In situ soil testing for foundation performance prediction

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A thesis submitted for the degree of
Doctor of Philosophy
March 2008
Abstract

Our understanding of non-linear soil behaviour at small strains hinges on laboratory static and dynamic element testing of ideal undisturbed samples. However such samples are prohibitively expensive for day to day geotechnical engineering projects, especially in sandy soils. In addition to soil’s complex nature, construction processes also have an important effect on foundations’ behaviour under working load. This is especially true in piling engineering. It is well acknowledged that the base stiffness of a displacement pile is much higher than that of an equivalent bored pile. To be able to understand and predict such effects of geotechnical processes on soil-foundation performance, it is necessary to develop practical techniques to measure both the in situ non-linear soil behaviour and any construction-induced changes. This thesis presents an attempt at fulfilling both of these objectives within the limited scope of wished-in and pressed-in model piles in a dense silica sand. Simplified methods of pile base settlement prediction are then developed using the analogy with spherical cavity expansion.

To measure monotonic soil properties in situ, a new miniature pressuremeter was developed for the centrifuge. The design successfully avoided any installation disturbance by adopting a wished-in installation process, and produced repeatable results in the small to intermediate strain rage. However, membrane penetration errors proved severe for the current design at small strains. The large-strain cross-hole method of Salgado et al. (1997a) was successfully implemented in the centrifuge. Small-strain stiffness and its dependency on mean stress level were successfully measured and compared well against data obtained from triaxial tests. The dynamic shear stiffness-strain relationship was also estimated assuming a power law constitutive model of Bolton & Whittle (1999).

The in situ shear stiffness field around a penetration pile was measured directly using the large-strain seismic method both during pile jacking and after pile unloading. An estimate of the corresponding in situ stress field was made from the stiffness field, giving direct evidence of stiffer soil behaviour and the existence of large locked-in stresses after pile jacking. These higher values of shear stiffness and large locked-in stress are shown to be the main reason behind the stiffer load-settlement behaviour of a displacement pile vs a non-displacement pile. The effect of soil densification due to the action of pile penetration does not contribute significantly to the above difference in behaviour.
Declaration

I hereby declare that, except where specific reference is made to the works of others, the contents of this thesis are original and have not been submitted, in whole or in part, to any other university for any degree or qualification. This thesis is entirely the result of my own work and includes nothing which is the outcome of collaboration.

Permission to exceed the recommended limits of 65,000 words and 150 figures was granted by the Board of Graduate Studies. This thesis is presented in less than the revised limits of 65,000 words and 170 figures.

Yueyang Zhao

04/March/2008
Acknowledgements

Foremost, I would like to thank my supervisor Prof. Malcolm Bolton for his untiring support and guidance. I was very much ill prepared for the many difficulties within a research environment. Too many times it could have ended in tragedy. Malcolm pulled me through with great faith. This work is as much as mine as is his. My girl friend, Rachel, commented succinctly that many PhDs, in one small way or another, turn out to be carbon-copies of their supervisors. I believe what little I managed to learn from Malcolm will serve me well in research and in life.

The first Sunday after my oral exam Prof. Andrew Schofield described to me the feeling of going through a tough viva: “one’s guts being thrown onto the floor and then must be picked up bits by bits”. It was a privilege to study at the Schofield Centre knowing its importance in the development of soil mechanics.

An old friend of mine, Zhou Yun, described the usual formatting of a thesis’ acknowledgement section as akin to a ‘shopping list’. And now I have my own!

I would like to thank my two examiners for their thorough review and criticism of this thesis which helped it to do better justice for the experimental data.

In no particular order, I would like to thank the centrifuge center’s technical and administration team Anama Lowday, Steve Chandler, Chris Collison, John Chandler, Kristian Pether, Chris McGinnie, Tom Johnston and Frank Sixsmith for their help day in, day out; Alistair Ross and his team of technicians of the Engineering Department Instrument Workshop for their top quality work, in particular Gerry and John for their patience with training a young lad like me the ‘art of the lathe’. I am also grateful to the technicians of the Engineering Department Structural Group for helping me with their
Instron; the past and present staff of the Departmental Library for their help in obtaining difficult references.

Chris Knight has been brilliant with the setting up of the triaxial apparatus. Profs. Kenichi Soga and Robert Mair have given much support and valuable advice. Mohammed El Shafie performed a number of triaxial tests, co-designed the triaxial sand pourer and tried his best to ‘coerce’ funding for the triaxial project; without his initiatives the triaxial data would be much the poorer.

