ratio of 7.5 seen in the 1g tests; similarly high $Q_D/Q_{UD}$ ratios were measured in the other tests (see Table 6.2). This mis-match points to deficiencies in the soil model employed in the numerical simulations of Yi et al. (2012), who assumed an elastic-perfectly plastic soil with a non-dilatant Drucker-Prager yield criterion. As observed in the undrained simple shear tests (see Section 3.6.4), the 25% kaolin and 75% sand mixture exhibits a far more contractant response than that of kaolin clay. A soil model that accounts for such softening behaviour is required to obtain realistic predictions for the 25% kaolin mixture.

### 6.4.5 Boundary effects

Although the 1g test in this Chapter captured similar penetration rate effects to those reported in the other studies, boundary effects on the CPTU measurements made in relatively small containers cannot be dismissed. Simple numerical analyses employing the spherical cavity expansion method (e.g. Yu 2000) were used to assist in understanding of the boundary effects. The numerical analyses were conducted using the finite element program PLAXIS (Brinkgreve et al. 2012). A basic analysis procedure will be explained in Chapter 7, where a detailed parametric study on rate effects is presented. In this section, focus is placed on the distance to the side boundary to evaluate potential boundary effects on the 1g penetration test results.

In the 1g test, the penetrometer had a cone radius ($r$) of 5 mm and a minimum distance from the centre of the cone to the inner wall ($D$) of about 40 mm, as indicated by the
‘A-B’ line in Figure 6.5. To simulate this critical situation (i.e. $D/r = 8$) in axisymmetric numerical analysis, a soil domain radius ($R$) of 1.6 m and a final cavity radius ($a_f$) of 0.2 m expanded from the initial cavity radius ($a_0$) of 0.1 m are used (i.e. $R/a_f = 8$). The analysis case with $R/a_f = 8$ is compared with the analysis case with $R/a_f = 60$ for which no boundary effects on the cavity expansion pressures were observed. The analyses with $R/a_f = 60$ provided the limit pressure ($p_{\text{limit}}$) fairly comparable to the analytical close-form solutions, which will be described in Section 7.3.

As an example, the Hardening Soil model in PLAXIS (to be described in Section 7.2.3) is used here with a reference Young’s modulus at 50% of the maximum deviator stress ($E_{50}^{\text{ref}}$) of 5 MPa and a friction angle ($\phi$) of 25°, noting that boundary effects would be varying with friction angle and stiffness of soils. The other soil parameters are kept the same as Group I analyses (see Section 7.4.1).

Figure 6.14 compares excess pore pressure ($\Delta u$) distributions in the radial direction at the cavity depth of 12 m after undrained cavity expansion. It is apparent that the $\Delta u$ values predicted for $R/a_f = 8$ are generally $\sim 7\%$ higher than those with $R/a_f = 60$ due to close proximity to the side boundary (where radial water flow is restricted in the 1g experiments). In addition, it is shown in Figure 6.15 that the excess pore pressures ($\Delta u$) and the cavity limit pressures ($\sigma_1$) obtained close to the cavity wall are slightly affected by the proximity of the boundaries (for the two cases analysed) under a range of different rates of cavity expansion. For this analysis case and the assumed soil properties, at $R/a_f = 8$, $\Delta u$ values are 10% higher and cavity stresses ($\sigma_1$) are 5% higher compared to when $R/a_f = 60$.

Preliminary trials performed for a final year student project (Chong 2012) at UWA indicated that consistent results could not be obtained in the 1g set-up for sandy samples with lower clay fractions. In those tests, interaction between penetrations was evident from $q_c$ profiles because densification and change in lateral resistances might occur after one penetration. This could happen in the soils with high stiffness as a plastic zone generated by cone penetration is likely to depend on rigidity index ($I_r$) (e.g. Bond 1989). However, as indicated by the similarities between the results of 1g set-up and previous studies in the centrifuge and pressure chamber (see Figure 6.12 and Figure 6.13), it can be concluded that this 1g set-up can provide a cost-effective means of assessing rate effects in clayey soils.
6. Variable rate miniature CPTU at 1g in clayey soils

Figure 6.14 Boundary effect on excess pore pressure field around the cavity

Figure 6.15 Boundary effects at various cavity expansion velocities on (a) excess pore pressure generations at close to the cavity wall and (b) cavity limit pressure at the cavity wall
CHAPTER 7  ANALYSIS OF CPT RESISTANCE AT VARIABLE PENETRATION RATE

7.1  INTRODUCTION

This Chapter presents numerical analyses simulating CPT tip deep penetration under various drainage conditions. The cone installation is simply modelled as expansion of the cavity based on the spherical cavity expansion theory; this technique has been shown to provide reasonable estimates of cone end resistance (e.g. Yu & Mitchell 1998, Yu 2000) as reviewed in Chapter 2. In the numerical study presented here, spherical cavity expansion is modelled using the finite element analysis code PLAXIS (Brinkgreve et al. 2012). Use of a coupled consolidation calculation in PLAXIS allows prediction of the generation and dissipation characteristics of excess pore pressures due to the cavity expansion, hence assisting understanding of factors affecting cone end resistance under various drainage conditions. This PLAXIS study employs an advanced (and realistic) non-linear elasto-plastic Hardening Soil model, and models the spherical cavity expansion while the other similar studies modelled the cylindrical cavity expansion (Silva 2005, LeBlanc & Randolph 2008, Jaeger 2012).

In this Chapter, the method used in PLAXIS and the correlation method between the cavity expansion pressure and a cone resistance, is explained first, which is then followed by verification of the method; verification is provided through comparisons of the PLAXIS results with the closed-form solutions previously proposed for undrained and drained spherical cavity expansions. An extensive parametric study is then performed to investigate the effect of various soil parameters on the cone end resistance at various cavity expansion velocities (i.e. variable cone penetration rates). Focus is given to normally consolidated non-dilative/contractive soils but several analyses on overconsolidated and dilative soils are also presented. The PLAXIS analyses are compared with the experimental test results to assess the potential (and deficiencies) of
the numerical approach employed.

7.2 CAVITY EXPANSION IN PLAXIS

7.2.1 Use of PLAXIS

The commercial finite element program called PLAXIS Classic was employed in this study. PLAXIS has often been used for a wide range of geotechnical applications, including studies to examine a CPT tip resistance relating to limit pressure \( p_{\text{limit}} \) based on the SCE theory (e.g. Xu & Lehane 2008, Tolooiyan & Gavin 2011). The PLAXIS program consists of four main stages, ‘Input’, ‘Calculations’, ‘Output’ and ‘Curves’ programmes. The method used in this study is similar to that presented in Xu (2007), who examined a pile tip resistance in layered soils. The most significant aspect in this study is the analysis of CPT tip resistance in various drainage conditions using a coupled consolidation analysis (instead of fully drained or fully undrained analysis in PLAXIS) in combination with a realistic non-linear elasto-plastic soil model.

7.2.2 Analysis approach

The analysis procedure in PLAXIS Classic used in this study is described as follows:

(i) In the ‘Input’ program, an axi-symmetric soil domain with a size of 12m×24m in the x-y plane is created (see Figure 7.1a). The spherical cavity with a radius \( a_0 \) of 0.1 m is located at the 12 m depth. Mesh boundaries are totally fixed at the base, free at the surface and free only in vertical direction on both side boundaries. Material properties (as explained later) are assigned for the cavity and surrounding soils. A mesh comprising triangular elements with 15 nodes and 12 stress points is generated in this stage. A very fine mesh is required in the zone close to the cavity where large strains differences are expected to occur (see Figure 7.1b).

(ii) In the ‘Calculation’ program, the initial condition is firstly generated based on a specified earth pressure coefficient \( K_0 \) with a water table at the soil surface. Seven nodes and seven stress points are selected at the cavity wall and very close to the cavity wall respectively for output purposes (see Figure 7.1b).
(iii) In the following phases, a positive (i.e. expansive) volumetric strain is applied to the cavity cluster, which allows the cavity to expand while consolidation analysis is used with the *updated mesh* option. The cavity is required to expand to at least twice the initial cavity radius for modelling. Each calculation phase (always starting from the initial stress condition), is defined with various consolidation times, which represent various speeds of cavity expansion.

(iv) After all calculations are completed, strain/stress contours and plastic points can be checked in “Output” program.

(v) The ‘Curve’ program is used to plot the maximum principal stresses ($\sigma_1$) and the excess pore pressures ($\Delta u$) against the radial displacements ($\mid U \mid$) from the seven stress points and the seven nodes respectively.

(vi) All extracted data are further processed in excel. The averaged pressures at $a/a_0=2$ (when the cavity radius ($a$) is expanded to double the initial cavity radius ($a_0$) of 0.1 m) are used as approximate limit pressures in this study; see Section 7.3. The cone resistance is finally calculated from normal and shear stresses acting on the cone face using the relationship derived in Section 7.2.5.

Figure 7.1 Axisymmetric soil domain and generated mesh: (a) global, (b) local around the cavity with selected nodes and stress points for output purpose
Effect of mesh coarseness

Three different mesh sizes: a coarse mesh (~900 elements), a medium coarse mesh (~1300 elements) and a fine mesh (~1800 elements) were used to study effect of mesh coarseness following the analysis procedure detailed above. As shown in Figure 7.2, the difference in the cavity expansion pressure is small between the medium mesh and the fine mesh, which indicates that the results are not sensitive to the mesh coarseness provided sufficiently finer mesh are used around the cavity. The required time to perform the cavity expansion analysis was short in general, so the fine mesh was employed in this study to obtain accurate results.

![Figure 7.2 Effect of mesh coarseness for undrained and drained cavity expansions: (a) normalised cavity expanding pressure and (b) normalised excess pore pressures generated closed to the cavity wall](image)

Effect of boundary

Boundary effects due to the side boundary are investigated using three different soil domains with a soil width ($B$) of 20m, 24m and 48m in the x-axis while keeping the initial cavity radius ($a_0$) of 0.1m and the finial cavity radius ($a_f$) of 0.2m. It can be seen in Figure 7.3 that the almost identical cavity expanding pressure and the excess pore pressure are obtained for the three different cases ($B/a_f$ of 50, 60 and 120). It is therefore that no boundary effects were indicated for the selected mesh radius to the initial cavity radius ratio of $B/a_0$ of 120, which is the same ratio used in a similar study by Xu (2007).
The author also confirmed that this mesh size of 12m×24m was sufficiently large to allow use of an initial cavity radius of 0.3m without any significant boundary effects.

![Figure 7.3 Effect of lateral boundary for undrained and drained cavity expansions: (a) normalised cavity expanding pressure and (b) normalised excess pore pressures generated closed to the cavity wall](image)

7.2.3 Soil model

The Hardening Soil (HS) model (Schanz et al. 1999) was primarily employed in this study. In some cases, the Modified Cam Clay (MCC) model and the Mohr-Coulomb (MC) model were used to for validation purposes against closed-form solutions (see Section 7.3). A full description of soil models is provided in Schanz et al. (1999) and PLAXIS MATERIAL MODELS MANUAL (Brinkgreve et al. 2012). The HS model is briefly introduced below and its input parameters are listed in Table 7.1.

<table>
<thead>
<tr>
<th>Description</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure parameters according to the Mohr-Coulomb criterion</td>
<td>( c, \phi ) and ( \psi )</td>
</tr>
<tr>
<td>Power for stress-level dependency of stiffness</td>
<td>( m )</td>
</tr>
<tr>
<td>Secant stiffness for primary deviatoric loading in drained triaxial test</td>
<td>( E_{S0}^{ref} )</td>
</tr>
<tr>
<td>Tangent stiffness for primary oedometer loading</td>
<td>( E_{oed}^{ref} )</td>
</tr>
<tr>
<td>Elastic unloading/reloading stiffness</td>
<td>( E_{ur}^{ref} )</td>
</tr>
<tr>
<td>Poisson’s ratio for unloading-reloading</td>
<td>( \nu_{ur} )</td>
</tr>
<tr>
<td>Reference stress for stiffness</td>
<td>( p^{ref} )</td>
</tr>
</tbody>
</table>
In addition to the MC model failure criterion of \( c \) (cohesion), \( \phi \) (friction angle) and \( \psi \) (dilation angle), the HS model has advanced features including stress and strain dependent Young’s modulus. The HS model provides the following hyperbolic relationship for the drained secant Young’s modulus \( (E) \) defined as:

\[
E = \frac{2}{2-R_f} E_{50} \left( 1 - R_f \frac{q}{q_f} \right)
\]  

(7-1)

where \( E_{50} \) is the \( E \) value at 50% of the maximum deviator stress \( (q) \), \( q_f \) is the ultimate deviator stress and \( R_f \) is the failure ratio, set to 0.9. The \( E_{50} \) value and other stiffness parameters \( (E_{oed}, E_{ur}) \) as explained in Table 7.1, are calculated as a function of the stress level, formulated as:

\[
E_{50} = E_{50}^{ref} \left( \frac{c \cos \phi - \sigma_3' \sin \phi}{c \cos \phi + p^{ref} \sin \phi} \right)^m
\]  

(7-2)

\[
E_{oed} = E_{oed}^{ref} \left( \frac{c \cos \phi - \sigma_1' \sin \phi}{c \cos \phi + p^{ref} \sin \phi} \right)^m
\]  

(7-3)

\[
E_{ur} = E_{ur}^{ref} \left( \frac{c \cos \phi - \sigma_3' \sin \phi}{c \cos \phi + p^{ref} \sin \phi} \right)^m
\]  

(7-4)

where \( \sigma_1' \) and \( \sigma_3' \) are the major and minor principal effective stresses and the exponent \( m \) controls the stress level dependence. The HS model provides modulus degradation with large strain and models more realistic stress-strain paths compared to a linear elastic perfectly plastic model such as the MC model.

The ratio of the increment of plastic volumetric strain to that of plastic shear strain (i.e. the flow rule) is given by the well-known stress dilatancy theory of Rowe (1962). Plastic, shear induced, contractive volumetric strains are developed when the mobilised friction angle is less than the critical state angle. Plastic straining due to normal compression is controlled by the one-dimensional, stress-normalised, oedometric stiffness \( (E_{oed}^{ref}) \).

A cap yield surface allows plastic volumetric strain hardening under isotropic compression with the value of \( E_{oed}^{ref} \) controlling the magnitude of plastic strains that originate from the cap. The size of the elastic region bounded by the cap and shear
hardening yield surface (which are controlled by Equation (7-1)) is a function of the specified $K_0$ value for normal consolidation ($K_0^{NC}$).

A constant volume friction angle ($\phi_v$) is determined from a peak dilation angle ($\psi$) and friction angle ($\phi'$) according to Equation (7-5). The mobilised dilation angle ($\psi_m$) is calculated based on a $\phi_v$ and the mobilised friction angle ($\phi'_m$) following Equation (7-5).

$$\sin \phi_v = \frac{\sin \phi' - \sin \psi}{1 - \sin \phi' \sin \psi}$$

(7-5)

PLAXIS provides a Dilation cut-off option to limit dilation behaviour when the current void ratio reaches the maximum void ratio ($e_{max}$). However, it is noted in PLAXIS MATERIAL MODELS MANUAL (Brinkgreve et al. 2012) that the dilation cut-off does not help in limiting the shear strength when using the Undrained (A) drainage type with a positive dilation angle. As Undrained (A) drainage type was required for consolidation analysis in this study, an unrealistically large increase in strength with a positive dilation angle can occur at fast (undrained) expansions. The use of dilation angle is discussed in Section 7.4.15 and its interpretation is qualitative in view of the numerical limitations.

**Effect of the cavity stiffness**

The cavity cluster is modelled as a linear elastic material to allow expansion of the cavity. Three calculations were performed using a linear elastic stiffness ($E$) of the cavity to be 2, 20 and 200 MPa with $\nu=0.2$. As shown in Figure 7.4, the results of the cavity expanding pressure and the excess pore pressure are not sensitive to the stiffness of the cavity, while slightly varying values of the positive (i.e. expansive) volumetric strains are required to reach $a/a_0=2$. For the following analyses in this study, the cavity is modelled as a linear elastic material with $E=20$ MPa.
7. Analysis of CPT resistance at variable penetration rate

7.2.4 Cavity expansion velocity

The cavity with an initial radius \((a_0)\) of 0.1 m is expanded to approximately double the initial radius \((a/a_0≈2)\) by applying a volumetric strain to the cavity cluster. A limit pressure \((p_{\text{limit}})\) can be achieved after a large expansion of the cavity, but this limit is almost fully developed at \(a/a_0=2\) (some typical examples are provided in Section 7.3). In this study, the expanding pressure \((\sigma_t)\) at \(a/a_0=2\) is used as an approximate value of the limit pressure \((p_{\text{limit}})\). The cavity expansion pressure at \(a/a_0=2\) was also used as the approximate limit pressure in normally consolidated and over consolidated clays by Silva (2005). On the other hand, Jaeger (2012) observed that expansions to a maximum of \(a/a_0=2.2\) sufficiently developed the true limit pressure in undrained and partially drained conditions but much greater expansions (to \(a/a_0=8\)) were required to reach limit pressures under drained conditions.

In addition, taking an average of displacements and stresses from the seven nodes and seven stress points was necessary to provide overall response of the cavity to expansion, given the initial anisotropic stress condition around the cavity.

In a coupled consolidation analysis in PLAXIS, excess pore pressure generation and dissipation can be simulated during expansion of the cavity for a given consolidation time. The radial cavity expansion velocity \((v_{r,CE})\) was therefore calculated as:
\begin{align}
\nu_{r,CE} &= \frac{\Delta r_{CE}}{\Delta t_{CE}} \tag{7-6}
\end{align}

where \(\Delta r_{CE}\) is the radial displacement at the cavity wall and \(\Delta t_{CE}\) is the time for this displacement to take place. The cavity expansion velocity was varied (assigning \(\Delta t_{CE}\)) over eight orders of magnitude to investigate a wide range of cavity expansion rate effects. Figure 7.5 shows examples of cavity expansion velocities determined at the cavity wall. By plotting averaged radial displacement changes for different consolidation times \((\Delta t_{CE})\), it was found that the velocity of expansion at the cavity wall was closely comparable to the ideal constant velocity defined by Equation (7-6), noting that the velocity at the cavity wall was not exactly linear (but slightly curved).

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure7_5.png}
\caption{Examples of radial displacements at the cavity wall during consolidation analysis}
\end{figure}

### 7.2.5 Correlation between cavity expansion pressure and CPT tip resistance

As reviewed in Chapter 2, a limit pressure \((p_{\text{limit}})\) can be related to a steady state cone end resistance \((q_c)\). Assuming that the normal stress acting on the cone face is equal to that required to expand a spherical cavity, the relationship between \(q_c\) and \(p_{\text{limit}}\) from the spherical cavity expansion analysis can be obtained from the vertical force equilibrium on the cone face during penetration (see Figure 7.6), and defined as:
Analysis of CPT resistance at variable penetration rate

Figure 7.6 Relation between cavity expansion pressure and tip resistance

\[ q_c \pi r^2 = (p_{\text{limit}} \sin \left(\frac{\alpha_c}{2}\right) + \tau \cos \left(\frac{\alpha_c}{2}\right)) \cdot \frac{\pi r^2}{\sin \left(\frac{\alpha_c}{2}\right)} \]  \hspace{1cm} (7-7)

\[ q_c = p_{\text{limit}} + \frac{\tau}{\tan \left(\frac{\alpha_c}{2}\right)} \]  \hspace{1cm} (7-8)

where \( r \) is the cone radius, \( \alpha_c \) is the cone apex angle, \( p_{\text{limit}} \) and \( \tau \) are the normal limit stress and shear stress acting on the cone face. For a typical cone penetrometer, \( \alpha_c \) is 60° (i.e. \( 1/\tan(\alpha_c/2) = \sqrt{3} \)). It is also assumed here that the values of \( p_{\text{limit}} \) and \( \tau \) are the average values acting on the surface area of the cone.

Although it is common practice to have separate expressions for cavity expansion in clays and sands (i.e. undrained and drained), a general relationship is required here to allow treatment of the partially drained condition. Following the principal of effective stress, the shear stress (\( \tau \)) is expressed as a function of the effective stress:

\[ \tau = c' + \sigma'_{\text{n}} \tan \phi' = c' + (\sigma_{\text{n}} - u) \tan \delta \]  \hspace{1cm} (7-9)

where \( c' \) is the effective cohesion, \( \sigma'_{\text{n}} \) and \( \sigma_{\text{n}} \) are the effective and total normal stresses and \( \phi' \) is the interface friction angle between the cone and surrounding soil. Assuming that \( c' \) is relatively small (or close to zero), the relationship between \( q_c \) and \( p_{\text{limit}} (=\sigma_{\text{n}} \text{ assumed}) \) from Equations (7-7) and (7-9) can then be derived as:
Analysis of CPT resistance at variable penetration rate

\[ q_c = p_{\text{limit}} + \sqrt{3} (p_{\text{limit}} - u) \tan \delta \quad \text{where} \quad \delta = \phi_{cv} \]  

(7-10)

where the interface friction angle was assumed as same as the constant volume friction angle (\(\phi_{cv}\)) of the soils in this study. This relationship has previously been used by Silva et al. (2006) for the cylindrical cavity expansion with variable expansion rates using the finite element code CAMFE, which was originally developed by Carter (1978).

### 7.3 NUMERICAL MODEL VARIFICATION

#### 7.3.1 Undrained spherical cavity expansion

Numerical modelling of undrained and drained spherical cavity expansion computed in PLAXIS is verified by comparing calculations to the closed-form analytical solutions. Cao et al. (2001) presented the closed-form solution for undrained cavity expansion in Modified Cam Clay model. Using the MCC model in PLAXIS (following the analysis procedure as described in Section 7.2.2), PLAXIS analysis results are compared with the Cao et al. solutions. Undrained conditions were established by specifying a very low soil permeability and a rapid expansion (i.e. short consolidation time). In total, eight cases (MCC1 to MCC8) as listed in Table 7.2, were analysed for this comparison. These eight cases consider effects of a stiffness (\(G/p'_{0}\)), the slope of critical state line (\(M\)) as a function of the friction angle (\(\phi\)), an isotropic overconsolidation ratio (\(R\)) and a Poisson’s ratio (\(\nu\)).

As seen on Figure 7.7, the PLAXIS calculations (using the updated mesh option and consolidation analysis) show very good agreement with the closed-form solutions of Cao et al. (2001). All the cavity expansion curves show a large increase in the normalised cavity expanding pressures (\(\sigma_c/p_0\)/\(p'_0\) within the cavity radius after doubling of the initial radius (i.e. \(a/a_0=2\)). The differing effects of various parameters are also well match in the PLAXIS numerical results. Values of (\(\sigma_c/p_0\)/\(p'_0\)) at \(a/a_0=2\) calculated in PLAXIS compare well with limit pressures indicated by closed-form solutions; difference of less than 4 % are indicated in Table 7.2.

Excess pore pressures (\(\Delta u\)) generated during undrained cavity expansion in PLAXIS calculations are also in good agreement with the closed-form solutions (see Table 7.3 and Figure 7.8) and it is concluded here that seven stress points selected for output in
PLAXIS (see Figure 7.1b) can provide a close approximation of the $\Delta u$ pressures at the cavity wall derived from the closed-form solutions. The $\Delta u$ values were used for $q_c$ calculations in accordance with Equation (7-10). It should be noted that the excess pore pressures calculated from the spherical CE theory may not necessarily represent the pore pressure measurements at the cone shoulder (i.e. the so-called $u_2$ position).

Table 7.2 Comparison of normalised spherical cavity pressures at $a/a_0=2$ between PLAXIS results and the closed-form solutions presented by Cao et al. (2001)

<table>
<thead>
<tr>
<th>CASE</th>
<th>$G/p'_0$</th>
<th>$M$</th>
<th>$R$</th>
<th>$\nu'$</th>
<th>($\sigma_a-p_0)/p'_0$ (closed-form)*</th>
<th>($\sigma_a-p_0)/p'_0$ (PLAXIS)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCC1</td>
<td>100</td>
<td>1.2</td>
<td>1.1</td>
<td>0.3</td>
<td>3.29</td>
<td>3.24</td>
<td>-1.5</td>
</tr>
<tr>
<td>MCC2</td>
<td>20</td>
<td>1.2</td>
<td>1.1</td>
<td>0.3</td>
<td>2.46</td>
<td>2.38</td>
<td>-3.3</td>
</tr>
<tr>
<td>MCC3</td>
<td>300</td>
<td>1.2</td>
<td>1.1</td>
<td>0.3</td>
<td>3.85</td>
<td>4.00</td>
<td>3.9</td>
</tr>
<tr>
<td>MCC4</td>
<td>100</td>
<td>1.2</td>
<td>3</td>
<td>0.3</td>
<td>6.16</td>
<td>6.28</td>
<td>1.9</td>
</tr>
<tr>
<td>MCC5</td>
<td>100</td>
<td>1.2</td>
<td>10</td>
<td>0.3</td>
<td>12.77</td>
<td>12.40</td>
<td>-2.9</td>
</tr>
<tr>
<td>MCC6</td>
<td>100</td>
<td>0.772</td>
<td>1.1</td>
<td>0.3</td>
<td>2.26</td>
<td>2.30</td>
<td>1.8</td>
</tr>
<tr>
<td>MCC7</td>
<td>100</td>
<td>1.636</td>
<td>1.1</td>
<td>0.3</td>
<td>4.26</td>
<td>4.10</td>
<td>-3.8</td>
</tr>
<tr>
<td>MCC8</td>
<td>100</td>
<td>1.2</td>
<td>1.1</td>
<td>0.2</td>
<td>3.29</td>
<td>3.22</td>
<td>-2.1</td>
</tr>
</tbody>
</table>

* $\sigma_a$: cavity expansion pressure at the cavity wall

Table 7.3 Comparison of normalised excess pore pressures around the cavity at $a/a_0=2$ between PLAXIS results and the closed-form solutions presented by Cao et al. (2001)

<table>
<thead>
<tr>
<th>CASE</th>
<th>$G/p'_0$</th>
<th>$M$</th>
<th>$R$</th>
<th>$\nu'$</th>
<th>$\Delta u/p'_0$ (closed-form)*</th>
<th>$\Delta u/p'_0$ (PLAXIS)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCC1</td>
<td>100</td>
<td>1.2</td>
<td>1.1</td>
<td>0.3</td>
<td>3.14</td>
<td>3.04</td>
<td>-3.2</td>
</tr>
<tr>
<td>MCC4</td>
<td>100</td>
<td>1.2</td>
<td>3</td>
<td>0.3</td>
<td>4.72</td>
<td>4.71</td>
<td>-0.2</td>
</tr>
<tr>
<td>MCC5</td>
<td>100</td>
<td>1.2</td>
<td>10</td>
<td>0.3</td>
<td>7.76</td>
<td>7.95</td>
<td>2.4</td>
</tr>
</tbody>
</table>

* $\Delta u_c$: excess pore pressure at the cavity wall
7. Analysis of CPT resistance at variable penetration rate

Figure 7.7 Comparison of normalised spherical cavity pressure variations between PLAXIS results and the closed-form solutions presented by Cao et al. (2001)

Figure 7.8 Comparison of normalised excess pore pressure variations around the spherical cavity between PLAXIS results and the closed-form solutions presented by Cao et al. (2001)
7.3.2 Drained spherical cavity expansion

Drained spherical cavity expansion pressures computed in PLAXIS are compared here to the closed-form solution of Yu & Houlsby (1991) using the Mohr-Coulomb (MC) model (see Figure 7.9). Three different cases (MC1 to MC3), involving varying stiffness ($E$), friction angle ($\phi'$) and dilation angle ($\psi$) were analysed using MC model in PLAXIS by following the same analysis procedure as explained above. It was noticed that the cavity expansion curves for relatively stiff soils encountered instability, which may reflect deficiencies in the PLAXIS numerical iteration procedures. In order to solve this problem (if encountered), calculation control parameters were modified slightly from Default values in the PLAXIS ‘Calculation’ program, such as the Tolerated error of 0.03 (default=0.01), the Over-relaxation coefficient of 1.1 (default=1.2) and the Maximum number of iterations of 100 (default=60); see PLAXIS REFERENCE MANUAL (Brinkgreve et al. 2012).

The PLAXIS numerical model provides similar cavity expansion pressures up to $a/a_0=2$ to those derived using Yu & Houlsby (1991), although the expanding pressure curves calculated in PLAXIS tended to stop due to the problems mentioned above encountered at larger strains in stiff soils (such as the MC2 and MC3 cases). The differences between the PLAXIS computed pressures at $a/a_0=2$ and the closed-form solution is less than 10% (as summarised in Table 7.4). The vast majority of the total increase in the expansion pressure occurs within $a/a_0=2$ (as also observed in the undrained spherical cavity expansions).

### Table 7.4 Comparison of normalised spherical cavity pressures between PLAXIS results and the closed-form by Yu & Houlsby (1991)

<table>
<thead>
<tr>
<th>CASE</th>
<th>$p_0'$ (kPa)</th>
<th>$E$ (MPa)</th>
<th>$\nu$ (-)</th>
<th>$c$ (kPa)</th>
<th>$\phi'$ (°)</th>
<th>$\psi$ (°)</th>
<th>$p_1$ ($p_{\text{limit}}^{-1}$)</th>
<th>$p_2$ (PLAXIS)</th>
<th>$(p_2-p_1)/p_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC1</td>
<td>120</td>
<td>5</td>
<td>0.2</td>
<td>0.1</td>
<td>20</td>
<td>0</td>
<td>527 (550) 543</td>
<td>2.9 (-1.3)</td>
<td></td>
</tr>
<tr>
<td>MC2</td>
<td>120</td>
<td>100</td>
<td>0.2</td>
<td>0.1</td>
<td>42</td>
<td>12</td>
<td>7766 (8913) 8419</td>
<td>8.4 (-5.6)</td>
<td></td>
</tr>
<tr>
<td>MC3</td>
<td>120</td>
<td>50</td>
<td>0.2</td>
<td>0.1</td>
<td>40</td>
<td>0</td>
<td>2817 (3010) 2872</td>
<td>1.9 (-4.6)</td>
<td></td>
</tr>
</tbody>
</table>

$p_1$ and $p_2$ are the expanding pressures at $a/a_0=2$ while $p_{\text{limit}}$ is the limit pressure according to the closed-form solution.
7. Analysis of CPT resistance at variable penetration rate

7.4 ANALYSIS RESULTS

7.4.1 Analysis cases

An extensive parametric study to evaluate parameters influencing penetration rate effects was performed using the HS model in PLAXIS. The basic parameters regularly used for all analysis cases are listed in Table 7.5 and all variables are summarised in Table 7.6. In each soil type, variable cavity expansion velocities ranging over eight orders of magnitude were analysed by changing the consolidation time in each calculation phase. In total, 47 cases were analysed and all cases were categorised into seven groups as follows (and in Table 7.6):

*Group I:* 15 cases (HS01 to HS15) study effects of soil stiffness \((E)\) and a friction angle \((\phi')\) on the cavity expansion pressure (or cone end resistance), but a dilation angle \((\psi)\) was maintained as zero. \(E_{50}^{ref}\) (2, 5, 10, 30 and 50 MPa) and \(\phi'\) (20, 30 and 40°) are combined in the 15 cases but actual stiffnesses vary slightly due to the assumed anisotropic (normally consolidated) condition (i.e. \(K_0=1-\sin\phi'\)). Soil isotropic permeability is maintained constant at 0.0003 m/day (equivalent to 3.5×10⁻⁹ m/s), which is a typical value in many clayey soils.

*Group II:* Six cases (HS16 to HS21) investigate effects of the coefficient of earth pressure at rest \((K_0)\). An isotropic condition (i.e. \(K_0=1\)) is used (while keeping the model

Figure 7.9 Comparison of normalised spherical cavity pressure variations between PLAXIS results and the closed-form solutions presented by Yu & Houlsby (1991)
parameter $K_0^{NC}=1-\sin\phi$) for these six cases, where the soils had two different stiffness values $E_{50}^{ref}$ (5 and 30 MPa) combined with three different friction angles $\phi$ (20, 30 and 40°). Comparisons are made with Group I focusing on effects of $K_0$ on the cavity expansion pressure.

**Group III:** Seven cases (HS22 to HS28) examine effects of hydraulic conductivity, by changing the horizontal and vertical permeability ($k_h$ and $k_v$), but maintaining all other soil parameters the same as those employed in the HS08 case. Firstly, isotropic permeabilities ($k_h=k_v$) are changed to 0.003, 0.03, 0.3, 3, 30 m/day (30 m/day is equivalent to $3.5\times10^{-4}$ m/s). Secondly, anisotropic permeability is specified in terms of the $k_h/k_v$ ratio; $k_h/k_v$ ratios of 2 and 10 are compared with the isotropic permeability case ($k_h/k_v = 1$). It is noted that permeability anisotropy in natural marine clay is not usually large (e.g. Leroueil & Hight 2003).

**Group IV:** One case (HS29) used the initial cavity radius ($a_0$) of 0.3 m whereas all the other cases used $a_0=0.1$ m. This is to examine pore pressure generations and drainage characteristics of the soils surrounding the cavity. In the HS29 case, the cavity was expanded to 0.6m (i.e. $a/a_0=2$) in the same soil domain size as others (with only minor boundary effects expected compared to the $a_0=0.3$m case (Xu 2007). All the other soil parameters were kept as same as those employed in the HS08 case.

**Group V:** Six cases (HS30 to HS35) were analysed with the cavity at an effective stress level ($\sigma'_{v0}$) about five times higher than used in all the other cases. In the six cases, the cavity was at a depth of 60 m (while the cavity was 12 m deep in all other cases). The six cases cover two different stiffness values $E_{50}^{ref}$ (5 and 30 MPa) combined with $\phi$ of 20, 30 and 40°. The results in this Group V are compared with those in Group I to evaluate effects of stress level in the same soils.

**Group VI:** Four cases (HS36 to HS39) involved overconsolidated soils and the results from these are compared with those of the normally consolidated soils in Group I. Overconsolidation ratios (OCR) of 2, 5, 10 and 15 were examined while the other input soil parameters were kept the same as those employed in the HS08 case.

**Group VII:** Eight cases (HS40 to HS47) used non-zero peak dilation angles ($\psi$) to inspect potential effects of dilative or contractive soil behaviours. Stiff dilative soils were modelled with $\psi$ angles of 0.5, 1, and 5° (with all other soil parameters being the
same as those employed in the HS15 case). The coefficient of earth pressure \((K_0)\) was set to be constant of 0.45, instead of using \(K_0=1-\sin\phi'\). The initial void ratio \((e_{int})\) was set to be 0.6 with the maximum and minimum void ratios of 0.8 and 0.5, which implied a relative density \((D_r)\) of 0.67. A dilation cut-off option, which PLAXIS offers in HS model, was selected. Effects of \(\psi\) (2, 5, and 10°) on the drained expansion pressure are only examined because of program limitations (discussed later) associated with predictions involving a dilation angle when the soil is not drained. A very soft contractive soil (on a trial basis) was modelled with \(\psi\) of -0.5° based on HS01 case, noting that negative dilation angle is impractical to use in the HS model. It should be noted that use of a dilation angle in the undrained condition leads to generation of excessive strength due to limitations of the HS model.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\gamma_{sat}) (unit weight)</td>
<td>20</td>
<td>kN/m³</td>
</tr>
<tr>
<td>(e_{int}) (initial void ratio)</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>(c) (cohesion)</td>
<td>1</td>
<td>kPa</td>
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## 7. Analysis of CPT resistance at variable penetration rate

Table 7.6 Analysis program

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7. Analysis of CPT resistance at variable penetration rate

7.4.2 Cavity expansion curves and pore pressure curves

Figure 7.10 shows a typical result of the cavity expansion pressure curve and excess pore pressure curve during the spherical cavity expansion to $a/a_0=2$. The total radial stresses ($\sigma_1$) (i.e. the maximum principle stresses towards expanding direction), the excess pore pressures ($\Delta u$) and the radial displacement at the cavity wall ($|U|$) were averaged from the seven nodes and the seven stress points as described above.

As a result of varying the cavity expansion velocity ($v_{r,CE}$) from 0.00000116 to 116 mm/s (over eight orders of magnitude), undrained to drained conditions were evident in the normalised cavity expanding pressures $(\sigma_1-\sigma_{vo})/\sigma_v'$ and the normalised excess pore pressures $(\Delta u/\sigma_v')$. The $\Delta u/\sigma_v'$ value began to decrease when $v_{r,CE}$ was reduced to 1.16 mm/s followed by larger reductions over two orders of magnitude of $v_{r,CE}$ between 0.116 and 0.00116 mm/s. Significant increases in the $(\sigma_1-\sigma_{vo})/\sigma_v'$ values occurred at $v_{r,CE}$ of 0.116~0.000116 mm/s, where the cavity expansions were likely to be partially drained condition in this HS08 case. Fully drained conditions were confirmed at $v_{r,CE}$ below 0.000116 mm/s with zero (or very close to zero) excess pore pressures generated during the cavity expansion.

Figure 7.10 A typical result on: (a) normalised cavity expanding pressure and (b) normalised excess pore pressures generated closed to the cavity wall (in case of HS08)
7.4.3 Effect of cavity expansion velocity

The cone resistance ($q_c$) was calculated based on the computed cavity expansion pressure (i.e. $\sigma_1$ value at $a/a_0=2$) from Equation (7-10); this was then used to calculate a normalised cone resistance ($Q$) defined as:

$$Q = \frac{q_c - \sigma_{v0}}{\sigma'_{v0}}$$

Figure 7.11 shows $Q$ variations for Group I at different cavity expansion velocities for the soils with various combinations of stiffness and friction angle. It can be observed that rate dependence (within the $v_{r,CE}$ range computed in this study) on cone resistance is clearly evident but it varies in intensity depending on soil stiffness and friction angle. Overall, the $Q$ values increase as $v_{r,CE}$ reduces for all cases, which suggests that drained normalised cone resistance ($Q_D$) is always higher than undrained normalised cone resistance ($Q_{UD}$) for normally consolidated soils.

In soils with higher stiffness for a given friction angle, the normalised cone resistance is higher at any particular velocity. For the soils with $\phi'=20^\circ$ and $E_{50}/p'_0$ (the ratio of the Young’s modulus at 50% strength mobilisation at the initial mean effective stress, $p'_0$, to the value of $p'_0$) ranging from about 20 to 480, the $Q_{UD}$ values vary from 2.0 to 3.4 while the $Q_D$ values vary from 3.6 to 12.5. For the soils with $\phi'=40^\circ$ at the same $E_{50}/p'_0$ range, the $Q$ values vary from 3.5 to 6.0 in undrained conditions but vary more widely from 7.4 to 42 in drained conditions. For a given modulus $E_{50}/p'_0$, the $Q$ values are generally higher in the soils with higher $\phi'$. Further discussion on $Q_D$ and $Q_{UD}$ is provided in the following sections.

Different drainage conditions are observed by varying $v_{r,CE}$. For example, $v_{r,CE}$ values greater than 0.1 mm/s tend to lead to undrained condition while $v_{r,CE}$ values less than 0.0001 mm/s tends to be drained for all cases. Exact transition points from undrained to partially drained or partially drained to fully drained conditions seem to vary with the soil stiffness and friction angle. Greater increases in $Q$ values occur between $v_{r,CE}$ between 0.2 and 0.0002 mm/s, which generally defines the partially drained zone. These analyses indicate that there is a need for about a 3 order of magnitude change in the cone velocity to move from a fully drained to a fully undrained condition. This range is
Figure 7.11 Variations of normalised cone resistance at various representative penetration rates for soils varying stiffness with constant friction angle of: (a) $\phi'=20^\circ$, (b) $\phi'=30^\circ$ and (c) $\phi'=40^\circ$ ($K_0=1-\sin\phi'$)
smaller (by about 2 order changes) than that derived in numerical simulations of Silva (2005) and Jaeger (2012), which were based on the cylindrical cavity expansions. This difference has suggested an importance of a combination of vertical and horizontal drainage around the spherical cavity.

7.4.4 Drained resistance

The drained normalised cone resistances ($Q_D$), determined at $\nu_{r,CE}$ less than 0.00002 mm/s, were extracted from Group I cases and summarised in Figure 7.12. It can be seen that the PLAXIS results on $Q_D$ variations with varying $E_{50}/p'_0$ and $\phi'$ show an almost linear relationship between log $Q_D$ and log $E_{50}/p'_0$. (see Figure 7.12b). For comparison purposes, Yu’s analytical solutions (Yu 2004) are also plotted in Figure 7.12 assuming $E/p'=E_{50}/p'$. Yu (2004) proposed a combined cylindrical and spherical cavity expansion method using the Mohr-Coulomb model with a parameter $F$ reflecting the extent and shape of the plastic zone around the cone; after Yu’s suggestion of a value of between 0.7 and 0.8, $F$ value of 0.7 is used in Figure 7.12.

Overall for a given $\phi'$, the PLAXIS results reveal a lower sensitivity to stiffness than Yu’s solutions with $F=0.7$. For a $\phi'=30^\circ$, the PLAXIS results suggest that $Q_D$ approximately doubles (from 7 to 15) when $E_{50}/p'_0$ is increased from 95 to 480, while $Q_D$ values predicted by Yu (2004) are tripled (from 10.7 to 33) over the same range of $E_{50}/p'_0$. The difference in gradient evident in Figure 7.9b may be associated with the stiffness degradation with strain incorporated in the HS model - as the soil stiffness surrounding cavities in both the plastic and elastic zone has a significant effect on the expanding pressures.

As the drained resistance ($Q_D$) is function of both $\phi'$ and $E_{50}/p'_0$, the following relationship reasonably captures $Q_D$ estimates based on the HS cavity expansion analyses:

$$Q_D = \frac{q_{cD} - \sigma_{v0}}{\sigma'_{v0}} = 4.1(\sin \phi')^{1.48} \left(\frac{E_{50}}{p'_0}\right)^{0.47}, \ (R^2 = 0.98) \quad (7-12)$$

where $q_{cD}$ is the drained cone end resistance estimated according to Equation (7-10). It is noted that this relationship is derived from Group I case analysis, where $\phi'$ ranged
from 20 to 40°, $E_{50}/p'_0$ ranged from 20 to 480 with OCR=1, $K_0=1-\sin\phi'$, and no shear induced volume change was considered.