Marcelo Silva, Stuart Haigh spent valuable time with me in the centrifuge control room, averting disasters and completing Sudokus in record time. Many fellow researchers and visitors of the centrifuge centre provided valuable discussions and help which created a enjoyable environment to work in. In particular my next-desk neighbours Jonathan Knappett wasted hours discussing my work and helped in his capacity as an excellent photographer and Rich Laver who spent many a supper time correcting mundane grammar errors of this thesis.

I would like to thank Cementation Foundations Skanska and Cambridge Overseas Trust for funding this study, especially the support given by Rab Fernie and Martin Pedley. William SD Louey Hong Kong Educational Foundation’s extraordinary generosity made my study in the UK possible. Finally I would like to thank Rachel and my family for their unwavering support and encouragement over the difficult times of the past four years.
Nomenclature

$V$ Wave velocity
$\rho$ Mass density
$G, E$ Shear and Young’s modulus respectively
$G_{\text{max}}, G_0$ Small strain shear modulus
$E_{\text{max}}, E_0$ Small strain Young’s modulus
$G_{\text{secant}}, G_{\sec}$ Secant shear modulus
$G_{\text{tangent}}, G_{\tan}$ Tangent shear modulus
$E_{\text{secant}}, E_{\sec}$ Secant Young’s modulus
$E_{\text{tangent}}, E_{\tan}$ Tangent Young’s modulus
$\nu$ Poisson’s ratio
$p'$ Effective confining stress
$q$ Triaxial shear stress
$q_{\text{max}}$ Maximum triaxial shear strength
$\sigma'_1, \sigma'_2, \sigma'_3$ Major, intermediate and minor effective principle stresses respectively
$\sigma'_h, \sigma'_v$ Effective horizontal and vertical stress respectively
$p_a$ Atmospheric pressure
$K$ Earth pressure coefficient
$e, e_{\text{min}}, e_{\text{max}}$ Void ratio, minimum and maximum void ratio respectively
$\gamma$ Engineering shear strain
$\phi'$ Angle of friction
$\phi'_{\text{peak}}, \phi'_{\text{crit}}$ Peak and critical state angle of friction respectively
$D_x$ Particle size such that $x\%$ fraction-by-volume of all particles are smaller than $D_x$
$G_s$ Specific gravity
$q_c$ Cone penetration resistance
$q_b$ Pile base load
$W$ Pile settlement
$R, D$ Pile radius and diameter respectively
Contents

1 Introduction
  1.1 Overview ......................................................... 2
  1.2 Measurement of static soil stiffness .................................... 3
  1.3 In situ measurement of dynamic soil stiffness .......................... 3
  1.4 Pile base performance under working load ........................... 4
  1.5 Dissertation structure ........................................... 5

2 Literature review
  2.1 Introduction .................................................... 8
  2.2 In situ stress and strain measurement ............................... 9
  2.3 Material behaviour ............................................. 12
    2.3.1 Stress-strain behaviour ...................................... 12
      2.3.1.1 Analytical models of $G_0$ ............................... 12
      2.3.1.2 The existence of quasi-elastic behaviour ................. 13
      2.3.1.3 Stress-strain relationship ............................... 13
    2.3.2 Introduction to anisotropy .................................. 14
      2.3.2.1 Anisotropy beyond the quasi-elastic region in the dilative
               region of behaviour ...................................... 14
      2.3.2.2 Anisotropy towards failure in the contractive region of
               behaviour .................................................. 15
    2.3.3 In situ measurement of $G_0$ and stress-strain relationship ...
      2.3.3.1 Seismic methods ......................................... 16
      2.3.3.2 Pressuremeter ............................................ 17
      2.3.3.3 Correlations with $q_c$ .................................... 17
    2.3.4 Limitations .................................................. 18
  2.4 On deep penetrating probes ...................................... 19
    2.4.1 Probe size effect on $q_c$ ................................... 19
2.4.1.1 Early literature evidence .......................... 19
2.4.1.2 Recent literature evidence ........................ 20
2.4.1.3 \( q_c \) and \( q_b \) ............................... 20

2.4.2 Predicting \( q_c \) and \( q_b \) ............................... 21
  2.4.2.1 Pile base load settlement prediction ............ 22

2.4.3 Pile shaft and sand interaction ........................... 22
  2.4.3.1 Overview ...................................... 22
  2.4.3.2 Survey of past experimental work .................... 23
  2.4.3.3 Survey of more recent experimental works ............ 24
  2.4.3.4 Inside the shear band mechanism ...................... 25
  2.4.3.5 Constitutive modeling of an interface ................. 26
  2.4.3.6 Physical modeling of a rough interface ............... 26
  2.4.3.7 Summary ..................................... 27