Figure 7.12 Induced drained normalised cone resistance as a function of friction angle and stiffness on: (a) normal scale and (b) log-log scale ($K_0=1-\sin\phi'$)
7. Analysis of CPT resistance at variable penetration rate

7.4.5 Undrained resistance

In a similar fashion to the treatment of $Q_D$ above, the undrained normalised cone resistance ($Q_{UD}$) values were extracted from the Group I analyses (when $v_{r,CE}$ values were greater than 10 mm/s) and summarised as a function of stiffness and friction angle in Figure 7.13. As stiffness/ friction angle increases, the $Q_{UD}$ values increase but variations in $Q_{UD}$ are less significant compared to the $Q_D$ variations for the soils analysed in this study. It is also seen that there is an approximate linear relationship between log $Q_{UD}$ and log $E_{50}/p'$.

The undrained cone resistance ($q_{c,UD}$) is often related to undrained strength ($s_u$) defined as (e.g. Lunne et al. 1997b):

$$q_{c,UD} = N_{kt} s_u + \sigma_{v0}$$  \hspace{1cm} (7-13)

where $N_{kt}$ is the cone factor and $\sigma_{v0}$ is the total overburden stress. Lu et al. (2004) performed large-displacement finite element (LDFE) analysis of cone penetration in clay and concluded from their parametric study that $N_{kt}$ is influenced by a soil rigidity index ($I_r$), an initial stress anisotropy ($\Delta$) and a cone roughness ($\alpha_{int}$); the following expression was proposed:

$$N_{kt} = 3.4 + 1.6 \ln I_r - 1.9 \Delta + 1.3 \alpha_{int}$$  \hspace{1cm} (7-14)

where $I_r$ ranges between 50 and 500, $\Delta$ ranges -0.5 to 0.5 and $\alpha_{int}$ varies from 0 (smooth) to 1 (rough).

Yi et al. (2012) derived $Q_{UD}$ values from Equations (7-11) to (7-14), assuming the $s_u$ value to be $M p'_0/2$, where $M$ is the friction coefficient and equal to $6 \sin \phi'/(3-\sin \phi')$. With $\alpha_{int} = \tan \phi'$ and $K_0 = 1-\sin \phi'$(as used in PLAXIS analyses), the $Q_{UD}$ values predicted using this approach are compared with the PLAXIS predictions on Figure 7.13, assuming $E/p'=E_{50}/p'$.

Overall, increasing trends based on the spherical cavity expansion in PLAXIS are very similar to Lu’s LDFE analysis results although the use of different soil models can be expected to cause discrepancies between the $Q_{UD}$ values. Part of this discrepancy is associated with the $s_u$ assumption, which would cause overall resistance to be overestimated for normally consolidated soils.
The $Q_{UD}$ resistance is also function of both $\phi'$ and $E_{50}/p'_0$, and the following relationship based on the PLAXIS cavity expansion analyses reasonably captures the $Q_{UD}$ trend:

$$
Q_{UD} = \frac{q_{cUD} - \sigma_{v0}}{\sigma'_{v0}} = 3.15(\sin \phi')^{0.90} \left( \frac{E_{50}}{p'_0} \right)^{0.17}, \quad (R^2 = 0.99) \quad (7-15)
$$
where \( q_{cUD} \) is the undrained cone end resistance estimated according to Equation (7-10). It is noted again that this relationship is derived from Group I case analysis, for which \( \phi' \) ranged from 20 to 40°, \( E_{50}/p'_0 \) ranged between 20 and 480 with OCR=1, \( K_0 \) was taken as 1-\( \sin \phi' \), and no shear induced dilation/contraction was considered.

### 7.4.6 Normalised cone resistance ratio

The normalised cone resistances \( Q \) at various cavity expansion velocities were normalised by either \( Q_{UD} \) or \( Q_D \) as a reference resistance on Figure 7.14. Ratios of \( Q/Q_{UD} \) have often been discussed in related research, most of which considered clayey soils (e.g. DeJong & Randolph 2012).

It can be observed that \( Q/Q_{UD} \) (and also \( Q_D/Q_{UD} \)) is influenced significantly by stiffness \( (E_{50}/p'_0) \). For example, when \( E_{50}/p'_0 \) values is increased by a factor of 10 (from \( \sim 50 \) to 480), the \( Q_D/Q_{UD} \) ratio doubles (from \( \sim 2.7 \) to 5.4) in the soils with \( \phi'=30° \) (see Figure 7.14a). Another tendency apparent on Figure 7.14a is that the transition to partially drained condition (i.e. when \( Q/Q_{UD} \) ratio increases above unity) occurs at higher velocities (\( v_{r_CE} \sim 1 \) mm/s) in the stiffer soils. For example, there is \( \sim 8\% \) increase in \( Q/Q_{UD} \) when \( v_{r_CE} \) changed from 1.2 to 0.12 mm/s in HS02 , while there is \( \sim 7\% \) increase in \( Q/Q_{UD} \) when \( v_{r_CE} \) changed from 12 to 1.2 mm/s in HS14.

The computed \( Q_D \) values were used for resistance normalisation in the alternative representation in Figure 7.14b. As may be inferred from Figure 7.14a, the \( Q/Q_D \) ratios are lower at larger stiffness moduli \( (E_{50}/p'_0) \). For example, \( Q_{UD}/Q_D \) is \( \sim 0.5 \) in the soil with \( E_{50}/p'_0 \sim 20 \) whereas \( Q_{UD}/Q_D \) is only \( \sim 0.2 \) in the soil with \( E_{50}/p'_0 \sim 480 \).

The \( Q_D/Q_{UD} \) and \( Q_{UD}/Q_D \) ratios for the normally consolidated soils with different \( \phi \) values of 20, 30, 40° are summarised in Figure 7.15a and 7.15b, respectively. The \( Q_D/Q_{UD} \) variation with friction angle is relatively small ranging from \( \sim 1.8 \) to 2.1 at the lowest stiffness \( (E_{50}/p'_0\sim 20) \) but \( Q_D/Q_{UD} \) variations are evidently more sensitive to soil stiffness. As both \( Q_D \) and \( Q_{UD} \) values show approximately linear relationships with \( E_{50}/p'_0 \) on the log-log plot (see Figure 7.12b and Figure 7.13b), an approximate linear trend on a log-log plot is again suggested for the relationship between \( Q_D/Q_{UD} \) and \( E_{50}/p'_0 \).
Figure 7.14 Variations of normalised cone resistance at various representative penetration rates for soils varying stiffness with $\phi' = 30^\circ$ on: (a) $Q/Q_{UD}$ and (b) $Q/Q_D$ ($K_0 = 1 - \sin \phi'$)

With both $Q_D$ and $Q_{UD}$ being approximated as functions of $\phi'$ and $E_{50}/p'_0$ as derived in Equations (7-12) and (7-15), the following relationships on $Q_D/Q_{UD}$ and $Q_{UD}/Q_D$ ratios are inferred:

\[
\frac{Q_D}{Q_{UD}} = 1.30(\sin \phi')^{0.58} \left(\frac{E_{50}}{p'_0}\right)^{0.3}, (R^2 = 0.95) \tag{7-16}
\]

\[
\frac{Q_{UD}}{Q_D} = 0.77(\sin \phi')^{-0.58} \left(\frac{E_{50}}{p'_0}\right)^{-0.3}, (R^2 = 0.95) \tag{7-17}
\]

where $\phi'$ ranges from 20 to 40°, $E_{50}/p'_0$ ranged from 20 to 480 with OCR=1, $K_0=1-\sin \phi'$, and no shear induced dilation/contraction is considered.
This finding contrasts with the observations of Yi et al. (2012), who conducted LDFE analyses using an elastic-perfectly plastic soil with a non-dilatant Drucker-Prager yield criterion. The authors concluded that \( \frac{Q_D}{Q_{UD}} \) for a given modulus \( (G/p') \) is relatively independent of a friction coefficient, \( M = 6\sin\phi'/(3-\sin\phi') \) and generally follows the relationship defined as:

\[
\frac{q_D}{q_{ref}} = 0.022 \frac{G}{p'} + 1.331 \tag{7-18}
\]

where \( q_D \) is the drained net cone resistance, \( q_{ref} \) is the reference (undrained) net cone resistance. As the effective stress \( (\sigma'_{vo}) \) is constant for \( q_D \) and \( q_{ref} \) at a particular penetration depth, a \( q_D/q_{ref} \) ratio is equivalent to \( Q_D/Q_{UD} \) enabling Equation (7-18) to be drawn on Figure 7.15 assuming \( E/p' = E_{50}/p' \), where \( G = E/(2(1+\nu)) \) with \( \nu = 0.2 \) is used. Based on Equation (7-18), \( Q_D/Q_{UD} \) values range from 1.7 to 4.4 as \( G/p' \) is increased from 17.5 to 140. The trends of \( Q_D/Q_{UD} \) variations obtained in the PLAXIS results are in broad general agreement with those of Yi et al. (2012) but the PLAXIS results suggest that \( Q_D/Q_{UD} \) values also vary with the friction angle in the HS model, especially at higher normalised stiffness values \( (E_{50}/p'_0) \). According to Equation (7-16), \( Q_D/Q_{UD} \) values range from 1.71 to 6.41 as \( E_{50}/p'_0 \) is increased from 20 to 480 for the soils with \( \phi' \) in the range of 20 to 40°.

Figure 7.15b also shows that the \( Q_{UD}/Q_D \) values are dependent on both \( E_{50}/p'_0 \) and \( \phi' \). It seems that the \( Q_{UD}/Q_D \) values are strongly dependent of stiffness \( (E_{50}/p'_0) \), as the \( Q_{UD}/Q_D \) values decrease from 0.5 to 0.2 as \( E_{50}/p'_0 \) is increased from 20 to 480 for the soils with \( \phi'=30^\circ \). For a given \( E_{50}/p'_0 \), \( \phi' \) also affects the \( Q_{UD}/Q_D \) albeit with a lesser influence than stiffness.
7. Analysis of CPT resistance at variable penetration rate

The coefficient of earth pressure at rest ($K_0$) strongly influences cone resistances at any velocities computed in this study (see Figure 7.16a). Compared to soils with $K_0=1-\sin \phi'$ employed in the original cases (HS10 to HS12 cases), soils with isotropic stress conditions ($K_0=1$) (HS19 to HS21 cases) resulted in higher $Q$ values at any velocities. This is simply because $K_0$ value of unity provides higher horizontal effective stress field ($\sigma'_{ho}$) for a given vertical effective stress ($\sigma'_{vo}$) around the cavity. As defined in Equations (7-2) to (7-4) in the HS model, higher initial stiffness values of $E_{50}$ and $E_{ur}$
are induced from higher $\sigma'_{h0}$. Thus, a higher expanding pressure is required to expand the cavity.

The effect of $K_0$ is examined here by comparing results obtained with $K_0=1$ and $K_0=1-\sin\phi'$ at two different stiffness levels ($E_{50}/p'_0$ of $\sim50$ and $\sim290$); these results are summarised in Figure 7.16. It can be seen that soils with a high friction angle and $K_0 = 1$ have much higher resistances. However, there seems to be little difference between a drained resistance ratio and undrained resistance ratio for a given stiffness and a given friction angle (see Figure 7.16b). Furthermore, those drained and undrained resistance ratios are likely to be independent to a modulus ($E_{50}/p'_0$) as similar ratios are obtained at two different moduli ($E_{50}/p'_0$ of $\sim50$ and $\sim290$).

![Graph](image.png)

Figure 7.16 Effect of in situ earth pressure: (a) $Q$ against velocity and (b) resistance ratios
7.4.8 Effect of permeability

Analyses demonstrated that varying the isotropic permeability \((k_h = k_v)\) of the soil by an order of magnitude led to a shift of the \(Q-v_{r,CE}\) curve by one order to the right (see Figure 7.17a) e.g. an increase by a factor of 10 of the velocity at which undrained conditions existed. Also, at a given \(v_{r,CE}\) of 10 mm/s, for example, a soil with \(k < 0.003\) m/day penetration is undrained whereas for the same soil type but with \(k > 30\) m/day penetration tends to be almost fully drained (for the case analysed with \(E_{50}/p'_0 \sim 95\) and \(\phi = 30^\circ\)). These analyses confirmed the unique dependence on the \(v_{r,CE}/k\) ratio of the resistance in a given soil with isotropic permeability with a given stiffness.

Effects of anisotropic permeability are examined in Figure 7.17b, where the \(k_h/k_v\) ratios of 2 and 10 are compared with the isotropic permeability of 0.0003 and 0.003 m/day in the same soil type \((E_{50}/p'_0 \sim 95\) and \(\phi = 30^\circ\)). When \(k_h\) is doubled to 0.0006 m/day, the \(Q-v_{r,CE}\) curve is shifted to right, which indicates that the soil has faster drainage characteristics compared to the original case (HS08 case). When the \(k_h\) is increased to 0.003 m/day (but \(k_v\) is maintained at 0.0003 m/day), the \(Q-v_{r,CE}\) curve is moved further to the right but does not merge with that of the soil with \(k_h = k_v = 0.003\) m/day (HS22 case). This observation confirms that a combination of both horizontal and vertical permeability \((k_h \text{ and } k_v)\) contributes to drainage characteristics during expansion of the cavity (i.e. cone penetration) but that \(k_h\) effects are greater than \(k_v\).

Further investigations on the effect of permeability are presented in Figure 7.18. Considering observations above, two normalised velocity terms \((v_{r,CE}/k_{h0}\) and \(v_{r,CE}/k_{v0}\)), where \(k_{h0}\) and \(k_{v0}\) (as same as \(k_h\) and \(k_v\) in this case) represent the horizontal and vertical permeability at the in situ stress level, are used to assess variable cavity expansion rate effects. It can be seen, for the cases examined, that \(v_{r,CE}/k_{h0}\) is a better normalisation term than \(v_{r,CE}/k_{v0}\) for defining general drainage conditions when anisotropic permeability of soils is evident.

Partially drained conditions occur at \(v_{r,CE}/k_{h0}\) ratios between \(\sim 10^2\) and \(\sim 10^5\), although this ratio may not be comparable with other studies. For example, Abu-Farsakh et al. (2003) found that a potential partial drained condition may be expected at the normalised conductivity \(v/k\) between \(\sim 10^4\) and \(\sim 10^6\). This is because the \(v_{r,CE}/k_{h0}\) normalisation term does not account for other important factors related to consolidation such as soil stiffness and drainage distance. A recent study by Sheng et al. (2014)
commented from the LDFE analyses that drained condition is achieved when \( v/k \) is less than 10, while the undrained condition is achieved when \( v/k \) is greater than 10\(^6\). However in Sheng et al. (2014), the greatest increase in cone resistance occurs at \( v/k \) between \( \sim 10^2 \) and \( \sim 10^5 \).

All calculations in Group III used the same soil parameters as those used in HS08 case, except for the permeability (see Table 7.6). Examination of the calculated \( Q \) values also confirmed, as expected, that any change in permeability only affects \( Q \) values in partially drained conditions but does not alter \( Q_D \) and \( Q_{UD} \) values.

\[
Q = \frac{q_c - \sigma' v_0}{\sigma'_v} \quad \text{Cavity expansion velocity, } v_{CE} \text{ (mm/s)}
\]

(a) 
![Figure 7.17 Effect of permeability: (a) isotropic and (b) anisotropic permeability (\( K_0=1-\sin\phi' \))](image)

(b)
Analysis of CPT resistance at variable penetration rate

Figure 7.18 Normalised penetration velocity using: (a) \( v/k_h \) and (b) \( v/k_v (K_0=1-\sin\phi') \)

7.4.9 Effect of cavity radius

One PLAXIS analysis employed an initial cavity radius \( (a_0) \) of 0.3 m (three times the standard value of 0.1 m) with a final cavity radius displacement \( (a_f) \) of 0.6 m (=2\( a_0 \)) in HS29 case (Group IV). The drainage distance can be expected to be longer and the influenced zone larger than the standard \( a_0=0.1 \) m case (HS08 case from Group I). Comparing two results of the two different \( a_0 \) cases, it is seen on Figure 7.19a that the \( Q-v_{r,CE} \) curve shifts to left with an increase in \( a_0 \). This implies that for a given velocity the cavity expansion is more undrained due to longer drainage path lengths when the cavity is larger.

Assuming a drainage distance in the cavity expansion analysis \( (r_{CE}) \) is proportional to the initial cavity radius \( (a_0) \), (and \( r_{CE}=a_0 \) assumed here), the \( Q \) curves for the two cases are replotted against the product of \( v_{r,CE} \) and \( r_{CE} \) in Figure 7.19b. As a result, the \( Q-v_{r,CE} \) curves are well matched, verifying the need for use of cavity radius (i.e. cone diameter) in a normalised velocity term so that the size of the penetrometer (or pile) and its effect on the drainage path lengths is accounted for.

With the other soil parameters kept same as the original HS08 case \( (E_{50}/p_0^{'} \sim 95, \phi'=30^{\circ} \) and \( k_h=k_v=0.0003 \) m/day), the \( Q_D \) and \( Q_{UD} \) values obtained with \( a_0=0.3 \) m are, as expected, identical to those obtained in the HS08 case. This observation also confirms
the absence of any significant mesh boundary effects even for the cavity expansion from 0.3 to 0.6m radius.

\[ Q = \frac{q_c - \sigma_{v0}}{\sigma_{v0}'}, \quad \text{Cavity expansion velocity, } v_{r, CE} (\text{mm/s}) \]

(a) \hspace{1cm} (b)

Figure 7.19 Effect of initial cavity radius: (a) \( Q \cdot v_{r, CE} \) and (b) \( Q \cdot v_{r, CE} \cdot r_{CE} \) (\( K_0 = 1 - \sin\phi' \))

7.4.10 Effect of stress level

The cavity was simulated at depth of 60m in this Group V analysis while the cavity was located at the 12m depth in the original case. The mesh set-up employed in PLAXIS followed a similar approach to that of Tolooiyan & Gavin (2011), who used a dummy layer at the surface of the soil domain and varied its unit weight to apply a higher initial stress field. In the Group V analysis, the initial total vertical overburden stress (\( \sigma_{v0} \)) and vertical effective stress (\( \sigma_{v0}' \)) at the cavity were 1200 kPa and 611 kPa respectively; \( K_0 \) was retained as \( 1 - \sin\phi' \). In order to simulate the same soil types, reference stiffness values were kept the same (\( E_{50}^{\text{Ref}} \) of 5 and 30 MPa), noting that all stiffness parameters are a function of stress level as defined in Equations (7-2), (7-3) and (7-4).

The higher drained cone resistances \( (q_{dD}) \) are observed at the higher \( \sigma_{v0}' (=611 \text{ kPa}) \) due to higher stress level and higher stiffness of the surrounding soils compared to those at the lower \( \sigma_{v0}' \) of 122 kPa (see Figure 7.20a). The \( E_{50} \) values increased about twofold when \( \sigma_{v0}' \) increased by five times (noting a \( \sqrt{5} \) change would be expected base on Equation (7-2)). The increases in \( E \) values resulted in \( q_{dD} \) values that are 2.5 to 4 times higher. All of the \( Q_D \) and \( Q_{UD} \) values obtained at \( \sigma_{v0}' = 611 \text{ kPa} \) are plotted against \( E_{50}/p_0' \) in Figure 7.20a and b, respectively. By comparing the original case for a given \( E_{50}^{\text{Ref}} \),
both $Q_D$ and $Q_{UD}$ values (which represent resistance normalised by $\sigma'_{v0}$) are much lower and $E_{50}/p'_0$ values are also much lower. This is because increases in $\sigma'_{v0}$ and $p'_0$ outweigh increases in $q_c$ and $E_{50}$, respectively, noting that stiffness is a function of stress level to the power ($m$) of 0.5 in this study. However, the $Q_D$ and $Q_{UD}$ values induced at deeper depth are in good agreement with the relationship with $E_{50}/p'_0$ developed for the shallower depth. It is therefore concluded that Equations (7-12) and (7-15) are relevant to different stress levels.

![Graph showing the effect of overburden stress level on $q_{cd}$, $Q_D$, and $Q_{UD}$](image)

Figure 7.20 Effect of overburden stress level on: (a) drained $q_{cd}$-$E_{50}$, (b) $Q_D$-$E_{50}/p'_0$ and (c) $Q_{UD}$-$E_{50}/p'_0$

As a typical example (using $E_{50}^{ref}=30$ MPa and $\phi'=30^\circ$) of the effects of stress level on $Q$ at variable cavity expansion velocities is shown in Figure 7.21. For a given reference stiffness ($E_{50}^{ref}=30$ MPa), the $Q$ values for the deeper depth (HS34 case) are generally
lower (clearly lower at $v_{r, CE}<0.01 \text{ mm/s}$) than those for the shallower depth (HS11 case). The $Q$ values for HS34 case show similar trends with the HS08 case, except for a gap in partially drained region, such as at $v_{r, CE}=0.01 \text{ mm/s}$. This can be explained by similar moduli obtained in two cases (i.e. $E_{s0}/p'_0=128$ and 96 in HS34 and HS08 cases) but $E_{s0}$ values that differ by about 6.5 times.

![Figure 7.21 Effect of stress level on $Q$-$v_{r, CE}$](image)

### 7.4.11 Use of stiffness in normalising velocity

Although the normalised velocity term of $v_{r, CE}/k_{s0}$ was found to be effective to assess penetration rate effects (see Section 7.4.8), other factors influencing the consolidation process need to be taken into account. The other factors include drainage distance (as discussed in Section 7.4.9) and soil stiffness, both of which affect the rate of consolidation.

Previous research investigating the influence of penetration rate in the partially drained range attempted to unify data using normalised velocity ($V$) originally proposed by Finnie & Randolph (1994) and defined as:

$$V_v = \frac{vd}{c_v} \quad (7-19)$$

where $v$ is the cone penetration rate, $d$ is the cone diameter, $c_v$ is the vertical coefficient of consolidation; the $v$ subscript on $V_v$ refers to the normalised velocity defined using $c_v$. 

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As the primarily direction of drainage is radial as indicated in the analyses in Section 7.4.8, a horizontal coefficient of consolidation ($c_h$) is preferred to assess penetration rate effects. Silva et al. (2006) and Lehane et al. (2009) suggest the use of $c_h$ term from piezcone dissipation tests rather than using $c_v$ values from 1-D consolidation on a laboratory sample.

For the PLAXIS analyses, a $c_h$ value may be calculated as:

$$c_h = \frac{k_{h0}E_{50}}{\gamma_w}$$

(7-20)

where $E_{50}$ is defined at the in situ horizontal effective stress ($\sigma'_{h0}$) and $\gamma_w$ is the unit weight of the water.

A (dimensionless) normalised velocity for the cavity expansion analyses ($V_{h\_CE}$) may then be calculated using Equations (7-19) and (7-20), and considering the influence of the drainage distance as:

$$V_{h\_CE} = \frac{v_{r\_CE}d_{CE}\gamma_w}{k_{h0}E_{50}}$$

(7-21)

where $\gamma_w$ is the unit weight of the water. $d_{CE}$ values were taken as the same as the final cavity diameter (which is two times the final cavity radius, i.e. the four times the initial cavity radius (=4$a_0$)) in the following analysis. A constant $k_{h0}$ value was assumed during expansion and $E_{50}$ adapted here was at the initial stress condition before expansion of the cavity. Using the $V_{h\_CE}$ normalisation term, rate effects on the $Q/Q_{UD}$ and $Q/Q_D$ were assessed for the soils with various stiffness values from the Group I set of analyses with $\phi'=30^\circ$ (see Figure 7.22). Interestingly, there is a good agreement between the curves for various stiffness at $V_{h\_CE}$ values greater than 2. Fully undrained conditions can be found at $V_{h\_CE}$ greater than 30 for all cases, which is clearly indicated on Figure 7.22a. At $V_{h\_CE}$ values lower than 1, the curves diverge towards the various $Q_D/Q_{UD}$ values.
When $Q/Q_D$ is plotted against $V_{h,CE}$ term, it may be seen that fully drained conditions can exist at higher $V_{h,CE}$ values in soils with lower $E_{50}/p'_0$. For example, for a given $V_{h,CE}$ of 0.1, expansion is essentially drained in soil with $E_{50}/p'_0 \approx 20$ and is partially drained in soil with $E_{50}/p'_0 \approx 480$ (see Figure 7.22b). This divergence may be partly due the extent of the excess pore pressure field being different multiples of $d_{CE}$ in soils with different stiffness. Although a stiffness correction to normalised velocity term could be made (analogous to the rigidity term correction applied to the time factor in the Teh and Houlsby solution), such a correction would not unify the curves on Figure 7.19 essentially because the extent of the partially drained range is about three orders of
magnitude change of $V_{h,CE}$ (i.e. $V_{h,CE}$ between about 0.01 and 10) for the range of the normalised stiffness ($E_{50}/p'_0$) investigated.

This partially drained range is also independent of the friction angle, as illustrated on Figure 7.20 for normally consolidated soils with various friction angles ($\phi'=20$, 30 and 40°) and the similar $E_{50}/p'_0$ ratio. From the $Q/Q_D$-$V_{h,CE}$ curves, it can be seen that the transition between fully drained and partially drained conditions is at very similar $V_{h,CE}$ of ~0.01 (see Figure 7.23b).

![Diagram](image)

Figure 7.23 Normalised cone resistance ratios with different friction angles against normalised penetration velocity: (a) $Q/Q_{UD}$ and (b) $Q/Q_D$, ($K_0=1-\sin\phi'$)
Figure 7.24 confirms that drainage transitions are also consistent for soil with different stress levels using the normalised velocity, $V_{h, CE}$. The use of $V_{h, CE}$ is seen to be a substantial improvement on Figure 7.18 which plots the resistances against cavity velocity $V_{r, CE}$. Figure 7.24 shows fully undrained conditions occur at $V_{h, CE}$ greater than 10 while fully drained conditions tend occur at $V_{h, CE}$ smaller than ~0.004.

![Figure 7.24 Normalised cone resistance ratios with different stress levels against normalised penetration velocity on: $Q/Q_{UD} = V_{h, CE}$](image)

### 7.4.12 Use of a hyperbolic function for partially drained resistance

The use of the normalised velocity ($V_{h, CE}$) has been shown to be reasonable means of characterising the rate dependence of penetrometer resistance. As suggested in previous studies (e.g. House et al. 2001, Lehane et al. 2009), a hyperbolic function relating normalised cone resistance ($Q$) and the normalised velocity ($V_{h, CE}$) may be expressed as:

$$Q = \frac{q_c - \sigma_{v0}}{\sigma'_{v0}} = a + \frac{b}{1 + c(V_{h, CE})^d}$$  \hspace{1cm} (7-22)

where $a$, $b$, $c$ and $d$ are constants and $V_{h, CE}$ is given in Equation (7-21). When $V_{h, CE}$ is very small, $Q$ equals to $a+b$ (which is likely to be drained resistance, $=Q_D$). On the other hand, $Q$ is equal to $a$ when $V_{h, CE}$ becomes very large in the undrained region ($=Q_{UD}$).
The effects of $E_{50}/p'_0$ on $Q/Q_{UD}$ and $Q/Q_D$ ratios (for the soils with $\phi'=30^\circ$) in the partially drainage range are well captured using Equation (7-22), as seen in Figure 7.25. The four parameters $a$ to $d$ are also listed in the figure. The two parameters $a$ and $b$ can be determined from $Q_D$ and $Q_{UD}$ values (Equations (7-12) and (7-15)) and both increase with normalised stiffness. The fitting parameter $c$ values also tend to increase as $E_{50}/p'_0$ values increase, whereas the fitting parameter $d$ in independent of soil stiffness and has a constant value of 0.80.

The foregoing has shown that a hyperbolic function, as given in Equation (7-22) is effective in representing the numerical PLAXIS predictions of cone resistance. The strong influence of soil stiffness on the values of $a$, $b$ and $c$ is noteworthy.

Figure 7.25 Comparison of PLAXIS rate effect data (after Figure 7.19) with the hyperbolic curves with the fitting parameters: (a) $Q/Q_{UD}$ and (b) $Q/Q_D$, ($K_0=1-\sin\phi'$)
7.4.13 Prediction of cone resistance for standard CPT

The PLAXIS spherical cavity expansion analysis results presented in the previous sections are used here to predict cone resistance for a CPT at the standard penetration velocity in various soils. A comparison of predicted and measured cone resistances is provided in Section 7.5, where limitations of the approach are outlined. In particular, it is noted that the predictions are not reliable in highly contractant or dilative soils. Therefore only FE analysis results from Groups I, II and V above are examined here in this section.

The net cone resistances \( q_{net} = q_c - \sigma_{v0} \) at a given cavity expansion velocity \( v_{r,CE} \) for Groups I, II and V were affected by soil stiffness \( (E) \), friction angle \( (\phi') \), coefficient of earth pressure at rest \( (K_0) \) and effective stress level \( (\sigma'_{v0}) \).

The analyses above have shown that the variable velocity cavity expansion tests with constant permeability can be converted to constant velocity (at the CPT standard velocity) test with a permeability \( (k^*) \) such that:

\[
\frac{v_{r,CE}d_{CE}}{k_{h0}} = \frac{v_{cone}d_{cone}}{k^*}
\]

where \( v_{cone} \) is the standard penetration rate of 20 mm/s and \( d_{cone} \) is the standard cone diameter of 35.7 mm. In the original FE analyses, \( v_{r,CE} \) was varied while the final cavity diameter \( (d_{CE}) \) and the permeability \( (k_{h0}) \) remained constant. The calculated \( k^* \) values ranged from 5.3\times 10^{-11} to 5.3\times 10^{-3} \text{ m/s} for the analyses conducted.

According to the observations from the PLAXIS analyses, net cone resistance \( q_{net} \) at the standard penetration rate depends primarily on the horizontal effective stress \( (\sigma'_{h}) \), soil stiffness \( (E_{50}^{ref}) \), friction angle \( (\phi') \), and permeability \( (k^*) \) i.e. \( q_{net} \) can be represented as a function of these parameters (noting that \( E_{50}^{ref} \) is stress level independent and essentially reflects the void ratio and structure of the soil deposit):

\[
q_{net} = q_c - \sigma_{v0} = f\{\sigma'_{h}, E_{50}^{ref}, \phi', k^*\}
\]

The following relationship, Equation (7-25) was developed for normally consolidated non-dilative/contractive soils for which \( \sigma'_{h} \) ranged from 40 to 400 kPa, \( E_{50}^{ref} \) ranged...
from 2 to 50 MPa, \( \phi' \) ranged from 20 to 40\(^\circ\), and \( k^* \) ranged from \( 5.3 \times 10^{-11} \) to \( 5.3 \times 10^{-3} \) m/s. These parameters were obtained from the Groups I, II and V FE analyses.

\[
q_{\text{net}} = A(\sigma'_{h})^{n_1}(E_{50}^{\text{ref}})^{n_2}(\sin \phi')^{n_3}
\]

where

\[
A = 2.9 + \frac{6.7}{1 + 10^3(k^*)^{0.5}}
\]

\[
n_1 = 0.75 + \frac{0.125}{1 + 10^3(k^*)^{0.5}}
\]

\[
n_2 = 0.5 - \frac{0.35}{1 + 10^3(k^*)^{0.5}}
\]

\[
n_3 = 2.1 - \frac{0.5}{1 + 10^3(k^*)^{0.5}}
\]

where \( A, n_1, n_2, \) and \( n_3 \) are a function of \( k^* \) respectively as formulated above and presented in Figure 7.26.

![Figure 7.26 Variations of A, n1, n2, and n3 with k* values](image-url)

The net cone resistance \( (q_{\text{net}}) \) values calculated using Equation (7-25) are compared in Figure 7.27 with the \( q_{\text{net}} \) values obtained from the PLAXIS numerical analyses, where a reasonable fit is apparent. Although some variations of \( \pm 20\% \) exist, the standard deviation of about 0.14 indicates that Equation (7-25) predicts the PLAXIS \( q_{\text{net}} \) calculations well.
7. Analysis of CPT resistance at variable penetration rate

7.4.14 Effect of OCR

The numerical analyses described above (Groups I to V) dealt with normally consolidated soils. This section examines predicted rate effects for overconsolidated soils with overconsolidation ratios (OCR) of 2, 5, 10 and 15. Due to the limitations of the HS model in PLAXIS described in Section 7.2.3, these analyses needed to employ a zero dilation angle. Other soil properties adopted were $E_{50}^{\text{ref}}=10$ MPa and $\phi=30^\circ$, and thus they can be compared to the HS08 case with OCR=1.

According to Brinkgreve et al. (2012), the PLAXIS program calculates a coefficient of earth pressure at rest ($K_0$) in the overconsolidated condition as follows:

$$K_0 = K_0^{\text{NC}} \cdot \frac{OCR - 1}{1 - \nu_{ur}}$$  \hspace{1cm} (7-26)

where $K_0^{\text{NC}}$ is the lateral earth pressure in the normally consolidated condition and $\nu_{ur}$ is the unloading/reloading Poisson’s ratio. The $K_0$ values obtained from the ‘Output’ program in PLAXIS followed Equation (7-26) for the soils with OCR=10 but did not match for OCR=15 (see Figure 7.28a) and it would appear that the program limits the $K_0$ values to the Rankine $K_p$ values. Figure 7.28b plots calculated stiffness values of $E_{50}$.
and $E_{uw}$ for various OCRs, according to Equations (7-2) and (7-4). Stiffness increases with OCR as, for any fixed vertical effective stress, horizontal stresses increase as the OCR increases.

The normalised cone resistance ($Q$) values at the cavity expansion velocities ($v_{t,CE}$) are compared in Figure 7.29a for various OCRs. As the OCR increases, the $Q$ values increase for all drainage conditions. This trend is due to the higher stiffness at higher OCRs. When the $Q$ values are normalised by the drained resistance ($Q/Q_D$), the $Q/Q_D$ - $V_{h,CE}$ curves indicate that $Q_{UD}/Q_D$ values increase as OCR increases (see Figure 7.29b); $Q_{UD}/Q_D$ values of 0.3 and 0.4 are observed at OCR=1 and 10 respectively. Equation (7-25) indicates that the $n_1$ exponent to $\sigma'_h$ in the undrained region is slightly higher than that in the drained condition and hence the rise of $Q_{UD}/Q_D$ with OCR is related to the increasing $\sigma'_h$ with OCR.
The $Q_D/Q_{UD}$ ratios (for the soils with $\phi' = 30^\circ$) decrease from ~3.3 to ~2.5 as OCR increases from 1 to 10. Despite the limitations of the PLAXIS analyses for large OCRs (due to the need to keep the dilation angle at zero at partially drained/undrained rates), this predicted $Q_D/Q_{UD}$ variation with OCR is in agreement with the trend observed experimentally using T-bar penetrometers in kaolin by Lehane et al. (2009), who reported the $Q_D/Q_{UD}$ ratio of ~3.1, ~2.8, ~2.3 for OCR=1, 2 and 5 respectively.
7.4.15 Effect of dilative and contractive behaviour

In this Group VII, a positive peak dilation angle (ψ) was used to simulate cone penetration testing in dilative materials. The coefficient of earth pressure (K₀) was set to a constant value of 0.45 and the initial void ratio (eᵢₙₐ) was set to equal to 0.6 with the maximum and minimum void ratios of 0.8 and 0.5; the in-situ e value equates to a relative density (Dᵣ) of 0.67. The dilation cut-off option, which PLAXIS offers in HS model, was selected. It is noted again that, as mentioned in Section 7.2.3, this option over-predicts shear strength hence cone resistance in the undrained region (and partially drained region) as the dilation cut-off option is not functioning correctly in the current version of PLAXIS.

Potential effects of a positive dilation angle on the Q values in a relatively stiff soil (E₅₀/p′₀~480 and ϕ′=40°) are shown in Figure 7.30. For the relatively slow expansions (i.e. drained conditions), it was confirmed that the mobilised dilation angles were controlled as the current void ratios did not exceed the given maximum void ratio i.e. the dilation cut-off option works in the drained analyses. An increase in ψ resulted in Q increasing for all drainage conditions with a greater influence evident in the undrained condition. Even with a very small ψ value of 0.5°, the cavity expansion pressure continues to increase as the cavity expansion velocity increases above 0.1mm/s. Although this is understood to be a limitation of the current version of PLAXIS, as described above, the calculated results presented in Figure 7.30 do indicate that, with a correctly working model, undrained resistance can be greater than drained resistance in highly dilative soils. In addition, it can be surmised that a partially drained resistance can be the lowest resistance, such as seen at νᵢ,CE =0.01 mm/s in the soil with ψ=1° on Figure 7.30.
Figure 7.30 Potential effects of dilatant behaviours on penetration rate effects

Figure 7.31 summarises the results only of drained resistances ($Q_D$) in the stiff soils with $\psi$ values of 0, 2°, 5° and 10° with the dilation cut-off option. The input (peak) friction angles were back-calculated from the peak dilation angles and the constant volume friction angle ($\phi_{cv}$) according to Equation (7-5). Figure 7.31 shows that the $Q_D$ values increase as $\psi$ increases for a given modulus ($E_{50}/p'_0$), and also the effect of dilation angle tends to be similar for the soils with different $E_{50}/p'_0$ ratios (~290 and ~480).

Figure 7.31 Potential effects on $Q_D$ due to dilation angle: (a) $\psi$ and (b) $E_{50}/p'_0$
A negative $\psi$ was (on a trial basis) used to simulate the potential effect of contractant behaviour on cone penetration resistance. The analysis result is shown in Figure 7.32. The $Q$ values with $\psi = -0.5^\circ$ (HS47 case) are lower than those with zero dilation angle (HS01 case) at any given velocity below the drained range. Despite the current limitations of the model mentioned above, this analysis suggests that highly contractive soils have lower undrained resistances during a cone penetration test.

**Figure 7.32 Potential effects of contractant behaviour on penetration rate effects**
7.5 COMPARISON WITH EXPERIMENTAL RESULTS

7.5.1 Assumptions

The PLAXIS spherical cavity expansion analyses described above are compared here with the experimental results of variable penetration rate tests in kaolin clay at 1g; see Chapter 6. The comparison was made using the following assumptions:

- The velocity for the spherical cavity expansion is considered as a resultant in a direction normal to the cone face, which implies \( v_{r,CE} = 2v_{cone} \) for a cone with an apex angle of 60°, where \( v_{cone} \) is the cone penetration velocity in vertical direction.
- The volume of the soil pushed out due to the cavity expansion is equivalent to the volume of the cone tip i.e. \( \frac{4}{3} \pi a_i^3 - \frac{4}{3} \pi a_0^3 = \frac{1}{3} \pi h r_{cone}^2 \) (where \( h \) is the height of the cone tip and \( r_{cone} \) is the radius of the cone penetrometer, see Figure 7.6). For \( a_i/a_0 = 2 \), \( r_{cone} \approx 2.5a_0 \) and thus \( d_{CE} = 0.8d_{cone} \), \( d_{cone} \) is the diameter of the cone penetrometer.
- The equivalent normalised velocity for the cavity expansion (\( V_{h,CE} \)) is calculated according to Equation (7-21).
- The \( Q/Q_D - V_{h,CE} \) profiles, developed for normally consolidated, non-dilative/contractive soils with \( K_0 = 1 - \sin \phi' \) (after Section 7.4.11) are used to obtain \( Q/Q_D \) values corresponding to \( V_{h,CE} \) values. Figure 7.33 presents the PLAXIS prediction for \( Q/Q_D \) vs. \( V_{h,CE} \) for the soils with \( \phi' = 20, 30 \) and 40° and varying \( E_{50}/\sigma_{0}' \) values with \( K_0 = 1 - \sin \phi' \).
- \( Q_D \) values are estimated using Equation (7-12). Accordingly, \( q_{net} \) values (\( = q_c - \sigma_{vo} \)) are obtained for a given velocity and for a given effective stress level.
7. Analysis of CPT resistance at variable penetration rate

Figure 7.33 Normalised cone resistance ratios ($Q/Q_D$) against normalised penetration velocity ($V_{h,CE}$) for normally consolidated and non-dilative soils
7. Analysis of CPT resistance at variable penetration rate

7.5.2 Kaolin clay

The relevant soil parameters for normally consolidated kaolin are summarised below:

- A friction angle ($\phi'$) of 24° in undrained simple shear tests (see Section 3.6.4) is consistent with $\phi' = 24\pm 2°$ in Lehane et al. (2009) and $\phi' = 23-24°$ in Stewart (1990), both of which were obtained in isotropically-consolidated undrained triaxial compression tests.

- The ratio of the undrained Young’s modulus at 50% strength mobilization ($E_{50}$) to the initial mean consolidation stress ($p'_0$) of about 42 to 62 are reported in triaxial compression tests by Stewart (1990).

- Schneider et al. (2007) reported a vertical permeability ($k_v$) of $1\times10^{-9}$ m/s at $\sigma'_v=100$ kPa. Anisotropic permeability with $k_h=2k_v$ is assumed here.

The net cone resistances ($q_{net}$) computed from the PLAXIS CE analyses are compared on Figure 7.34 with the experimental results at 1g (see Chapter 6). The PLAXIS CE analyses provide a very good representation of the $q_{net}$ variations with velocity and hence drainage condition. Effects of modulus ratio (stiffness) on penetration resistance are quite noticeable. The 1 g test results in kaolin are generally bounded by $E_{50}/p'_0$ values of between 30 and 60, which agree with the triaxial data. The agreement seen on Figure 7.34 provides significant confidence in the numerical approach adopted to assess factors affecting the rate dependency of penetration resistance.

![Figure 7.34 Comparison of net cone resistance in kaolin between PLAXIS analysis results and experiment at 1g (presented in Chapter 6)](image-url)
7.5.3 25% kaolin mixture

The PLAXIS CE analysis results were also compared with the experimental results in the 25% kaolin mixture at 1g (see Chapter 6). The related soil parameters for the normally consolidated 25% kaolin mixture are summarised below:

- Peak friction angle ($\phi'$) in the range of 28 to 34° (average of 32°) was obtained in the undrained simple shear tests (see Section 3.6.4). This is slightly higher than the constant volume $\phi_{cv}$ of 28° reported in Jaeger et al. (2010). The $\phi'$ value of 28° is used here.
- The compression index and recompression index of this 25% mixture are about one quarter and one sixth respectively of the corresponding indices of 100% kaolin (see Section 3.6.3). These relative magnitudes suggest a modulus ratio ($E_{50}/p'_0$) in the range of 160 to 240 (compared to the range of 42 to 62 for 100% kaolin).
- A permeability ($k_v$) of $5 \times 10^{-9}$ m/s was measured at a similar stress level to that used in the 1g tests (~55kPa, see Section 3.6.3). Anisotropic permeability with $k_h=2k_v$ is assumed here.