2.4.4 Pile shaft upon base interaction ............................ 27

3 Triaxial testing 47
  3.1 Introduction ........................................ 48
  3.2 Apparatus, sample preparation and testing procedures ........... 48
    3.2.1 Sample former .................................. 49
    3.2.2 Sand pluviation ................................ 50
      3.2.2.1 The Triaxial Sand Pourer ....................... 50
      3.2.2.2 Density of the samples ......................... 51
    3.2.3 Friction ends .................................... 51
    3.2.4 Placing of the top cap and applying a suction ............. 52
    3.2.5 Installing the instrumentation ......................... 52
      3.2.5.1 The radial belt ................................ 52
      3.2.5.2 Mounting the axial LVDTs ....................... 56
    3.2.6 Docking ......................................... 56
      3.2.6.1 Dimple and knob arrangement ..................... 56
      3.2.6.2 Knob-on-flat plate arrangement .................. 57
  3.3 Anomaly during shearing .................................. 58
    3.3.1 Identification of anomalies .......................... 58
    3.3.2 Strain rate effect ................................ 58
    3.3.3 Effect of the anomalies on stiffness data ............... 59
    3.3.4 Other considerations ................................ 59
3.4 Test Data ................................................................. 60
  3.4.1 Test summary ...................................................... 60
  3.4.2 Axial strain rates ................................................ 61
  3.4.3 Preliminary tests and repeatability ............................ 61
  3.4.4 Standard TC tests ............................................... 62
  3.4.5 Constant-p TC tests ............................................. 62
3.5 Soil material model parameters .................................. 62
  3.5.1 Critical state angle of friction ................................. 62
  3.5.2 Peak angle of friction .......................................... 63
  3.5.3 Comparison with Lee ........................................... 63
  3.5.4 Small strain stiffness, Poisson’s ratio and stiffness degradation .
        3.5.4.1 Fahey-Carter model ..................................... 63
        3.5.4.2 Curving-fitting results ................................. 64
4 Experimental methods ................................................. 109
  4.1 Introduction .......................................................... 110
  4.2 Test sand and sand pouring ...................................... 110
        4.2.1 Fraction E silica sand ..................................... 110
        4.2.2 Soil model preparation ..................................... 111
  4.3 Piling Package ..................................................... 112
        4.3.1 Model pile design .......................................... 112
        4.3.2 Manufacture and calibration of tip load cells ............ 113
        4.3.3 Piling locations ........................................... 114
        4.3.4 Installation and load test of a pressed-in pile .......... 114
        4.3.5 Wished-in pile placement .................................. 114
        4.3.6 Load test of a wished-in pile ............................ 115
        4.3.7 Calibration of the short-range LVDT ...................... 115
  4.4 Seismic package .................................................... 115
        4.4.1 Air hammers and accelerometers ........................... 115
        4.4.2 Arrangement of accelerometer arrays ...................... 116
        4.4.3 Accelerometers data processing ........................... 117
        4.4.4 Testing procedure ......................................... 119
  4.5 Centrifuge test series .......................................... 120
        4.5.1 Failed experiments ......................................... 121
        4.5.1.1 Failed pile tests .................................. 121
5 Centrifuge in situ testing

5.1 Introduction ......................................................... 141
5.2 Results of centrifuge pile tests ................................. 141
  5.2.1 Sample uniformity and test repeatability .................... 141
  5.2.2 Influence zone .................................................. 142
  5.2.3 Mobilized base resistance ..................................... 142
  5.2.4 Mobilized shaft resistance .................................... 142
  5.2.5 Residual loads .................................................. 143
5.3 In situ pressuremeter testing ................................. 150
  5.3.1 Design ............................................................ 150
  5.3.2 Preparation ...................................................... 151
  5.3.3 Raw measurement corrections ................................ 152
    5.3.3.1 System compliance ....................................... 152
    5.3.3.2 Correction for water head ............................... 152
    5.3.3.3 Correction for membrane thickness ................... 153
    5.3.3.4 Corrections for membrane strength .................... 153
  5.3.4 Prediction and measurement ................................ 153
    5.3.4.1 CAMFE finite element program ......................... 153
    5.3.4.2 CAMFE prediction ....................................... 154
    5.3.4.3 Measurements ............................................. 154
    5.3.4.4 Membrane Penetration .................................. 155
5.4 In situ seismic testing ........................................ 163
  5.4.1 Interpretation of velocity measurements .................... 163
  5.4.2 Small-strain stiffness variations with stress level ........ 165
  5.4.3 Stiffness degradation with strain level ..................... 165
  5.4.4 Comparison with the triaxial stiffness degradation data ... 166
    5.4.4.1 Strain rate .............................................. 166
    5.4.4.2 Cyclic shear strain amplitude ......................... 166
    5.4.4.3 Number of loading cycles ............................... 167
    5.4.4.4 Stress path and anisotropy ............................. 168
    5.4.4.5 Comparison with monotonic triaxial data .............. 168
6 Back analysis using cavity expansion methods