Figure 7.35 Comparison of net cone resistance in the 25% kaolin mixture between PLAXIS analysis results and experiment at 1g (presented in Chapter 6)

A comparison of the PLAXIS predictions for $q_{net}$ in the 25% kaolin mixture and the experimental data at 1g is presented in Figure 7.35. The drained $q_{net}$ values calculated in the PLAXIS CE analysis are fairly similar to the 1g results for the range of anticipated
7. Analysis of CPT resistance at variable penetration rate

modulus ratios. In addition, the computed transition region between drained and partially drained conditions (approximately between $v=0.002$ to $2$ mm/s) is comparable with the $1g$ test results. However, the numerical analysis does not capture the very low $q_{net}$ values measured in undrained conditions in the experiments. The analysis reported in Section 7.4.15 indicates that this likely because of the Hardening Soil model (with $\psi=0$) does not provide realistic predictions for highly contractant soils.

7.5.4 Performance of numerical predictions

The analyses presented in the foregoing ($\psi=0$) have shown that the numerical approach adopted to study penetration rate effects for the CPT:

- provides reasonable predictions of drained penetration resistance ($Q_D$) for 100% kaolin and the 25% kaolin-75% sand mixture. A similar observation was made for $Q_D$ in sand by Xu & Lehane (2008) and Suryasentana & Lehane (2014)
- provides good predictions in terms of magnitude and rate dependency of the CPT resistance in 100% kaolin over the full velocity range
- overestimates the resistance in the partially drained and undrained regions for highly contractant soils such as the 25% kaolin-75% sand mixture; the approach under-estimates resistance in these regions for highly dilative soils.
- predicts the relative importance of strength, stiffness and permeability on the velocity dependence of penetrometer resistance
- provides a reference resistance, corresponding to zero dilation and zero viscous effects, at any given normalised velocity, which enables comparisons with actual soil penetration rate dependency to be interpreted.
CHAPTER 8  ANALYSIS AND DISCUSSION

8.1  INTRODUCTION

This Chapter assesses penetration rate effects in various soil types and formulates proposals to assist interpretation of penetration rate effects. It is known that natural soils can vary widely not just in terms of their composition but also in terms of their response under penetration loading. In this Chapter, penetration rate effects observed in three sets of field experiments (Chapter 4), centrifuge tests (Chapter 5) and 1g tests (Chapter 6) are further evaluated by drawing on the findings of the numerical analyses (Chapter 7). The available database of CPT rate effects in various soil types (Chapter 2) assist general conclusions on rate effects to be summarised. Finally, proposals for variable rate penetration tests are provided to assist geotechnical interpretation.

8.2  ASSESSMENT OF PENETRATION RATE EFFECTS

8.2.1  Data collection

From the literature reviewed in Chapter 2, measured data on penetration rate dependency were collected from both in situ and laboratory tests. Although the penetration tests often did not cover a wide range of velocities, the findings from these tests are of significant value to the interpretation described here. Many field scale penetration tests only employed variable penetration rates between at \( v = 0.2 \) and 20 mm/s. On rare occasions, responses at penetration rates up to at \( v = 200 \) mm/s or as slow as 0.01 mm/s are reported. The field tests performed in this study included the slowest CPTs performed to date (see Chapter 4).

Figure 8.1 compares data collected from in situ penetration rate effect studies. This figure plots CPT or CPTU tip resistances (\( q_c \) or \( q_t \)) relative to those values at \( v = 20 \) mm/s against the penetration rate employed at each test site. These data were digitally collected from the graphs in the related literature. It should be noted that averaged data over specified depths were used for some cases to reflect the average penetration rate
effects at the respective sites. It can be concluded from Figure 8.1 that rate dependency of penetration tip resistance is highly variable and is significantly influenced by the soil type. It is evident that most CPT penetration resistances within a velocity range of 2 < v < 200 mm/s are within ±40% of the cone resistances at the standard penetration rate (i.e. v = 20 mm/s) in the 15 soils included in Figure 8.1.

**Figure 8.1 Data collection from in situ rate dependent observations**

### 8.2.2 Operational coefficient of consolidation

During penetration of a cone, an ‘operational’ coefficient of consolidation is essentially a function of horizontal permeability ($k_h$) and constrained modulus ($D'$) representing mostly unloading-reloading behaviour. The following relationship for $c_h$ was proposed by Lehane et al. (2009):

$$c_h = \frac{k_h D'}{\gamma_w} = \frac{k_h(1 + e)\sigma'_h}{\gamma_w \sqrt{\lambda \kappa}}$$  \hspace{1cm} (8-1)

where $\gamma_w$ is the unit weight of water, $e$ is the void ratio, $\sigma'_h$ is the horizontal in-situ effective stress (= $K_0 \sigma'_v$), $\lambda$ is the normally consolidated compression index ($\sim C_v/2.3$) and $\kappa$ is the recompression/swelling index ($\sim -C_v/2.3$). It is noted that $k_h$, $e$ and $\sigma'_h$ vary
during the penetration process at a given penetration velocity and hence the initial (pre-penetrometer installation) parameters are used to determine $c_h$ (e.g. Lehane et al. 2009).

One method to estimate an in-situ operational $c_h$ value is to conduct a dissipation test during the piezocone penetration process. This approach can be expected to provide a reasonable $c_h$ value operating during undrained penetration. The Teh & Houlsby (1991) formulation is most commonly used to derive $c_h$ and gives $c_h$ as:

$$c_h = \frac{T^*}{t} r^2 \sqrt{I_r} \quad (8-2)$$

where $T^*$ is the normalised time factor ($=0.245$ for $50\%$ dissipation), $t$ is the time, $r$ is the cone radius and $I_r$ is the soil rigidity index ($=G/s_u$). The $50\%$ of consolidation time ($t_{50}$) is often used to estimate $c_h$ according to Equation (8-2). Some points to note concerning $c_h$ and the Teh & Houlsby (1991) method are as follows:

1. Teh & Houlsby (1991) accounts for dissipation after undrained penetration in normally to lightly consolidated clays. It cannot be applied to partially drained penetrometer installation and must be adapted to allow $c_h$ estimates in heavily overconsolidated clays.
2. The method assumes that the soil can be represented as a linear elastic perfectly plastic material. It therefore requires an assessment of $I_r$ ($=G/s_u$) which cannot be measured as the $G$ variation of soil with shear strain is non-linear. $I_r$ (which is inversely proportional to the shear strain at failure) is commonly assumed to vary between 50 and 500 in natural clays; this variation can lead to inferred $c_h$ values which can vary by a factor of 3.
3. Schnaid et al. (1997), and Krage et al. (2014) recommend $G_{50}$ to be used for $I_r$ predictions, where $G_{50}$ is the shear modulus at half the mobilised strength ($s_u$). An additional uncertainty is the strain rate dependence of undrained shear strength (e.g. Kulhawy & Mayne 1990, Lunne & Andersen 2007).
4. Partially drained penetration delays $t_{50}$ and leads to an underestimation of the actual $c_h$ value of the soil (Silva 2005, Schneider et al. 2007, DeJong & Randolph 2012).
5. $c_h$ values obtained from piezocone tests are typically 10 times lower than in situ overconsolidated $c_h$ values and 10 times higher than in situ normally-consolidated $c_h$ values (Leroueil & Hight 2003).
In this study, $c_h$ values from dissipation tests (following undrained penetration) are primarily used for evaluation of rate dependence in the partially drained range. In some cases, $c_v$ values are alternatively used due to data non-availability. It is acknowledged that (i) the operational coefficient of consolidation is higher than that the normally consolidated value and (ii) soil structure and heterogeneous stratified soil layers (with a high lateral permeability) lead to a higher coefficient of consolidation than the laboratory $c_v$ value.

### 8.2.3 Probe diameter

As suggested in the PLAXIS numerical analyse, the cavity size related to the probe diameter ($d$) is one of key factors to assess penetrometer rate effects in a formulation of the normalised velocity; see in the next section. Instead of a diameter ($d$), an equivalent diameter of the penetrometer ($d_e$) are suggested by Chung et al. (2006) to be used for T-bar penetrometers, following the investigation of T-bars with different aspect ratios ($l/d$, length to diameter of the cylindrical probe). The $d_e$ is defined as:

$$d_e = \sqrt{\frac{4A_p}{\pi}}$$

where $A_p$ is the penetrometer projected area ($=dl$ for T-bar). As $l>d$ for a T-bar probe, $d_e>d$ and a correspondingly greater $V_h$ value is obtained, while no change in $V_h$ occurs for conical and ball probes as $d_e=d$ is maintained. In the evaluation of T-bar penetrometer rate effects below, $d_e$ is used for comparison between the different probes when they are used.

### 8.2.4 Use of normalised velocity

The PLAXIS spherical cavity expansion numerical analyses presented in Chapter 7 have shown that that drainage characteristics during cavity expansion (representing cone penetration) are governed by (i) horizontal permeability (and vertical permeability to a lesser extent), (ii) cavity size (i.e. probe size with respect to the drainage path length) and (iii) soil stiffness. Taking account these factors, a normalised velocity ($V$) using a horizontal coefficient of consolidation ($c_h$) is preferable:

$$V_h = \frac{vd}{c_h}$$
8. Analysis and discussion

where \( v \) is the penetrometer velocity and \( d \) is the probe diameter; \( d_e \) for T-bar probe according to Equation (8-3). Equation (8-4) was originally proposed by Finnie & Randolph (1994), employing a vertical coefficient of consolidation \((c_v)\), and this \( c_v \) form has been adopted widely by many researchers.

The validity of Equation (8-4) is examined on Figure 8.2 using field test results obtained at Bassendean and Burswood (presented in Chapter 4). This figure compares the variations of normalised penetration resistance \((Q/Q_D)\) with the penetrometer velocity \((v)\) and the normalised velocity \((V_h)\), where \( Q \) is the normalised cone resistance and \( Q_D \) is the drained normalised cone resistance. The three soils horizons examined are indicated in Table 8.1 and the quoted \( c_h \) values were determined from dissipation tests at these depths; see Sections 4.3.5 and 4.4.3.

Regression analyses were performed to obtain the best fit hyperbolic equations to fit the field measurements obtained at each soil layer. The form of the hyperbolic equation employed, originally proposed by House et al. (2001), was as follows:

\[
Q = \frac{q_{\text{net}}}{\sigma'_v v_0} = a + \frac{b}{1 + cV^d} \tag{8-5}
\]

where \( Q \) is the normalised cone resistance, \( q_{\text{net}} \) is the net cone resistance \((=q_t - \sigma_v)\), \( \sigma'_v \) is the in situ vertical effective stress, \( V \) is the velocity term and \( a, b, c \) & \( d \) are the fitting parameters. Equation (8-5) accounts for partial consolidation effects on cone resistance during the penetration but the soils are assumed to be inviscid materials.

When \( V \) is very small or very large, penetrations are essentially fully drained or fully undrained, and hence drained or undrained resistance \((Q_D \text{ and } Q_{UD})\) are given as:

\[
Q_D = a + b \ , \ Q_{UD} = a \tag{8-6}
\]

Combining Equations (8-5) and (8-6), \( Q \) values can be expressed as follows using a reference \( Q \) value of \( Q_{UD} \) or \( Q_D \).

\[
\frac{Q}{Q_{UD}} = 1 + \frac{b/a}{1 + cV^d} \tag{8-7}
\]

\[
\frac{Q}{Q_D} = \frac{1}{a + b} \left[ a + \frac{b}{1 + cV^d} \right] \tag{8-8}
\]
Equation (8-8) is modified to compare use of the penetrometer velocity \((v)\) and the normalised velocity \((V_h)\), defined as:

\[
\frac{Q}{Q_D} = \frac{1}{a_1 + b_1} \left[ a_1 + \frac{b_1}{1 + cv^d} \right] \tag{8-8a}
\]

\[
\frac{Q}{Q_D} = \frac{1}{a_2 + b_2} \left[ a_2 + \frac{b_2}{1 + cV_h^d} \right] \tag{8-8b}
\]

It is noted again that Equations (8-8a) and (8-8b) do not incorporate any gains in resistance at high velocities due to viscous effects. The regression analyses therefore ignored the sections of the data where such gains were evident.

If undrained conditions are presumed to have been reached after a 95\% drop from the maximum (drained) resistance (so that \(Q_{UD}=a+0.05b\)) and if drained conditions are presumed after an increase of 95\% from the undrained cases (with \(Q_D=a+0.95b\)), the best fit regression curves indicate the thresholds indicated in Table 8.1 between drained and partially drained conditions and between partially drained and undrained conditions.

It is clear from Table 8.1 that the range of \(V_h\) values at the drained-partially drained and partially drained-undrained thresholds is considerable less than that of penetrometer velocity. The \(V_h\) value marking the onset of drained conditions is 0.028 ±0.008 whereas corresponding velocities \((v)\) vary by a factor of 30 between 0.0003 mm/s and 0.009 mm/s. The \(V_h\) value at undrained conditions is 7±5 whereas corresponding velocities \((v)\) also vary by over 30 times between 0.09 mm/s and 3 mm/s. The data on Figure 8.2 and Table 8.1 therefore confirm the hypothesis that normalised velocity, as defined by Equation (8-4), provides a useful means of generalising penetrometer rate dependency in different soils and for different penetrometer sizes. Partially drained conditions are seen to occur over about 2 and a half orders of magnitude change in \(V_h\).

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth</th>
<th>(c_h)</th>
<th>(v) (mm/s)</th>
<th>(V_h) (mm/s)</th>
<th>(v) (mm/s)</th>
<th>(V_h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bassendean</td>
<td>3 to 4</td>
<td>252</td>
<td>0.009</td>
<td>0.036</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>Bassendean</td>
<td>5 to 6</td>
<td>126</td>
<td>0.001</td>
<td>0.03</td>
<td>0.6</td>
<td>6</td>
</tr>
<tr>
<td>Burswood</td>
<td>6 to 7</td>
<td>16</td>
<td>0.0003</td>
<td>0.02</td>
<td>0.09</td>
<td>2.3</td>
</tr>
</tbody>
</table>

*Note that these thresholds are indicative as they represent estimates corresponding to 95\% drained resistance and 95\% drop from the drained resistance*
Use of $c_h$

Observed variations of $Q/Q_D$ with penetrometer velocity ($v$) are shown on Figure 8.3a for normally consolidated 100% kaolin and 25% kaolin-75% sand mixture measured at elevated $g$ in the centrifuge and at 1g in the laboratory (Chapters 5 and 6). Additional data from other sources are also provided. No $c_h$ results for the kaolin-sand mixture were available and therefore the normalised velocity in this instance, referred to as $V_v$, is defined using the normally consolidated coefficient of consolidated measured in Rowe cell tests ($c_{vNC}$). Figure 8.3b plots the $Q/Q_D$ data against $V_v$. A similar regression procedure was conducted to that described for the Bassendean and Burswood clays above and the assessed threshold $v$ and $V_v$ values are summarised in Table 8.2.
8. Analysis and discussion

(a)

(b)

Figure 8.3 Use of $c_v$ for normalising velocity in kaolin and the 25% kaolin mixture

<table>
<thead>
<tr>
<th>Site</th>
<th>$c_{vNC}$ (m$^3$/yr)</th>
<th>Drained to Partially Drained</th>
<th>Partially Drained to Undrained</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$v$ (mm/s)</td>
<td>$V_v$ (mm/s)</td>
<td>$V_v$</td>
</tr>
<tr>
<td>Kaolin</td>
<td>2-3.5</td>
<td>0.0007</td>
<td>0.08</td>
</tr>
<tr>
<td>25% kaolin</td>
<td>20-50</td>
<td>0.0033</td>
<td>0.017</td>
</tr>
</tbody>
</table>

Table 8.2 Comparison of drainage transitions using $v$ and $V_v$ using laboratory test results

It is evident on comparison of Figure 8.3a and 8.3b and on inspection of Table 8.2 that, unlike $V_h$, the use of $V_v$ does not bring the trends for both soil types together. In fact it is seen that threshold values of (non-normalised) velocity are very similar for both soil types and the threshold $V_v$ values for kaolin are between 5 and 20 times higher than those of the 25% kaolin-75% sand mixture. It would appear that $c_{vNC}$ values obtained in
one dimensional consolidation are not reflective of the consolidation mechanism operating during the shearing induced by cone insertion in the relatively contractive 25% kaolin-75% sand mixture.

8.2.5 Penetration resistance at variable rate

As formulated in Section 8.2.4, Equation (8-5) can represent trends associated partial consolidation effects on cone resistance during the penetration in inviscid soils. The four fitting parameters ($a$ to $d$) in Equation (8-5) play a role in determining the shape of the $Q$ vs. $V$ curve, as illustrated in Figure 8.4a.

The $Q/Q_{UD}$ normalisation method (in a similar form as Equation (8-7)) has often been used in previous studies in clayey soils (e.g. House et al. 2001, Randolph & Hope 2004, DeJong & Randolph 2012), and is used because standard cone penetration (with a diameter of 35.7mm) at a velocity of 20 mm/s is expected to be undrained in clays. However, the selection of $Q_{UD}$ is hampered by the significant viscous effects that arise in clayey soils at these velocities.

In contrast, when the velocity is slow enough, penetration is fully drained and no excess pore pressures are present. Viscous effects are negligible in both clays and sands when penetration is drained (due to the lower viscous effects in sand and the low velocities when conditions are drained in clay). Therefore, penetration rate effects may be better
expressed by employing the drained resistance ($Q_D$) as the reference resistance i.e. as given by Equation (8-8), which has been used in Section 8.2.4.

Viscous effects may influence measured resistances in the partially drained region as well as the undrained region and are likely to vary with $v/d$ (Lehane et al. 2009); the coexistence and separation of viscosity and consolidation effects during penetration has not been studied well. Viscous effects are often referred to as strain rate effects, as shown, for example, by the increase in undrained strength with rate in soil laboratory element tests (e.g. Lunne & Andersen 2007). When viscous effects are taken into account, the penetration resistance may be written as the following form, which is similar to that proposed in Lehane et al. (2009).

$$Q = \left( a + \frac{b}{1 + cV^d} \right) \left[ \frac{V}{V_{ref}} \right]^m$$  \hspace{1cm} (8-9)

where $V_{ref}$ is defined as the reference normalised velocity (where the viscous effects are presumed to start to significantly influence the penetrometer resistance) and $m$ is the exponential coefficient of viscous effects. The strain along the advancing penetrometer is proportional to $v/d$ and a relative strain rate ($v/d)/(v/d)_{ref}$ can be expressed as $V/V_{ref}$ for the same penetrometer size and soil. Equation (8-9) predicts smaller resistances at $V<V_{ref}$ compared to Equation (8-5) and $Q_D$ is no longer equal to $a+b$, as illustrated in Figure 8.4b. It is presumed here that viscous effects are insignificant when $V<V_{ref}$ and thus the formulations are combined as follows (using the notation adopted above):

$$Q = \begin{cases} 
  a + \frac{b}{1 + cV^d} & \text{where } V < V_{ref} \\
  \left( a + \frac{b}{1 + cV^d} \right) \left[ \frac{V}{V_{ref}} \right]^m & \text{where } V > V_{ref}
\end{cases}$$ \hspace{1cm} (8-10)

While viscous effects can be varied over partially drained and undrained region depending on $V_{ref}$, $Q_D$ and $Q_{UD}$ can be obtained from Equation (8-10) as:

$$Q_D = a + b \hspace{0.5cm} , \hspace{0.5cm} Q_{UD} = a \left[ \frac{V}{V_{ref}} \right]^m$$ \hspace{1cm} (8-11)

The rate of increase of $Q_{UD}$ with velocity in the undrained region depends on $V_{ref}$ and $m$. For example, $m$ values of 0.05 and 0.08 with $V_{ref}=0.1$ (assumed) correspond to 12% and 21% increases in $Q$ per log cycle increase in $V$. 
8.3 **EVALUATION OF PENETRATION RATE EFFECTS**

This section examines available data related to penetration rate effects in:

- Normally consolidated reconstituted kaolin clay
- Overconsolidated reconstituted kaolin clay
- Normally consolidated to lightly overconsolidated intact clayey soils
- Normally consolidated reconstituted clayey sands
- Normally consolidated reconstituted silty soils
- Intact silty soils
- Very dense or highly overconsolidated soils

8.3.1 **Normally consolidated reconstituted kaolin clay**

As for the rate dependence of penetrometer resistance, normally consolidated (NC) kaolin clay has been investigated extensively in previous centrifuge studies (House et al. 2001, Randolph & Hope 2004, Schneider et al. 2007, and Lehanne et al. 2009) as well as at 1g in Chapter 6. It has already been shown in Section 7.5.2 that the PLAXIS numerical analyses are capable of predicting the rate dependency of penetration resistance observed experimentally in 1g tests in kaolin. The relevant soil parameters for NC kaolin are also summarised in Section 7.5.2. This section provides further assessment of penetrometer rate dependency using the previous studies in NC kaolin.

Figure 8.5a summarises the normalised resistances ($q_{uel}/\sigma_{v0}$ or $q_{T-bar}/\sigma_{v0}$) measured from the variable rate CPTU and T-bar penetration tests in NC kaolin at UWA. The normalised excess pore pressure ratios ($\Delta u/\sigma_{v0}$) obtained from the piezocone tests are also plotted in Figure 8.5b. The normalised velocity ($V_h$) was calculated according to Equation (8-4), for which $c_h$ values were obtained from dissipation test data for piezocone tests (Equation (8-2)) or using Equation (8-1) for T-bar data. The $c_h$ values (which vary with stress levels) reported in each study were within a small range of 3 to 12 m²/year with $c_h/c_{vNC}$ ratios of 2 to 6 ($c_{vNC}$ from 1-D Rowe cell consolidation tests).