6.1 Estimation and prediction of in situ stiffness and stress during pressed-in piling

6.1.1 Introduction

6.1.1.1 Stress-induced anisotropy and limitations on prediction

6.1.2 Measurement of in situ stiffness

6.1.3 Predictions using the cavity expansion method

6.1.3.1 Step 1: Matching the cavity limit pressure to $q_b$

6.1.3.2 Step 2: Large-strain spherical cavity expansion-contraction

6.1.4 Discussions

6.2 Pressed-in pile base load settlement curve

6.2.1 Cavity expansion

6.2.2 Base hemisphere

6.2.3 Pile base settlement model

6.2.3.1 Cavity expansion

6.2.3.2 Hemisphere compression

6.2.3.3 Total settlement

6.2.3.4 The value of $G_{cav}$

6.2.3.5 The value of $G_{hemi}, \nu_{hemi}$

6.2.4 Prediction of pile base stiffness

6.2.5 Discussions

6.3 Wished-in pile base load settlement curve

6.3.1 Introduction

6.3.2 Modeling of cavity expansion

6.3.2.1 Equivalent model for cylindrical cavity expansion in Fahey-Carter material

6.3.2.2 Prediction of model pile base stiffness

6.3.3 Discussions

6.3.3.1 At the limiting design load

6.3.3.2 Comparison of pressed-in and wished-in piles

6.3.3.3 Impact of installation procedure

6.4 Pile shaft load-settlement curve

6.4.1 Mechanism of pile shaft friction mobilization

6.4.2 Analysis of pile shaft friction mobilization
6.4.3 Simple theoretical prediction of shear distortion
6.4.3.1 Governing equation
6.4.3.2 Power law relationship for stiffness and strain
6.4.3.3 Evaluating the definite integral
6.4.3.4 Estimated magnitude of shear deformation
6.4.4 Discussions

7 Conclusions and future work

7.1 Conclusions
7.1.1 Triaxial testing techniques
7.1.2 Centrifuge mini-pressuremeter
7.1.3 Centrifuge seismic method
7.1.4 Prediction of in situ stress during pile jacking
7.1.5 Pile base load-settlement prediction
7.1.6 Evaluating model pile shaft data and shaft-upon-base interaction

7.2 Discussions and recommendations for future work
7.2.1 Development of centrifuge soil testing techniques
7.2.2 Large-strain seismic field testing
7.2.3 Assessing the impact of geotechnical processes on pile performance: THINK STRESS!
7.2.4 Strategy for the reuse of foundations
7.2.5 Implications to piling design and construction practices

References
Chapter 1

Introduction
1.1 Overview

Our understanding of the practical significance of non-linear soil behaviour at smaller strains hinges on laboratory static and dynamic element testing of ideal and undisturbed samples, although they are still prohibitively expensive$^1$ for day to day geotechnical engineering projects, especially in sandy soils. The overriding importance of in situ soil fabric also renders the cheaper standard triaxial testing of remoulded samples much less useful, which is anyway not usually equipped to measure strains smaller than $10^{-3}$.

Turning to in situ techniques, traditional cross-hole or down-hole tests can provide direct measurement of small strain stiffness; CPT (cone penetration test) or SPT (standard penetration test) can provide penetration resistance or an indication of soil types, density and strength; whilst the self boring pressuremeter is theoretically capable of covering the strain range between the previous two classes of method, careful interpretation of unload-reload loops is required to account for insertion disturbance in granular materials. It is becoming more popular to combine small, intermediate and large strain methods into one tool, saving money and time; prominent examples are seismic CPT, seismic dilatometer and cone pressuremeter.