Overall trends of rate dependency are comparable between the studies using the same probe shape. A small disparity in resistance measurements may be seen due to slight difference in sample properties, which is related to the preparation method for the samples (void ratios). $\Delta u/\sigma_{v0}$ changes in the piezocone tests suggest that the undrained
condition occurs at $V_h > 50$ and drained conditions are likely to occur at $V_h < 0.1$ (whereas $Q$ variations indicate $V_h < 0.03$ defines the drained region). It is evident that the partially drained region exists over about three orders of magnitude in $V_h$.

Transitions from undrained to partially drained conditions with the T-bar penetrometers tend to occur at lower $V_h \sim 20$ (using $d_c$; see Section 8.2.3), which is slightly lower than $V_h$ value of 50 with the piezocone penetrometer. This tendency is explained as a T-bar penetration is essentially a plain strain installation (whereas a piezocone penetration is axisymmetric) so that excess pore pressures tend to dissipate slower around the cylindrical bar than the spherical tip.
The PLAXIS cavity expansion (CE) numerical analyses using the Hardening Soil model in Chapter 7 (see Equations (7-12) and (7-15)) predict drained and undrained resistances ($Q_D$ and $Q_{UD}$ CE estimates) of 5.1-7.7 and 2.5-3.0 respectively for the soils with $E_{50}/p'_0$=30-60, $\phi'$=23-25° (which is reasonable for normally consolidated kaolin; see Section 7.5.2). The $q_{ud}/\sigma'_{vo}$ or $q_{t-bar}/\sigma'_{vo}$ measurements at the slowest rates in the experiments were not always fully drained but $Q_D$ values fall approximately into the range of the $Q_D$ CE estimates of 5.0 to 7.9. On the other hand, the $Q_{UD}$ CE estimates of 2.4 to 2.9 agree fairly well with the $Q_{UD}$ measurements in the two piezocone studies (Randolph & Hope (2004) and in the 1g tests; see Chapter 6) but $Q_{UD}$ values were 30% higher in the experiments of Schneider et al. (2007). The minimum $Q$ measurements recorded by the piezocones are about 50% higher than those of the T-bars. This is largely because of the dependence of the undrained bearing factor ($N$) on the penetrometers shape, as deduced theoretically by Randolph (2004) and others.

With a focus on the CPTUs in Figure 8.6, the rate dependency of $Q$ and $Q/Q_D$ variations can generally be captured by upper-bond (UB) and lower-bound (LB) hyperbolic curves with the form given in Equation (8-5). The fitting parameters are summarised in Table 8.3. The two parameters of $a$ and $b$ were determined from the $Q_D$ and $Q_{UD}$ CE estimates; see Equation (8-6). The $Q$ piezocone measurements from the three separate studies are within the UB and LB hyperbolic functions, except for the high $Q_{UD}$ values reported by Schneider et al. (2007) and $Q$ fluctuations at $V_h$$<0.1$ in the 1g tests, which may be less reliable. The $Q_{UD}/Q_D$ ratios between 0.4 and 0.5 are consistent with a $Q_{UD}/Q_D$ ratio of ~2.5 in kaolin clay suggested by DeJong & Randolph (2012).

Although there are insufficient data available at high velocities for the piezocone penetrometers, it can be inferred from the T-bar data on Figure 8.5a that viscous effects are significant. Undrained viscous effects were evident from the T-bar tests at $V_h$ values greater than 0.1 s$^{-1}$ (Lehane et al. 2009) which, for normally consolidated kaolin clay in the experiments under consideration, corresponds to a $V_h$ value of about 10. Presuming that viscous effects on piezocone $Q$ measurements in the undrained and also partially drained conditions, Equation (8-10) with the fitting parameters (listed in Table 8.3) is seen to provide a reasonable representation of the rate dependency. The formulation with $V_{ref}=0.1$ and $m=0.05$ (noting that $m=0.08$ was deduced by Lehane et al. (2009) for T-bar tests) predicts undrained viscous effects of approximately 11% at per log $V_h$ cycle between 100 and 10000.
Table 8.3 Fitting parameters used in Figure 8.6

<table>
<thead>
<tr>
<th>Equation</th>
<th>Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq(8-5) UB</td>
<td>(a = 2.91, b = 4.94, c = 1.2, d = 1)</td>
</tr>
<tr>
<td>Eq(8-5) LB</td>
<td>(a = 2.41, b = 2.64, c = 1.2, d = 1)</td>
</tr>
<tr>
<td>Eq(8-10)</td>
<td>(a = 1.8, b = 4.6, c = 1.2, d = 0.85)</td>
</tr>
</tbody>
</table>

Figure 8.6 Evaluation of penetration rate effects in normally consolidated kaolin on: (a) \(Q-V_h\) and (b) \(Q/Q_D-V_h\)
8.3.2 Overconsolidated reconstituted kaolin clay

In addition to normally consolidated kaolin clay, some results on variable rate penetration tests in lightly overconsolidated (LOC) kaolin (OCR<6.5) are available from the centrifuge studies (Schneider et al. 2007 for piezocone, Lehane et al. 2009 for T-bar). Using the normalised velocity \( V_h \) calculated with the equivalent diameter \( d_e \), the effects of drainage conditions on \( Q \) are compared on Figure 8.7 for piezocone and T-bar tests, employing ‘operational’ \( c_h \) values (rather than \( c_{vNC} \) values). \( c_h \) estimated in OC kaolin using the Teh & Houlsby (1991) method is about 20 times higher than the vertical coefficient of consolidation \( c_{vNC} \) at the same stress level (Schneider et al. 2007). Although an analytical framework is available to accommodate dilatory pore pressure dissipation in overconsolidated soils (e.g. Burns & Mayne 1998a), the centrifuge data show a monotonic reduction of excess pore pressure dissipation after undrained penetration at \( v=3 \) mm/s (Schneider et al. 2007).

As expected, and as also observed in the numerical analyses, \( Q \) at any penetration velocity increases strongly with increasing OCR (see Figure 8.7a). Higher \( \Delta u/\sigma'_{v0} \) ratios are obtained in OC kaolin but the velocity range of the partially drainage region is comparable to that in NC kaolin (see Figure 8.5b). Due to the limited available data under fully drained conditions, \( Q_D \) values were estimated to allow variations of \( Q/Q_D \) ratios to be examined in Figure 8.7b. The \( Q/Q_D \) curve at OCR=5 for the T-bar is clearly above that for OCR=1 and is such that \( Q_{UD}/Q_D \) reduces from about 0.4 at OCR=5 to about 0.25 at OCR=1; see Lehane et al. (2009). This tendency is consistent with insights obtained in the PLAXIS numerical analyses (see Section 7.4.14). It is noted that \( Q_{UD}/Q_D \) ratios are likely to be dependent on penetrometer shape as the \( Q_{UD}/Q_D \) ratios from the piezocone tests were about 0.4 at OCR=1; see Figure 8.6. However in contrary to T-bar, effect of OCR is not as obvious in the piezocone tests reported by Schneider et al. (2007). Further experimental tests in overconsolidated clays would clarify this trend.
8. Analysis and discussion

Figure 8.7 Penetrometer rate effects experimentally obtained in overconsolidated kaolin clay (OCR<6.5) on: (a) $Q/V_h$ and (b) $Q/Q_D-V_h$
8. Analysis and discussion

8.3.3 Normally consolidated to lightly overconsolidated intact clayey soils

8.3.3.1 Gingin, Bassendean and Burswood soils

The series of field scale variable rate penetration tests described in Chapter 4 are further evaluated in this section. The relevant soil properties at the three sites: Gingin, Bassendean and Burswood are summarised in Table 8.4. As seen on Figure 8.8 and also previously demonstrated on Figure 8.2, drainage conditions during penetrations are well represented using $V_h$ where $c_h$ values were obtained from piezocone dissipation tests at the corresponding depths. It is seen that $\Delta u/\sigma_0'$ ratios of between 2 and 2.5 at $V_h \sim 50$ (in undrained conditions) reduce to about zero at $V_h \sim 0.1$ while $Q$ increases as $V_h$ falls from 10 to 0.03. Transitions from undrained to partially drained and partially drained to fully drained conditions are hence estimated to be at $V_h$ of 30±20 and $V_h$ of 0.06±0.03 respectively i.e. partially drained penetrations prevail over about two to three orders of magnitude in $V_h$. These drainage transitions in $V_h$ are fairly similar to those found in reconstituted kaolin clay described in the previous sections. It is also noted that the $V_h$ values assessed in Table 8.1 are approximate as they correspond to estimates relating to 95% drained resistance and 95% drop from the drained resistance with no account of pore pressures or viscous effects.

The $Q$ variations are generally similar at Bassendean and Burswood. The comparison shows somewhat lower $Q$ values at Burswood in any drainage condition, which is consistent with this material’s higher clay content and slightly lower normalised stiffness (see Chapter 3). A $E_50/p'_0$ and $\phi'$ range of 50 to 150 and 30 to 35º respectively was used to estimate $Q$ values based on the cavity expansion (CE) numerical analyses (Chapter 7), and the $Q$ estimates are also presented in Figure 8.8a. The $Q_{UD}$ CE estimates of 9.2 to 19 fairly agree with the $Q_{UD}$ measurements in Bassendean and Burswood and the $Q_{UD}$ CE estimates of 3.3 to 4.5 are also similar to the minimum $Q_{UD}$ measurements of 4.1 to 5.3 in these materials. It should be noted that the relatively high peak friction angle ($\phi'_p$) with softening behaviour cannot be modelled in the Hardening Soil model. It is also observed from the laboratory tests that the undrained shear strength ratios ($s_u/\sigma'_0$) in Bassendean and Burswood intact soils (see Section 3.4.5 and Section 3.5.5) are higher than those of un-structured normally consolidated clays due to a light degree of overconsolidation (OCR~1.5) in addition to the important effects of structure. As is typical of structured soils, they both exhibited higher compressibility post-yield than
equivalent reconstituted soils. The relatively high peak effective friction angles were similarly reported for Burswood clay by Low et al. (2011) and also for Bothkennar clay (which is a similar deposit), as described by Hight et al. (1992) and Hight et al. (2003).

In Gingin, the measured $Q_{UD}$ values were almost three times those measured at the other two sites, even though similar $\Delta u/\sigma_{v0}$ ratios of between 2 and 2.5 were generated in undrained conditions. The pore pressure variation with cone velocity at Gingin is consistent with that seen at Bassendean and Burswood using the $c_h$ value of 63 m$^2$/year in the calculation of $V_h$. The relative independence of the $Q$ on the cone velocity at this site requires further research but is believed to be associated with the more dilative nature of this material.

Table 8.4 Summary of soil properties related to in situ penetration rate effects (I)

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil type</th>
<th>$z$</th>
<th>$\sigma'$</th>
<th>PI</th>
<th>OCR</th>
<th>$\phi_p$</th>
<th>$G_0$</th>
<th>$G_30$</th>
<th>$c_{NC}$</th>
<th>$c_v$</th>
<th>$c_h$</th>
<th>$t_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gingin, Australia</td>
<td>Mine tailings</td>
<td>1.1</td>
<td>18</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>0.63</td>
<td>-</td>
<td>63</td>
<td>8</td>
<td>Chapters 3&amp;4</td>
<td></td>
</tr>
<tr>
<td>Bassendean, Australia</td>
<td>Sandy clayey silt</td>
<td>3-8</td>
<td>34-54</td>
<td>21-33</td>
<td>&lt;1.6</td>
<td>39-44</td>
<td>8-14</td>
<td>B 1.5-2.4</td>
<td>0.4-1</td>
<td>39-79</td>
<td>63-252</td>
<td>2.8</td>
</tr>
<tr>
<td>Burswood-I, Australia</td>
<td>Silty clay</td>
<td>6-11</td>
<td>42-66</td>
<td>34-60</td>
<td>&lt;1.6</td>
<td>33-38</td>
<td>9-28</td>
<td>B 0.9-4.9</td>
<td>-</td>
<td>1.7-8.8</td>
<td>4-20</td>
<td>21-132</td>
</tr>
<tr>
<td>Burswood-II, Australia</td>
<td>Silty clay</td>
<td>6-10</td>
<td>35-53</td>
<td>~60</td>
<td>~1.5</td>
<td>34-53</td>
<td>5-9</td>
<td>S 0.7-2</td>
<td>0.6-1.5</td>
<td>5-19</td>
<td>8-19</td>
<td>~55</td>
</tr>
</tbody>
</table>

*B: $G_0$ is measured by the bender element in laboratory  
*S: $G_0$ is measured by the in situ seismic CPTU
8. Analysis and discussion

It can be seen in Figure 8.9 that viscous effects on the $Q$ values are evident in Bassendean and Burswood soils. $Q$ reductions of about 15% ($Q/Q_{20} \sim 0.85$), where $Q_{20}$ is the $Q$ value at the standard penetration rate of 20 mm/s, were recorded at velocities ($v$) of 10 to 20 times slower than the standard rate of 20 mm/s, for which $v/d$ are about 0.03-0.05. Below these $v/d$ values, partial consolidation takes place and $Q$ starts to increase. It is also seen on Figure 8.9b that that viscous effects are dominant at $V_h$ greater than about 50, where an increase in $Q$ is approximately 12% per log cycle increase in $V_h$. The piezocone penetration rate dependencies observed at Bassendean and Burswood are summarised in Figure 8.10 which also shows the fitting equations obtained using the hyperbolic formulations of Equations (8-5) and (8-10) using the parameters given in

![Figure 8.8 Comparisons of in situ rate effects data between Gingin, Bassendean and Burswood](image-url)
Table 8.5. The upper and lower bound curves obtained using Equation (8-10), which take the viscous effects into account, provide a more credible description than Equation (8-5) of the actual rate dependency in Figure 8.10.

![Figure 8.9 Viscous effects in Bassendean and Burswood soils](image1)

![Figure 8.10 Evaluation of penetration rate effects in Bassendean and Burswood sites](image2)

Table 8.5 Fitting parameters used in Figure 8.10

<table>
<thead>
<tr>
<th>Equation</th>
<th>Parameter</th>
<th>a</th>
<th>b</th>
<th>c</th>
<th>d</th>
<th>V_{ref}</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq(8-5) UB</td>
<td></td>
<td>4.0</td>
<td>12</td>
<td>3</td>
<td>0.85</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Eq(8-5) LB</td>
<td></td>
<td>3.2</td>
<td>5.7</td>
<td>3</td>
<td>0.85</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Eq(8-10) UB</td>
<td></td>
<td>3.7</td>
<td>10.5</td>
<td>3</td>
<td>0.85</td>
<td>0.1</td>
<td>0.05</td>
</tr>
<tr>
<td>Eq(8-10) LB</td>
<td></td>
<td>3.0</td>
<td>5.7</td>
<td>3</td>
<td>0.85</td>
<td>0.1</td>
<td>0.05</td>
</tr>
</tbody>
</table>
8.3.3.2 Response of full-flow penetrometers in Bassendean soil

T-bar and Ball penetration tests also experienced rate dependence on the net tip resistance in Bassendean (see Section 4.3). In the undrained region, lowest $Q$ values (about 85% of $Q$ at the standard rate) were measured between 2 mm/s and 20 mm/s. As seen on Figure 8.11a, where the strain rate is normalised as $(v/d)/(v/d)_{ref}$ with a reference velocity of 20 mm/s, $Q$ values start to increase with increasing velocity above $(v/d)/(v/d)_{ref} \sim 0.2$ due to viscous effects and increase below $(v/d)/(v/d)_{ref} \sim 0.2$ due to partial consolidation. The results obtained are generally consistent with undrained viscous effects using T-bar and Ball penetrometers assessed by the others, where a rate coefficient ($\mu$) is reported (using a semi-logarithmic function) to be such as 0.10 to 0.15 in Burswood clay (Low et al. 2008) and 0.12 to 0.14 in four soft clay sites (Yafrate & DeJong 2007).

Overall, the penetration rate dependency for the full-flow penetrometers over a velocity range of about four orders of magnitude is closely comparable to that observed using the piezocone at the same stress level (see Figure 8.11b). More specifically, the $Q$ values for T-bar penetrometers tend to be the lowest values among the three penetrometers for a given $V_h$ in any drainage conditions, which tendency is also observed in normally consolidated reconstituted kaolin clay; see Figure 8.5a.

Using the velocity normalisation $V_h$ with the effective diameter ($d_e$), very similar threshold $V_h$ value of about 30 for the three different probes can be observed between undrained and partially drained conditions. Considering both consolidation and viscous effects, it can be concluded that, as for the piezocone, Equation (8-10) provides a reasonable representation of the rate effect tendency on $Q$ for T-bar and Ball penetrometers (see Figure 8.11). The fitting parameters used here are summarised in Table 8.6. Further investigation at higher $V_h$ values will assist in determining $m$ value, but typical $m$ values of 0.05 to 0.07 (with $a$ and $V_{ref}$ listed in Table 8.6) result in 12% to 18% increase per log cycle increase in $V_h$. This $m$ range is consistent with the $m$ values of 0.06 and 0.08 measured for T-bar and ball penetrations in kaolin (Lehane et al. 2009).
Table 8.6 Fitting parameters used in Figure 8.11b

<table>
<thead>
<tr>
<th>Equation</th>
<th>Parameter</th>
<th>(a)</th>
<th>(b)</th>
<th>(c)</th>
<th>(d)</th>
<th>(V_{ref})</th>
<th>(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq(8-10) UB</td>
<td></td>
<td>3.7</td>
<td>10</td>
<td>2</td>
<td>0.85</td>
<td>0.1</td>
<td>0.05</td>
</tr>
<tr>
<td>Eq(8-10) LB</td>
<td></td>
<td>2.5</td>
<td>7.5</td>
<td>2</td>
<td>0.85</td>
<td>0.1</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Figure 8.11 Penetration rate effects on T-bar and Ball penetration results at 3-5 m depth in Bassendean on: (a) \(Q/Q_{20}(v/d)/(v/d)_{ref}\) and (b) \(Q-V_h\)

\(Q=q_{net}/\sigma'v_0\)

\(V_h=v_{de}/c_h\)

\(\Delta T\text{-bar, 40mm x 250mm}\)

\(\square \text{Ball, d=75.35mm}\)
8. Analysis and discussion

8.3.3.3 Soils from extended database

From the database reviewed in Chapter 2, the following four studies are selected for further discussion. These studies investigated field scale penetrometer rate effects in lightly overconsolidated clay, silty clay and clayey silt sites.

- Indiana (Kim 2005)
- Onsoy and Drammen (Lacasse & Lunne 1982)
- McDonald Farm (Campanella et al. 1982)
- Saint-Alban (Roy et al. 1982a)

The relevant soil parameters for each site are summarised in Table 8.7.

Some of the \( Q \) data were available directly from the literature (Campanella et al. 1982, Kim 2005, Kim et al. 2008) but the original reported data were digitised and averaged over the depths focused for the study of rate effects from the others (Lacasse & Lunne 1982, Roy et al. 1982a). Soil properties in these sites from the relevant references are summarised in Table 8.7. \( c_h \) values were estimated from dissipation tests based on the Teh & Houlsby (1991) method (assuming \( I_r \) of 100–150 indicated in the table) apart from at the Indiana sites where, in the absence of dissipation test data, \( c_h \) was taken as \( 2c_v \), where \( c_v \) values were measured in oedometer tests at the in-situ stress (Kim 2005).

While a limited range of variable rate penetration tests were carried out at each testing site, it can be seen on Figure 8.12 that the penetration resistances increase due to partial consolidation when \( V_h \) is reduced to about 10. However, this transition between undrained and partially drained condition varies at between \( V_h \) values of 5 and 50 potentially due to inconsistencies in the selection of \( c_h \). In contrast, undrained viscous effects are evident at \( V_h > 100 \) in most cases.

The trends on Figure 8.12 can be generally captured by the hyperbolic functions according to Equation (8-5) with the parameters listed in Table 8.8. The \( a \) and \( b \) parameters were derived from the PLAXIS CE analyses (\( \psi = 0 \)) based on the estimated \( E_{50}/p'_o \) range of 50-150 and \( \phi' \) range of 30-35°; the selected \( c \) and \( d \) parameters require further investigation. Taking the viscous effects into account, Equation (8-10) with the fitting parameters listed in Table 8.8 provides a more representative trend of cone penetrometer rate effects in these clayey sites.
The Indiana-SR49 site shows $Q_{UD}$ values that are twice to three times the $Q_{UD}$ values at the Indiana-SR18 site (Kim 2005, Kim et al. 2008). Kim (2005) did not report the overconsolidation ratio for Indiana-SR49 site but much higher $\Delta\sigma_{v0}'$ may imply a higher degree of OCR, such as observed for kaolin clay with OCR=5 on Figure 8.5b. The higher $Q$ values in Saint-Alban site (Roy et al. 1982a) are also possibly because of a slightly higher overconsolidation ratio (OCR=2.1-2.3).

Table 8.7 Summary of soil properties related to in situ penetration rate effects (II)

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil type</th>
<th>z (m)</th>
<th>$\sigma_{vo}$ (kPa)</th>
<th>PI (%)</th>
<th>OCR</th>
<th>$\phi$ (°)</th>
<th>$G_{0}$ (MPa)</th>
<th>$c_v$ (m³/yr)</th>
<th>$c_h$ (m³/yr)</th>
<th>$t_{50}$ (min)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Indiana-SR49, USA</td>
<td>Silty clay, clayey silt</td>
<td>13-14</td>
<td>~135</td>
<td>9</td>
<td>-</td>
<td>-</td>
<td>11</td>
<td>[23]*</td>
<td>-</td>
<td>-</td>
<td>Kim et al. (2008)</td>
</tr>
<tr>
<td>McDonald Farm, Canada</td>
<td>Soft clayey silt</td>
<td>20</td>
<td>~130</td>
<td>15</td>
<td>~1.1</td>
<td>35</td>
<td>-</td>
<td>57-173</td>
<td>2.1-240</td>
<td>7.8</td>
<td>Campanella et al. (1982), Campanella et al. (1983), Robertson et al. (1992), Burns &amp; Mayne (1998a)</td>
</tr>
</tbody>
</table>

*assumed

Table 8.8 Fitting parameters used in Figure 8.12a

<table>
<thead>
<tr>
<th>Equation</th>
<th>Parameter</th>
<th>Parameter</th>
<th>Parameter</th>
<th>Parameter</th>
<th>Parameter</th>
<th>V_ref</th>
<th>m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq(8-5) UB</td>
<td>a=4.5</td>
<td>b=14.5</td>
<td>c=3</td>
<td>d=0.85</td>
<td>V_ref= -</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Eq(8-5) LB</td>
<td>a=3.3</td>
<td>b=6</td>
<td>c=3</td>
<td>d=0.85</td>
<td>V_ref= -</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Eq(8-10)</td>
<td>a=2.8</td>
<td>b=11</td>
<td>c=3</td>
<td>d=1</td>
<td>V_ref= 0.1</td>
<td>m=0.05</td>
<td></td>
</tr>
</tbody>
</table>
8. Analysis and discussion

Figure 8.12 Evaluation of penetration rate effects in intact clays to silty clays
8. Analysis and discussion

8.3.4 Normally consolidated reconstituted clayey sands

Figure 8.13 presents variable rate penetration test results obtained on clayey sands (as well as kaolin clay for comparison purposes). The majority of these results are presented in Chapters 5 and 6; published test results from other studies in these materials are incorporated in Figure 8.13. In addition, $Q_D$ and $Q_{UD}$ CE estimates (for $\psi=0$ material) based on the PLAXIS cavity expansion numerical analyses (for $\psi=0$ material) are presented to assist interpretation. Comparison of the PLAXIS numerical analyses and the experimental results in the 25% kaolin mixture at 1g has been presented in Section 7.5.3. This section summarises on the assessment of penetrometer rate dependency for reconstituted clayey sand samples from Chapter 5, Chapter 6 and previous studies.

It has been shown that the 25% kaolin mixture (with the host sand comprising 75% of the dry weight) has the higher $Q_D$ but the lower $Q_{UD}$ than measured in kaolin clay. Using a $E_S/p'_0$ range of 120 to 240 and $\psi$ value between 28 and 30° (for the soil parameters for the 25% kaolin mixture; see Section 7.5.3), the PLAXIS CE analyses (with $\psi=0$) predict $Q_D$ of 12.7 to 19.3, which are comparable with the $Q_D$ measurements of between 12 and 15. In contrast, the $Q_{UD}$ CE analyses (with $\psi=0$) using the same soil parameters predict $Q_{UD}$ values that are about twice those measured in experiment. The strongly contractive behaviour of this material during undrained shearing (even more contractive than kaolin clay) was not modelled in the original cases in the PLAXIS analyses; some insights into the effect of contractive behaviour are presented in Section 7.4.15.

Figure 8.13 (and Figure 8.3) suggests that there is a slightly narrower range in velocities over which partially drained conditions exist for the 25% kaolin mixture when compared to that in kaolin clay. This trend is partly associated with the contractive behaviour of the 25% kaolin mixture during undrained shearing, leading to a big $Q_D/Q_{UD}$ ratio and also possibly to a higher $d$ parameter in Equation (8-5), as illustrated in Figure 8.4a. A similar response has been observed by Schneider et al. (2007) in lightly overconsolidated silica flour, although these tests showed considerable scatter and poor repeatability.

As already discussed in Section 5.4.7, based on $V_v$ normalisation (i.e. using $c_v$ due to unavailability measured $c_h$ data) on Figure 8.11, it has been inferred that a standard penetration (with the standard cone diameter of 35.7mm and the standard penetration
rate of 20 mm/s) is potentially partially drained in soils with about 60 times greater \( c_v \) than that in the 25% kaolin mixture (corresponding to \( c_v \) of about 1800 m\(^2\)/year). Also, the centrifuge data indicate that partially drained conditions would arise in the 5% and 10% kaolin mixtures in a standard CPT but it is unfortunate that this response could not be recorded in the experiments due to the high permeable characteristics relative to the achievable velocities for the smaller diameter cone used in the centrifuge tests: averaged \( k_v \) values of \( 4.9 \times 10^{-6} \) and \( 8.4 \times 10^{-7} \) m/s were obtained in the 5% and 10% kaolin mixtures respectively (see Section 3.6.3).

Figure 8.13 Evaluation of penetration rate effects in reconstituted clayey sands

8.3.5 Normally consolidated reconstituted silty soils

Several studies have presented results relating to the penetration rate dependency of cone resistance in reconstituted silty soils. The data recorded in three such studies, all of which employed a centrifuge, are shown in Figure 8.14. Finnie & Randolph (1994) pushed a model foundation into calcareous silt and reported a constant volume \( \phi'_{cv}=38^\circ \) and a peak \( \phi'_p=42^\circ \) for this material. More recently Cassidy (2012) conducted T-bar tests in a similar calcareous silt. Oliveira et al. (2011) performed miniature CPT tests in silty tailings at relatively shallow penetrations.

There are large variations in \( V_v \) values between the three silty soils studied. Finnie & Randolph (1994) reported the \( c_v \) value of 50 mm\(^2\)/s (=1577 m\(^2\)/year), while Cassidy (2012) measured \( c_v \) of 15 to 82 m\(^2\)/year from Rowe cell consolidation test with silicon
oil (high viscosity) also being used in his centrifuge test. There is certainly a big difference in \( c_v \) between two studies even in similar calcareous silts. Oliveira et al. (2011) measured a minimum resistance at \( V_v \) between 50 and 75 with the adopted \( c_v \) value of \( 1.4 \times 10^{-6} \) m\(^2\)/s (=44 m\(^2\)/year). In their paper, permeability (\( k \)) values of 5 to \( 8 \times 10^{-6} \) m/s and a compression ratio \( C_c/(1+e_0) \) of 0.05 are also provided, but the \( c_v \) value adopted is not consistent with these parameters. It is suggested that the disparity in \( V_v \) drainage transitions evident on Figure 8.14 is likely not to be representative of operative conditions in these tests.

Interestingly, it is seen on Figure 8.14 that the drained \( Q \) values (at \( V_v \leq 0.1 \)) in these three soils are relatively similar. These \( Q_D \) values are equivalent to those predicted in the CE analyses (with \( \psi=0 \)) using soil properties of \( E_{50}/p'_0 \) of 300-400 and \( \phi' \) of 35-40\(^\circ\). Using these properties in the CE analyses leads to a prediction for \( Q_{UD} \) that is significantly higher than measured in the calcareous silt. This may be partly because of (i) contractant behaviour in reconstituted samples (although intact soils in these calcareous materials tend to dilate) and (ii) centrifuge modelling leading to lower than expected undrained strengths in the absence of an ageing effect. The calcareous silts consequently have a much larger \( Q_D/Q_{UD} \) ratio than shown by many of the material types discussed in previous sections above. Figure 8.14 highlights needs for further investigation on operative \( c_h \) and also understanding of soil behaviours in such silt soils.

![Figure 8.14 Evaluation of penetration rate effects in reconstituted silty soils](image-url)
8.3.6 Intact silty soils

Several in situ experimental studies, as reviewed in Chapter 2, have been carried out using piezocones with \( v \) greater than the standard rate of 20 mm/s in silty soils. These studies include the following testing sites:

- Opelika (Finke et al. 2001)
- Brazil tailings facilities (Schnaid et al. 2010, DeJong & Randolph 2012)
- Dronninglund (Poulsen et al. 2013)
- Emilia-Romagna (García Martínez et al. 2014).

Information related to the test sites is provided in Table 8.9. It is noted that the respective authors’ averaging of the data are used here, which means that original \( Q \) and \( \Delta u/\sigma'_{v0} \) values are not provided. Nevertheless the trends observed with the penetration velocity should remain.

According to the standard test results at \( v=20 \) mm/s (10 and 40 mm/s in Emilia-Romagna site) from the four sites, data are plotted on the well-known soil classification charts after Robertson (1990) and Schneider et al. (2008) in Figure 8.15. The normalised resistance, \( Q_{in} \), is used here to plot in \( Q_{t1}-F_t \) and \( Q_{t1}-\Delta u/\sigma'_{v0} \) space. The empirical soil classification charts suggest that all four soils are classified as transitional soils in the Schneider’s chart. The soil behaviour type index (\( I_c \)) based on the soil behaviour type (SBTn) (Robertson 2009) indicates that Opelika soil is classified as a silt mixture, Dronninglud is classified as a sand mixture, and Emilia-Romagna is classified as sand. The original piezocone profiles indicate appreciable fluctuations of \( q_t \) profiles due to non-homogeneity/layering especially at the Dronninglund site. The reported \( t_{50} \) values are very small (less than about one minute), which is consistent with the nature of these materials and with the soil classification chart (Robertson 2010). No \( t_{50} \) value was reported at the Dronninglund site; presumably \( t_{50} \) would be very small because of the very sandy nature of this material.
Table 8.9 Summary of soil properties related to in situ penetration rate effects (III)

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil type</th>
<th>z (m)</th>
<th>$\sigma'_v$ (kPa)</th>
<th>PI</th>
<th>Comments</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Opelika, US</td>
<td>Residual sandy silt to silty sand</td>
<td>4.8</td>
<td>62.95</td>
<td>5-12</td>
<td>- $d_s$=44 mm</td>
<td>Finke et al. (2001)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- negative pore pressures at $u_1$</td>
<td>Schneider et al. (1999)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- $t_u$=5-20 sec, $k_h$=1x10^3 m/s, $c_h$=1892 m^2/yr</td>
<td>Mayne et al. (2000)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- apparent OCR $\approx$2.4, $\phi'$=35º</td>
<td>Mayne &amp; Brown (2003)</td>
</tr>
<tr>
<td>Mine tailings, Brazil</td>
<td>Mine tailings</td>
<td>13.2</td>
<td>123</td>
<td></td>
<td>- $t_u$=14 sec, $c_h$=1734 m/year ($I_c=100$)</td>
<td>Schnaid et al. (2010)</td>
</tr>
<tr>
<td>Dronninglund, Denmark</td>
<td>Sandy dilative silt with clay stripes</td>
<td>4.5-114</td>
<td>41-165</td>
<td>0-27</td>
<td>- strong fluctuation in $q$, smoothed every 50 cm</td>
<td>DeJong &amp; Randolph (2012)</td>
</tr>
<tr>
<td>Emilia-Romagna, Italy</td>
<td>Silty sand to sandy silt</td>
<td>5.3-6.6</td>
<td>78-817</td>
<td></td>
<td>- fluctuation in $q$, data averaged over 5.3-6.6 m depth</td>
<td>Poulsen et al. (2013)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- $t_u$=32, 57, 74 sec, $c_h$=1514, 851, 694 m^2/yr ($I_c=380$)</td>
<td>Poulsen et al. (2012b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- $v$=40, 80, 130 mm/s respectively</td>
<td>Poulsen et al. (2012a)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>- $c_h$=978 m^2/yr, re-evaluated for partially drained</td>
<td>García Martínez et al. (2014)</td>
</tr>
</tbody>
</table>

Figure 8.15 CPT data using $Q_{res}$ at $v=20$ mm/s (10&40 mm/s in Emilia-Romagna) plotted in (a) $Q_{res}-F_r$ and (b) $Q_{res}-\Delta u/\sigma'_{vo}$ space (after Robertson 1990 and Schneider et al. 2008)

Figure 8.16 compares $Q$ resistances at various penetration rates at these four sites. It is evident that the $Q$ resistance at rates faster than $v=20$ mm/s are generally less than the $Q$ values at $v=20$ mm/s, which suggests that penetration at the standard rate is partially drained. The measured pore pressures are consistent with this inference, although the change from negative to positive pore pressure between $v$ of 20 and 200 mm/s at Opelika is unexpected (Finke et al. 2001).

In these materials, the evaluation of $c_h$ value from piezocone dissipation test at $v=20$ mm/s is unlikely to be adequate although some attempts at assessing this value are seen in the original papers as summarised in Table 8.9. This difficulty is primary due to partially drained conditions encountered during dissipation tests and highly...
heterogeneous stratified soil layers (with a high lateral permeability), as discussed in Section 8.2.2. In order to apply currently proposed interpretation methodology (for drained or undrained conditions), these uncertainties can be avoided by employing an additional cone penetration test at variable rate. The CPT should essentially be aiming for fully drained or fully undrained conditions; this issue is discussed in more detail in Section 8.4.2.

Figure 8.16 Data assembled on penetration rate effects in intact silts
8.3.7  Very dense or highly overconsolidated soils

Silva & Bolton (2005) and Silva (2005) present data from variable rate CPTs in dense dilatant silts tested in the centrifuge and found that the drained penetration resistance was lower than the undrained penetration resistance. The silty samples had a relatively low permeability ($k$) of about $6 \times 10^{-8}$ m/s with $c_v$ of 50 to 100 mm$^2$/s (=1577 to 3153 m$^2$/year). Several data points at the three different depths were extracted from the original reported data and replotted on $Q$-$V_v$ (using $c_v$) in Figure 8.17. Silva (2005) measured $Q$ values in a dense (vibro-compacted) sample of about 310±20 at $v$=0.05 mm/s and $Q$ values of between 600 and 1300 (at 160 mm and 40mm depth respectively) at $v$=4.0 mm/s. In this strong dilative soil with low permeability, penetration rate effects are affected by (i) a strong dilative behaviour (and associated high peak friction angle) especially at the lower stress levels (ii) high OCR and $K_0$ values induced by vibro-compaction and (iii) potential viscous effects. The friction sleeve data recorded by Silva (2005) at the fastest speeds coupled with the corresponding $q_c$ data imply that the soils would be classified as a very dense/ stiff soil (SBT=8) in a standard CPT. Thus, it can be inferred that such soils may exhibit $Q_D < Q_{UD}$.

![Figure 8.17 Rate dependence in dilatant silts (after Silva 2005)](image)

Some insights into the effects of dilatancy in these kinds of soils are provided Danziger & Lunne (2012) as reviewed in Chapter 2. Numerical investigations of these effects are suggested by LeBlanc & Randolph (2008) and Jaeger (2012), and are also examined in this thesis by employing a positive dilation angle in the PLAXIS cavity expansion.
analyses in Section 7.4.15; these analyses did not permit conclusions of a quantitative nature to be drawn due to the numerical analysis limitations.

The rate dependency observed in strongly dilative soils may be captured by using a negative $b$ value in the hyperbolic function of Equation (8-10); further investigation is required to allow adequate evaluation of combination effects due to consolidation and strain rate. Variable rate penetration testing is recommended to assess soil parameters from a cone penetration test in this type of soil. A slow test aimed at fully drained penetration is considered more useful for derivation of soil parameters because effects of dilatancy appear to have a more significant effect on penetrometer end resistance in the undrained and partially drained conditions.

Very stiff clays also have the potential to have higher undrained than drained resistance. Powell & Quarterman (1988) reported the in situ rate effects in relatively high OCR clays (UK clays). The penetration results at the four sites (Cowden, Brent Cross, Canons Park and Cambridge) are replotted on $Q$ against testing depths in Figure 8.18 after the original data reported; see Section 2.2.1. Some related soil properties are summarised in Table 8.10. At these four sites, negative pore pressures ($u_2$ position) were often observed and unreliable due to cavitation effects (Lunne et al. 1986). Corrections to obtain $q_t$ from $q_c$ are, however, relatively insignificant in these very stiff materials (Powell & Quarterman 1988).

\[
Q = \frac{q_c - \sigma' v_0}{\sigma' v_0}
\]

Figure 8.18 Rate effects in HOC UK clays (after Powell & Quarterman 1988)
It can be observed that, for a given velocity, $Q$ values decrease with depth i.e. due to the reducing OCR with depth at these sites (see Figure 8.18 and Table 8.10). The resistances at $v=0.1$ m/min are about 10 to 15% less than those at $v=1$ m/min. At slower penetrations with $v=0.01$ m/min resistances are lower than at 0.1 mm/min at Brent Cross but higher in the Gault clay at Cambridge. Data from shallow depths (i.e. the highest OCR) were further extracted and replotted in Figure 8.19a. Figure 8.18b plots $Q$ against the normalised velocity $V_h$, using approximate $c_h$ values deduced from the literature. The trends on Figure 8.18b suggest that penetrations are likely to be mostly undrained (with $V_h$ in excess of 30) but may be partially drained region at the lowest velocity at Cambridge. The observed increases of $Q$ with $V_h$ are consistent with typical viscous effects but may well also include additional resistance due to dilative effects. It is not possible to comment on expected $Q_D$ values in these sites but there is certainly the potential for higher $Q_{UD}$ than $Q_D$ due to very high OCR coupled with viscous effects. In these soils, it is also possible that resistance in the partially drained range is the minimum resistance for all drainage conditions (i.e. less than both $Q_D$ and $Q_{UD}$); this trend has not been observed experimentally but has been raised as being a possibility by LeBlanc & Randolph (2008). Further experimental research and advanced soil models in numerical analysis are required.

<table>
<thead>
<tr>
<th>Site</th>
<th>Soil type</th>
<th>$z$ (m)</th>
<th>$\sigma_{v0}$ (kPa)</th>
<th>PI (%)</th>
<th>OCR</th>
<th>$\phi_p$</th>
<th>$G_o$ (MPa)</th>
<th>Comments</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cowden, UK</td>
<td>Stiff glacial</td>
<td>4-10</td>
<td>55-120</td>
<td>17-20</td>
<td>5-10</td>
<td>27</td>
<td>50</td>
<td>150</td>
<td>Powell &amp; Quarterman (1988) | Lunne et al. (1986) | Robertson et al. (1992) | Lehane (1992) | Powell &amp; Butcher (2003)</td>
</tr>
</tbody>
</table>

8. Analysis and discussion
8. Analysis and discussion

8.4 COMMENTS ON VARIABLE RATE PENETRATION

8.4.1 Expected penetration rate effects

The observations of rate effects on penetrometer resistance in a large range of case studies including field observations and laboratory physical model tests has clearly shown that penetration rate effects are strongly soil type dependent. The range of expected behaviour as a function of soil type is illustrated in Figure 8.20. This figure plots trends of the ratio of normalised tip resistance to drained resistance ($Q/Q_D$) against a normalised velocity ($V_h = vd/c_h$). Figure 8.21 shows an image of the expected penetration rate effects on the soil classification chart proposed by Robertson (1990). Explanations of these figures are summarised as follows:

**Soil A vs. Soil B**

Rapid penetration into a normally consolidated or loose soil (i.e. positive state parameter) will have a reduced penetration resistance ($Q$) due to the generation of positive excess pore pressure. The PLAXIS numerical analyses showed that both drained and undrained resistances ($Q_D$ and $Q_{UD}$) are affected strongly by the stiffness of the soil mass surrounding the cone tip and, to a lesser extent, by the soil’s friction angle. $Q_D$ is more dependent on these factors, and hence $Q_{UD}/Q_D$ can be expected to be lower in loose sand than in typical normally consolidated clay.
Soil A vs. Soil C
The higher viscosity of clayey soils leads to increased $Q$ values. The effect of viscosity is larger at larger penetrometer velocities and hence its effect is more obvious in the undrained region for these materials; viscous effects in sand are relatively small (Lehane et al. 2009). The importance of viscosity in the undrained condition in clayey soils leads to variations in $Q_{UD}$ and higher ranges in apparent $N_k$ values. Consequently it is preferable to use $Q_D$ rather than $Q_{UD}$ to normalise data.

Soil A vs. Soil D
Some soils (e.g. the 25% kaolin mixture) have a structure which is heavily contractant (high state parameter). Such soils consequently exhibit low $Q_{UD}$ values despite the fact that they have higher stiffness and higher friction angle. $Q_{UD}/Q_D$ ratios in Soil D are therefore lower than those for Soil A.

Soil A vs. Soil E and Soil F
There is a tendency for $Q_{UD}/Q_D$ to increase with increasing overconsolidation ratio (OCR) because of the slightly larger influence of horizontal effective stress on $Q_{UD}$ (see Section 7.4.13) and also because of an increase in dilation angle with OCR. There is a potential to obtain $Q_{UD}/Q_D>1$ in very heavily overconsolidated soils (Soil F) but no published field data has demonstrated this yet (given the shortage of $Q_D$ data in these materials). A very high OCR would be required to bring the $Q_{UD}/Q_D$ above unity. When viscous effects are dominant in the undrained condition, a greater $Q_{UD}/Q_D$ occurs and it is possible for a partially drained resistance ($Q_{PD}$) to be the minimum resistance.