Salgado et al. (1997a) and Drnevich et al. (1995) have attempted validation of a large-strain cross hole method via full scale in situ testing. Fahey (1991), Fahey (1993), Fahey & Carter (1993), Fahey & Soliman (1994) and Soliman & Fahey (1995) have used full scale in situ testing and finite element simulation to validate the interpretation of self boring pressuremeter in sands. However, more work is required to develop economical site investigation tools to measure $G_0$ and the stiffness degradation behaviour and to test their applicability to pile performance prediction in a controlled environment afforded by the geotechnical centrifuge. Such advances could be based on the data of centrifuge model tests, but only if “in flight” soil tests can be developed to represent in situ stress-strain tests. This thesis explores an in situ seismic method to measure dynamic stiffness and stress-strain relationships; it also develops a new miniature pressuremeter for centrifuge static in situ testing albeit with only limited success in this case.

The seismic data was also used to estimate in situ stresses during pressed-in piling and to explore a cavity expansion view of the pile base capacity mobilization mechanism following the success of spherical cavity expansion solutions in predicting this

$^1$According to Karl (2006) resonant column or cyclic triaxial test on one sample only costs around 1200 pounds; compared to 1700 pounds for cross-hole test to 12m depth with one source and two receiver holes; and to 600 pounds for SCPT over 12m depth
measured stress field. Coupled with the measurement of non-linear stress-strain relationships, predictions of base settlement behaviour of pressed-in and wished-in piles were attempted.

1.2 Measurement of static soil stiffness

To measure the static behaviour of a soil, one approach is to carry out undisturbed sampling and advanced element testing in the laboratory. A reference static backbone can be obtained for comparison with other methods. This route is expensive.

The measurement of the static monotonic loading stress versus strain behaviour (known as the static backbone curve) in situ, at the present, relies on back analysis of in situ boundary problems, the most promising being cylindrical cavity expansion, afforded by a self-boring pressuremeter. However the insertion process introduces much uncertainty during the subsequent back-analysis; this has long been an active area of research (e.g. Fahey (1980)); however it is outside the scope of this thesis. Instead a pre-installed, miniature pressuremeter was developed for use in the centrifuge to measure intermediate strain behaviours. Triaxial test results afford a direct validation of this new centrifuge technique.

1.3 In situ measurement of dynamic soil stiffness

For a newly prepared sample under monotonically increasing load following a defined stress path one obtains a stress-strain curve. We will refer to this curve as the ‘static backbone’. If one proceeds to conduct unloading and reloading cycles following this monotonic loading stress path one will obtain different stress-strain curves (or loop, since the loading history is a closed loop) and it may be found that, when the number of loading cycles becomes large, they converge towards a well defined stress-strain loop with an associated secant stiffness (measured by joining the apex of the stress-strain loop); by varying the magnitude of loading a series of stress-strain curves can be obtained with associated secant stiffness for different magnitude of loading. These stiffness and magnitude of loads defines a stress-strain response of a cyclic loaded soil and it is referred to as the ‘dynamic backbone’. This terminology is popular within the soil mechanics community because the early dynamic backbone data were obtained from dynamic resonant column tests whilst the static backbone from static triaxial tests. A number of factors distinguishes the two backbones, they are number of loading
cycles, the magnitude of these cycles and the strain rate at which these cycles are applied.

If the strain rate effects is ignored for clean dry silica sand used in this study (this point will be discussed in more detail in later sections) the main factor that remains is the number of loading cycles applied, \( N \), where \( N = 1 \) for a static test and \( N > 1 \) for a dynamic (or simply cyclic) test. Both static and dynamic information are relevant to foundation design, especially in an era of increasing reuse of foundations. Numerous loading cycles of different amplitudes and loading rates are unavoidable even for structures onshore and the number of large loading cycles to modify the stress-strain behaviour of a soil element from one that is characterized by its static backbone to one that is better approximated by its dynamic backbone may be small (\(< 10\), for example).

To measure the dynamic backbone in situ, a modified cross-hole method presented by Salgado et al. (1997a) and Drnevich et al. (1995) appears to be the only published commercially available method other than from back-analysis of earthquake response, for example as carried out in Brennan et al. (2004). It employs a conventional cross-hole testing arrangement, with multiple sensors and a ‘tunable’ seismic source. The ground shear strain level achieved at different sensor locations ranges from \( 10^{-3} \) to \( 10^{-6} \) which are above the commonly assume linear quasi-elastic strain range of \( 10^{-6} \), allowing the data to be fitted by a non-linear stress-strain soil model; this method will be introduced in more detail later on in this thesis.

With the benefit of small scale centrifuge modeling, this thesis validates this seismic method and demonstrates its ability to measure both \( G_0 \) and subsequent stress-strain behaviour over the intermediate strain range (\(< 10^{-3}\) in the ideal conditions of the centrifuge.