Soil B vs. Soil G and Soil H
Penetration into medium and dense soils would result in higher $Q_{UD}/Q_D$ than that in loose soils (Soil B) due to the strong influence of dilatancy during undrained penetration. However, undrained penetration in very dense sands is less likely even for large penetrometers (such as piles) due to their high permeability.
8. Analysis and discussion

Figure 8.20 Illustration of potential penetration rate dependency on $Q/Q_D$ in varying soils

Figure 8.21 Evaluation of penetration rate effects on soil classification chart at the standard penetration rate $v=20$ mm/s (after Robertson 1990)
8.4.2 Recommendations for practice

In this section, recommendations on the use of variable rate penetration tests for practice are made to increase the value that a piezocone test can bring to a site investigation and to the interpretation of geotechnical parameters. The current semi-empirical correlations to interpret geotechnical properties are essentially for sand or clay (i.e. for which fully drained or fully undrained conditions expected). Interpretation methods for partially drained conditions in intermediate soils are virtually non-existent. It has been presented in the Thesis that for any piezocone test, it is firstly very important to identify the drainage condition. Once identified, piezocone penetration can then be performed at a rate other than standard rate to allow improved assessment of geotechnical properties.

The specific drainage condition pertaining during penetration is best assessed using a dissipation test; assessment can also be assisted with reference to the soil behaviour type charts (e.g. Robertson 1990, Schneider et al. 2008). The transition from undrained to partially drained penetration has been seen to occur at $V_h = 30\pm20$ in clay, clayey silts and silty clays; see Section 8.3. Following Equation (8-4) for a standard piezocone ($d=35.7$ mm) and the standard penetration rate ($v=20$ mm/s), a $V_h$ value 30 corresponds to a $c_h$ value of $23.8 \text{ mm}^2/\text{s} (=750 \text{ m}^2/\text{year})$. This $c_h$ value is consistent with $t_{50}$ values measured in a dissipation test (where $t_{50}$ is the time for 50% dissipation) of 50±25 seconds for $I_r$ in the range to 50 to 500. It is therefore inferred that partial drainage during penetration at the standard rate may be concerned when $t_{50}$ is less than about 75 seconds. When $t_{50}$ is greater than 75 seconds, penetration is very likely to be fully undrained. This is consistent with Robertson (2010), who suggests that the tip resistance is essentially equal to the undrained value if $t_{50}$ is longer than 30 seconds.

Figure 8.22 shows expected drainage conditions at the standard penetration rate plotted on a permeability ($k_h$) relationship with $t_{50}$ developed by Parez & Fauriel (1988). An approximate estimate of permeability ($k$) based on the CPT-based SBT chart of Robertson (2010) is also shown, where the numbers represent soil behaviour type number (SBT$n$); see Figure 8.21. The $t_{50}$ value of 75 seconds is located in the ‘silt zone’ defined by Parez & Fauriel (1988) for which $k_h$ is in the range of $10^{-8}$ to $10^{-7}$ m/s; this region is clearly one prone to the risk of partial drainage. On the other hand, while fully drained conditions are defined at $t_{50}=0$ (as there is no excess pore pressures generated during penetration), a $t_{50}$ value of 1 second can be assumed to correspond to drained
conditions. Thus, partially drained penetration for a standard piezocone is likely to occur in the $k_h$ range of about $5 \times 10^{-8}$ m/s to $1 \times 10^{-4}$ m/s. Further research is required to assess the operational $c_h$ value in very dense/ stiff soils (SBTn zone of 8 and 9) which can generate negative pore pressures during penetration.

Some examples where partially drained penetration was likely to have occurred during the standard CPT (details provided Section 8.3) are listed in Figure 8.22. Although some data on $t_{50}$, $k_h$, and SBTn are not complete for these soils, the available evidence supports the contention that conditions may be partially drained for $t_{50}$ values less than 75 seconds.

![](https://example.com/figure8_22.png)

**Figure 8.22** Drainage conditions on permeability ($k_h$) and $t_{50}$ correction after Parez & Fauriel (1988) and $k$ estimation based on SBTn by Robertson (2010)

Different sequences of variable rate penetration test are proposed dependent on the assessed drainage condition. Figure 8.23 illustrates the expected performance of a variable rate cone penetration test in SBTn of 2 to 6. For the case when $t_{50}$ is greater than 75 seconds (case1 in Figure 8.23), the soils are likely to be clay or silty clay. The influence of viscous/strain rate effects on the inferred undrained strength may be of concern and one suggestion would be to conduct the velocity sequence 20, 40, 20, 4, 2, and 0.4 mm/s over 150 mm for each velocity with the standard cone size $d=35.7$ mm. This sequence can be conducted over a 1 m cone rod interval and takes less than 10 minutes to complete. The test sequence allows a lower bound estimation of undrained strength in clayey soils.
For the case when $t_{50}$ is less 25 seconds (case3), a partially drained penetration test can be avoided by performing slower penetrations; this is preferable than adopting very fast penetrations due to potential strong dilation (and also viscous) effects in these kinds of intermediate soils. Sandy silt to silty sand (sand mixture to silt mixture) with $t_{50} < 25$ seconds often show some vertical variability and hence penetration should be for as long a distance as economically feasible. A velocity of 0.2 mm/s (i.e. 100 times slower than the standard rate) is expected to lead to essentially fully drained conditions and allows the designers to apply ‘drained correlations’ in the derivation of empirical parameters (with potentially some conservatism). The additional time required to conduct a 1m cone penetration is about 80 mins, which is not considered excessive and is broadly in keeping with typical times allocated to two piezocone dissipation tests.

For the case when $t_{50}$ values are between 25 and 75 seconds (case2), the undrained and drained geotechnical responses are likely to be of interest in design. For this case, reducing the standard velocity a hundredfold to 0.2 mm/s is expected to lead to give between about 80% and 100% of the drained resistance. Increasing the velocity to 100 mm/s is also recommended to obtain a minimum undrained resistance and avoid overestimation of undrained shear strength due to partially drainage.

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**Figure 8.23** Illustration of expected performance of recommended variable rate cone penetration tests on a typical cone resistance rate dependency.
CHAPTER 9  CONCLUSIONS

9.1  INTRODUCTION

This Thesis has presented an investigation into the rate dependency of CPT cone penetration by performing (i) field tests, (ii) physical experiments in the centrifuge and at 1g and (iii) supporting numerical simulations and laboratory soil element tests. Experimental results obtained in this study are combined with a database comprising results from other published experimental data, and these are then evaluated with the assistance of the trends interpreted from the numerical simulations. This final Chapter summarises the key conclusions from each element of the research and provides recommendations for future work in this area.

9.2  PRINCIPAL FINDINGS OF RESEARCH

(i)  Field test observations

- The rate dependence of cone penetrometer end resistance \( q_t \) and \( u_2 \) pore pressures (measured with the standard 10cm\(^2\) cone, a diameter \( d = 35.7 \) mm) was examined in a comprehensive series of tests performed at Gingin, Bassendean and Burswood. Changes in piezocone resistance over a velocity range extending over five orders of magnitude were observed at the Bassendean site (see Figure 9.1).

- The CPTU parameters in these soils were found to be highly dependent on the penetration velocity especially at velocities that are 10 to 1000 times slower than the standard penetration rate \( (v) \) of 20 mm/s. More specifically, the observed dependence on \( v \) of \( q_t \) and \( u_2 \) varies with the soil type and the stress level (or depth).

- Resistance increases are accompanied by a reduction in excess pore pressure as the penetration rate reduces from an undrained condition to a drained condition. Such partially drained conditions extend over two to three orders of magnitude of penetrometer velocity.
9. Conclusions

- Drained normalised penetration resistances ($Q_D = q_{net}/\sigma'_v$) were achieved at the extremely slow penetration tests at 0.002 to 0.0002 mm/s in Bassendean and Burswood (note that 0.0002 mm/s is $10^5$ times slower than the standard rate). The ratios of drained to undrained resistance ($Q_D/Q_{UD}$) observed were between 1.6 and 2.3. These ratios are lower than those observed in previous experimental studies, the majority of which were performed in geotechnical centrifuges.

- Full-flow devices such as the T-bar and ball show a similar rate dependency to shown by the piezocone. However the resistance of the cone is higher than that of the T-bar or ball due to their different geometries.

![Figure 9.1 Rate dependence on piezocone normalised resistance at the Bassendean site; see Section 4.3.4](image)

(ii) Physical experiments: Centrifuge & Pressure Chamber

- The variable rate CPT experiments confirm that the fines content (5%, 10% and 25% kaolin) of kaolin-sand mixtures significantly influences the variation of cone resistance with cone velocity. Typical results, shown on Figure 9.2, are consistent with available data from previous investigations into the rate dependency of penetrometer resistance of clayey sands.

- The transitions from drained to partially drained and partially drained to undrained conditions is shown to occur at the same normalised velocity for the 25% kaolin and 100% kaolin soil mixtures. Undrained conditions were encountered at normalised velocities, $V_v = v/d/c_v$ greater than $\sim10$ and drained conditions operated at $V_v$ values less than $\sim0.06$. 


In keeping with the greater stiffness and higher friction angles for mixtures with lower fines content (demonstrated in Section 3.6), the stress level normalised penetration resistances values increased as the fines content reduced.

The $Q_D$ values for kaolin-sand mixtures with as little as 5% kaolin are still significantly less than those estimated for clean sands with the same relative density.

Undrained conditions could not be achieved in the 5% and 10% kaolin mixtures due to limitations on the maximum achievable velocities and the high permeabilities of these mixtures ($k$ of $\sim 4.9 \times 10^{-6}$ m/s and $8.4 \times 10^{-7}$ m/s in the 5% and 10% mixtures respectively). A standard field CPT test ($d=35.7$mm, $v=20$mm/s) would be expected to fall into the partially drained range in these mixtures.

Both centrifuge and 1g tests showed that, compared to the 100% kaolin, the 25% kaolin mixture exhibited (i) higher $Q_D$ values of 12~15 due to lower compressibility and higher friction angle but (ii) lower $Q_{UD}$ values of 1.4~2.0 due to more contractant behaviour in undrained shearing; these observations are consistent with element test data presented in Section 3.6.
• The 1g and centrifuge tests gave closely comparable results for the 100% kaolin and for the 25% kaolin mixed with 75% sand. The 1g set-up was therefore demonstrated to be a cost-effective method for examination of rate effects on penetrometer resistance in clayey soils.

(iii) Numerical analysis

• The spherical cavity expansion numerical analyses using a coupled consolidation calculation with a non-linear elasto-plastic soil model (the Hardening Soil model) illustrated clear effects of the cavity expansion velocity on the cavity expansion pressure; it is shown in this thesis and by others how this pressure can be related directly to the cone end resistance \( (q_c) \). These analyses showed that the partially drained region operates over about 3 orders of magnitude change in the velocity. An example of normalised cone resistances \( (Q) \) at various cone expansion velocities computed by the PLAXIS analyses is presented in Figure 9.3.

• In normally consolidated non-dilative/contractive soils \( (\psi=0) \), both \( Q_D \) and \( Q_{UD} \) increase with higher values in soil stiffness and friction angle \( (\phi') \); stiffness has a relatively larger influence (due to the wide range of stiffness values soils can have). The analyses allow derivation of expressions for \( Q_D \) and \( Q_{UD} \) in terms of horizontal effective stress level, stress normalised stiffness and friction angle; corresponding \( Q_D/Q_{UD} \) and \( Q_{UD}/Q_D \) ratios were inferred accordingly. \( Q_{UD}/Q_D \) ratios decrease (i.e. \( Q_D/Q_{UD} \) ratios increase) as the soil stiffness and friction angle increase. As such \( Q_{UD}/Q_D \) ratios can be expected to be lower in coarse-grained materials which are usually stiffer and have a higher friction angle than clayey soils.

• The numerical analyses demonstrated the importance of the penetrometer diameter \( (d) \) and the soil permeability and provided support for use of a normalised of velocity \( (V) \) which is proportional to \( d \) and inversely proportional to the horizontal permeability \( (k_h) \).

• As illustrated on Figure 9.4, the numerical analyses provide very good predictions of the magnitudes and velocity dependence of the CPT end resistance in normally consolidated kaolin. The numerical approach does not, however, incorporate dilatancy and therefore underestimates resistance in...
Conclusions

- The numerical analyses provided a reference resistance corresponding to zero dilation and zero viscous effects, at any given normalised velocity, which enables comparisons with actual soil penetration rate dependency to be interpreted.

![Figure 9.3 Variations of normalised cone resistance at various representative penetration rates for soils varying stiffness with constant friction angle of $\phi' = 30^\circ$ (see Section 7.4.3)](image)

![Figure 9.4 Comparison of net cone resistance in kaolin between PLAXIS analysis results and experiment at 1g (see Section 7.5.2)](image)
(iv) Database Analysis

- Experimental data in both reconstituted and intact normally consolidated and lightly overconsolidated clayey soils show that use of $V_h (=v_d/c_h)$ is reasonably effective in consistently capturing transitions between penetrometer drainage conditions. The numerical analyses indicate that this is because $c_h$ reflects $k_h$ as well as the stress level dependency of stiffness. $c_h$ is the horizontal coefficient of consolidation determined from a piezocone dissipation test (performed after undrained penetration) and hence is likely to reflect the consolidation regime surrounding a cone.

- This Thesis shows that the partially drained condition generally occurs between $V_h$ values of about 0.06 (0.03-0.1) and 30 (10-50) as shown, for example, in Figure 9.5 for Bassendean and Burswood lightly overconsolidated silty clays.

- The use of $V_h$ to assess drainage conditions for standard cone penetration in silty sands is not possible as the derivation of $c_h$ in a dissipation test requires that the preceding cone penetration is undrained.

- The hyperbolic function (Equation (8-10)) provided a good representation of the rate dependence of penetrometer resistance over the full velocity range in normally consolidated and lightly overconsolidated clayey soils.

- Due to the strong effects of viscosity at the higher velocities in undrained regions, it is proposed future research efforts should employ the drained resistance as a representative reference in the presentation of penetrometer rate dependence.

- The experiments presented in Chapters 4, 5 and 6 together with the review of previous studies on rate dependence of cone penetration resistance performed in both intact and reconstituted soils illustrate the importance of soil type on the penetration rate dependency of $Q/Q_D$ on $V_h (=v_d/c_h)$ as shown in Figure 9.6.

- Although additional variable rate penetration tests are strongly recommended to improve confidence in predictions, existing data and analysis indicate that fully undrained cone penetration is likely to be fully undrained if the time for 50% pore pressure dissipation ($t_{50}$) around a standard cone is in excess of 75 seconds. If $t_{50}$ is less than 75 seconds, it is recommended to reduce the penetration rate to 0.2 mm/s for a distance of 1 m to obtain an estimate of the drained penetration resistance.
9. Conclusions

Figure 9.5 Evaluation of penetration rate effects at the Bassendean and Burswood sites (see Section 8.3.3)

Figure 9.6 Illustration of potential penetration rate dependency on $Q/Q_D$ in varying soils (see Section 8.4.1)

9.3 RECOMMENDATIONS FOR FUTURE WORK

This Thesis has presented an experimental and numerical study of cone penetration rate effects. The importance of soil type (in terms of composition and mechanical response) on the specific rate dependency to be expected has been raised. There is, however, considerable scope for further improvements through research and practice:

- More field scale CPTs at variable penetration rates should be performed in various soils on a routine basis. Accumulation of data would assist in developing...
soil classification charts at different velocities (by avoiding partially drained conditions), which would enhance the reliability of deriving geotechnical properties from in situ tests. This is strongly encouraged for intermediate soils as obtaining good sample of these materials is relatively difficult.

- The assessment of in-situ drainage conditions during penetration intermediate and layered soils needs to be improved. In the meantime, the penetration velocity needs to be reduced to allow application of ‘drained correlations’.

- Understanding of shearing behaviours under partially drained condition may be improved in laboratory element tests. Especially for dilative/contractive soils, laboratory tests are required to interpret soil behaviours under cone penetration.

- Viscous/strain rate effects need to be quantified in laboratory tests and taken into account for interpretation of cone penetration in clayey soils as to avoid overestimation of undrained shear strength.

- A spherical cavity expansion numerical analysis using a coupled consolidation calculation can be extended employing an advanced soil constitutive model incorporating further features such as modelling a high OCR, viscosity (strain rate), strain softening and a dilation cut-off in undrained/partially-drained calculations. The effect of each factor could be evaluated from a parametric study.

- Large deformation finite element analysis with such an advanced soil model using a coupled consolidation analysis can be used to further investigate rate effects of an ‘advancing’ cone. A unified interpretation method of CPT for any drainage conditions would be developed by performing an extensive parametric study.

- Cone penetration rate dependency has been evaluated using the normalised velocity term ($V_h$). Drainage conditions for penetrometers with large diameter (such as piles) should be addressed using $V_h$ to assist in the derivation of pile base resistance.
APPENDIX A

Appendix A reports a series of monotonic simple shear test results on clayey sands and kaolin clay in the UWA (Berkeley type) simple shear apparatus (refer to Section 3.6.4). Table 3.16 summarises the results of the 15 simple shear tests performed.

![Graphs of shear stress vs. shear strain](image)

(a) shear stress - shear strain (Traditional method)  
(b) shear stress - shear strain (New method)

![Graph of volumetric strain vs. shear strain](image)

(c) volumetric strain - shear strain

Figure A-1: Drained simple shear test results of clayey sands and kaolin clay: (a) $\tau_{xy}$-$\gamma$; (b) $\tau_{fail}$-$\gamma$; (c) $\Delta u$-$\gamma$; (d) $\tau_{xy}$-$\sigma_v$ and (e) $\tau_{fail}$-$\sigma_{vc}$ at $\sigma'_{vc} \sim 50$ kPa
Figure A-2: Undrained simple shear test results of clayey sands and kaolin clay: (a) $\tau_{xy}$-$\gamma$, (b) $\tau_{\text{fail}}$-$\gamma$, (c) $\Delta u$-$\gamma$, (d) $\tau_{xy}$-$\sigma'_v$, and (e) $\tau_{\text{fail}}$-$\sigma'_n$ at $\sigma'_v \sim 50$ kPa
Figure A-3: Stress paths from the simple shear test results for 100% kaolin clay and the 25% kaolin mixture.

(a) traditional interpretation (100% kaolin clay)
(b) traditional interpretation (100% kaolin clay)
(c) new interpretation (100% kaolin clay)
(d) traditional interpretation (25% K+75% S)
(e) traditional interpretation (25% K+75% S)
(f) new interpretation (25% K+75% S)
Figure A-4: Stress paths from the simple shear test results for the 10% and 5% kaolin mixtures.
APPENDIX B

Appendix B shows variable rate penetration test results at the Bassendean site, using piezocone, T-bar and ball penetrometers (refer to Section 4.3.4). Table 4.1 summarises all of the variable penetration rate tests details including the velocities and travelling distance used for each variable rate test.

Figure B-1: Variable rate CPTU test result: BassCRT12 at 6 to 7 m depth
Figure B-2: Variable rate CPTU test result: BassCRT13 at 4 to 5 m depth

Figure B-3: Variable rate CPTU test result: BassCRT14 at 4 to 5 m depth
Figure B-4: Variable rate CPTU test result: BassCRT14 at 7 to 8 m depth

Figure B-5: Variable rate CPTU test result: BassCRT15 at 5 to 6 m depth
Figure B-6: Variable rate CPTU test result: BassCRT15 at 7.3 to 8.3 m depth

Figure B-7: Variable rate CPTU test result: BassCRT18 at 3 to 3.5 m depth
Figure B-8: Variable rate T-bar test result: BassTRT2 at 3 to 4 m depth

Figure B-9: Variable rate T-bar test result: BassTRT2 at 4 to 5 m depth
Appendix B

Figure B-10: Variable rate Ball test result: BassBRT2 at 3 to 4 m depth

Figure B-11: Variable rate Ball test result: BassBRT2 at 4 to 5 m depth
APPENDIX C

Appendix C tabulates the variable rate cone penetration test results on clayey sands in the drum centrifuge test (refer to Section 5.4).

Table C-1: Variable rate CPT results on clayey sands in the drum centrifuge

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<th>$v$</th>
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Appendix C

Table C-2: Variable rate CPT results on clayey sands in the drum centrifuge (Cont.)
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10K+90S
10K+90S
10K+90S
10K+90S
10K+90S
10K+90S
10K+90S
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102
96
98
100
102
97
98
100
102
96
98
100
102

z

v

v0

u0

'v0

qc

qcnet

Q

cv

Vv

(mm)

(mm/s)

(kPa)

(kPa)

(kPa)

(kPa)

(kPa)

-

(m2/yr)

-

71
81
91
100
70
80
90
100
88
98
108
70
80
91
100
70
80
90
101
70
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91
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No Box4 data were used

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APPENDIX D

Appendix D provides the papers published along with the research of this Thesis.


NOTE: full text articles removed due to copyright restrictions.
REFERENCES


Itasca FLAC-Fast Lagrangian Analysis of Continua. Itasca Consulting Group Inc.


Stewart, D. P. (1990). *Lateral loading of piles in soft clay due to nearby embankment construction*. Research report GEO: 90086, Department of Civil and Resource Engineering, The University of Western Australia,