### 1.4 Pile base performance under working load

The construction process has an important effect on the foundation’s subsequent behaviour under load. This is especially true for piling engineering, where the affected soil volume is large relative to that of the foundation itself; also the observation that the base stiffness of a displacement pile is much higher than that of an equivalent bored pile is well acknowledged (e.g. Randolph (2003); Deeks (2004)). Therefore prediction of foundation behaviour must take into account construction induced changes in soil stiffness and strength. Hence methods to measure and predict the construction-induced in situ changes of strain, stiffness and stress must be developed, aided by numerical
or physical modeling. This is the aim of the thesis in relation to pile performance prediction.

Lee & Salgado (1999) and Lee & Salgado (2005) demonstrated that the finite element method (FE) is capable of modeling bored pile base settlement behaviour. However for displacement piles, FE modeling of insertion and subsequent load testing is currently not feasible and is the subject of active research outside the scope of this thesis. This thesis adopts the physical modeling approach. The seismic method’s ability to infer in situ stresses is employed to obtain an estimate of the stress field around a deep penetrating pile in dense sand and a spherical cavity expansion solution is developed to predict this field.

Displacement pile penetration mechanism has long been visualized via a spherical cavity expansion, which also proved useful in modeling non-displacement piles. For example, Yasufuku & Hyde (1995) and Yasufuku et al. (2001) presented a successful and simple semi-empirical method for predicting bored pile base settlement. Supported by the ability of spherical cavity expansion solutions in estimating ground stress during pile penetration, this thesis develops a simple cavity expansion mechanism of pile base load mobilization for displacement and non-displacement piles, following the spirit of Bolton (1993):

“For more than a method of prediction, the designer needs a descriptive mechanism which embodies the working of any geotechnical facility”

1.5 Dissertation structure

Chapter 2 explores the existing literature, upon which the experimental investigations are built. Chapter 3 presents a limited series of triaxial tests which provides static stiffness and strength parameters of the sand used in this study. Chapter 4 details the centrifuge testing methodology adopted by this thesis, concentrating on piling and a large-strain seismic method. Chapter 5 summarizes the main results of the centrifuge experiments, including a detailed description of the methodology of the miniature pressuremeter. Chapter 6 attempts back-analysis of the centrifuge piling data, including the measurement and prediction of piling induced in situ stiffness and stress changes, using cavity expansion methods; presents methods of pile base performance prediction using a simple cavity expansion based pile base load mobilization mechanism. Chapter 7 summarizes the contributions made by the previous chapters and presents paths for future research.
Definitions of commonly used symbols are given in a nomenclature section. Tables are placed in or near the page within which they are first mentioned. Figures are placed at the end of a section or a chapter within which they are first referenced and their page numbers are given within the main text. References are listed at the back of the thesis and the page numbers within which they are cited are listed at the end of each citation entries.
Chapter 2

Literature review
2.1 Introduction

This literature review aims firstly to:

(1) Review the current knowledge of soil behaviour; highlight important model parameters.

(2) Review the available laboratory and in situ soil testing techniques and explain both the necessity for, and the limitation of, in situ methods.

(3) Showcase the seismic method as a tool for stress, strain and stiffness measurement.

Thus far, the literature review hopes to convey the conviction that seismic in situ testing is capable of providing the essential information required to quantify soil non-linearities. Having made our survey of soil testing techniques and soil models, the following sections will review their application to the understanding of piled foundations in sand. We will deal with pile base and shaft separately:

(4) Review the literature on pile base capacity and load settlement behaviour, concentrating on displacement piles, especially the difficulty of predicting $q_b$ and the empirical proposition that $q_b = q_c$.\(^1\)

(5) Review the literature on soil-structure interface shear, which is fundamental to the understanding of pile shaft-soil interaction; concentrating on the importance of interface slippage, and on highlighting the difficulty of modeling the pile shaft interface behaviour.

(6) Review the literature on the evidence of shaft-upon-pile interaction\(^2\) and quantify its magnitude for the centrifuge pile tests of this thesis; the result is shown to reinforce the current practice of treating the base and shaft separately.

This chapter aims to give confidence in the empirical rule: $q_b = q_c$, which will form the foundation of back analysis methods proposed in this thesis; and to convince the reader that the practice of treating pile base and shaft independently may introduce only small errors for the model piles considered in this thesis. The reader’s attention will be directed towards the modeling of pile base response, but it must be acknowledged that the difficulties with analyzing model pile shafts is outside the scope of this thesis.

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\(^1\) $q_b$ and $q_c$ stand for pile base penetration resistance and CPT cone resistance respectively

\(^2\) One manifestation of this interaction is the following postulate: an increase in shaft friction, due to increased surface roughness say, may increase the base resistance at the same time
2.2 In situ stress and strain measurement

To obtain a direct stress-strain relationship must involve the measurement of both stress and strain fields. The measurement of in situ stress has proved very difficult due to soil-structure interaction between soil and the buried pressure cells. This is made much worse in a centrifuge due to the limitation on equipment size.

Many commercial earth pressure cells work on the principle of strain-gauged diaphragm action under uniformly distributed pressure. The active interface between soil and the instrument is usually a thin metal sheet, resembling the diaphragm of a drum. This sheet is usually strain-gauged so its deflection will generate an output strain-gauge signal that is proportional to the magnitude of its deformation. Therefore the deformation of the diaphragm is the mechanism behind these designs and the ‘stiffness’ of such a system can be quantified as the amount of diaphragm deformation per unit pressure applied to it. Since larger the deformation the higher the strain-gauge signal output, it is important to keep the stiffness low enough so that a tolerable signal to noise ratio is achieved. These designs are very successful when only fluid pressure is to be measured.

Turning to their applications in soils Chua (2006) has given a thorough review of the use of buried total pressure cells. It was found that the relative stiffness between the instrument and the surrounding soil is an important factor, this relative stiffness is sometimes known as the flexibility ratio. If the flexibility of an instrument is high, its diaphragm will deform more than the surrounding soil under a change of in situ stress. This difference in deformation is a 2 dimensional problem, since the diaphragm itself is 2 dimensional. At the center of the diaphragm the deformation must be the largest, whilst is zero at its edges if it’s rigidly supported there. As a result the local soil deformation along the face of the diaphragm can not be uniform. This is a serious problem given the fact that soil stiffness is a sensitive function of both strain and stress and this non-zero and non-uniform deformation itself must introduce change of local stiffness and modify the local stress field which it is trying to measure. This unwanted modification of the local stress field is sometimes known as the soil-structure interaction problem. The ideal design is an instrument that has the same small strain stiffness and the same stiffness degradation characteristics with strain as the soil and it does not introduce local variations in strain. This is not possible at the present. The instruments available to the researchers today have characteristic stiffness that can not be changed.
Chua (2006) has also examined the performance of some commercially available miniature pressure cells, suitable for centrifuge experiments. The instruments investigated are Entron miniature earth pressure cells with 7 bar and 25 bar pressure range variants. It was found during loading and unloading cycles these instruments recorded significant hysteresis which equated to as much as +/-40% variations from the manufacturer’s calibration coefficients. What more, the variations changed from test to test and were sensitive to sand density and the precise method of pressure cell placement. Therefore currently available commercial pressure cells are not suitable for centrifuge experiments in stiff granular soils like sand.

The problem of flexibility for the situation where a pressure cell is required to be mounted onto rigid structural surfaces and measure soil pressure is comparably simplified. In this case it only requires the instrument flexibility to be zero, since the rigid structure is usually much stiffer than soil. To resolve the soil-structure interaction problem, Talesnick (2005) has developed an active infinite-stiffness diaphragm pressure cell, which is able to totally avoid arching problems. The cell actively changes the fluid pressure inside the diaphragm to null the movement of it. But the cell is necessarily bulky and expensive. A schematic and control diagram of this device is shown in Figure 2.1 on page 29. As indicated in this figure, as soil pressure changes the diaphragm of the soil pressure transducer deforms and produces an output that is in proportion to the magnitude of the deformation. This signal is filtered for noise and sent to a computer for analysis. Depending on the magnitude of the signal the computer generates a command signal to the electro pneumatic converter which varies the regulated air pressure inside the pressure transducer and the transducer’s output is fed back to the computer for another cycle of pressure adjustment; the aim of this control process is to reduce the diaphragm movement to zero and the required increment in regulated air pressure to achieve is equal to the soil pressure increment which the instrument is measuring. Therefore the system never deforms under load and has a practical diaphragm stiffness of infinity.

Garnier et al. (1999) and Gaudin et al. (2005) demonstrated that using a large diameter (80mm diameter) diaphragm pressure cell in a uniform fine sand, one can confidently measure the uniform stress field due to centrifuge swing up and down, without introducing visible hysteresis. However the size of the instrument seriously limits its use in the centrifuge for measuring non-uniform stress fields and any attempt
to reduce its size would eventually run into soil-arching related hysteresis problems, discussed in Chua (2006) and Talesnick (2005).

Seismic wave velocity measurement employed in the cross hole method is much less influenced by the details at the soil-structure interface and it can be used to estimate in situ stress, since relationships between the velocity and stress are well established (Santamarina & Cascante (1996); Jovicic & Coop (1997); Viggiani & Atkinson (1995)). It is extensively used in the lab and in the field to quantify in situ stress and void ratio (Cunning et al. (1995); Fioravante et al. (1998); Robertson et al. (1995); Hatanaka & Uchida (1996);), in situ fabric (Tatsuoka (1999); Shibuya et al. (2001)) and fabric change due to densification (Thomann & Hryciw (1992)). Accelerometers signal can be integrated to give an estimate of displacement, and hence shear strain, assuming a simple shear mode as performed in Brennan et al. (2004). This technique may therefore provide estimates of stress and strain simultaneously. A disadvantage is that instead of measuring at a point, it is measuring over a finite range, the distance spanning the adjacent accelerometers; therefore it is measuring an average or integrated behaviour over this distance. Therefore the more numerous and closely-spaced the instruments, the better the approximation to the true stress distribution.

An existing method that utilizes these ideas is a large-strain cross hole method described by Salgado et al. (1997a). Its principle is demonstrated in Figure 2.2 on page 30 and with typical data and back analysis in Figure 2.3 on page 31, reproduced from their paper. Sensors placed at different distances from the anchor hole receive signals of different strength at different times after the moment of hammer impact; the difference in arrival times between 2 sensors can be obtained by correlation of the 2 signals for example . From these streams of information one can construct a plot of wave speed vs shear strain. By assuming a particular soil constitutive model, one can perform a curve-fitting exercise to obtain the parameters of the chosen model. The wave speed is linked to secant shear stiffness by: \( G = \rho V_s^2 \); also \( G_0 = \rho V_{max}^2 \) and the shear strain, \( \gamma \), is calculated as the ratio between peak particle velocity, \( v_s \), measured by the sensors and the shear wave speed, \( V_s \), such that \( \gamma = v_s / V_s \). The interpretation of seismic data will be described in more detail in section 5.4 and 6.1 and the rest of this chapter.
2.3 Material behaviour

2.3.1 Stress-strain behaviour

Two themes of development regarding $G_0$ of granular materials are of particular interest to this thesis: 1) The ability to analytically model the bulk $G_0$ from more fundamental properties like the grain’s physical properties and some assumptions about the contact between grains. 2) The measurement of $G_0$ and stress-strain relationship in situ.

Traditionally this behaviour is either measured using static/monotonic tests or dynamic/cyclic tests. The relationship between dynamic and static behaviour will be discussed in more detail in section 5.4.4. In the following discussion strain ranges below $10^{-6}$ will be referred to as ‘small strain’, between $10^{-6}$ and $10^{-3}$ as ‘intermediate strain’ and those above $10^{-3}$ as ‘large strain’. This distinction is arbitrary.

2.3.1.1 Analytical models of $G_0$

Liao et al. (2000) states that stiffness of a granular assembly can be related to the properties of the granular micro-structure by using homogenization theories, for example the effect of void ratio on $G_0$, see Figure 2.4 on page 32; and the effect of stress on $G_0$ for ideally packed spheres calculated by Santamarina & Cascante (1996), see Table 2.1 on page 13. Homogenization theories typically take these steps in modeling:

1) First assume a particle contact model. The most common is Hertzian contact, which is developed for two spheres; it links the particle displacement and particle contact force (for example please referred to Goddard (1990)).

2) Then pick a relationship between particle contact displacement and continuum strain relationship. For example using a simple ‘kinematic’ relationship:

$$\text{Displacement} \div \text{Centre-to-centre Distance} = \text{Strain}$$

3) Finally make a hypothesis that links macro-stress to contact force, typically using a ‘mean stress’ or a ‘virtual work’ relationship. For example:

$$\text{Stress} \times \Delta \text{Unit Volume} = \text{Summed Forces} \times \text{Displacement Vector}$$

These elegant simple analytical models give physical meaning to the observed empirical relationship between $G_0$, density and stress (Liao et al. (2000); Santamarina & Cascante (1996); McDowell & Bolton (2001)). They are incorporated into theories based on Critical State soil mechanics for remoulded soils, which states that $G_0$ may be
Table 2.1: Theoretical prediction of the effect of stress on $G_0$; $C$ is the coordination number, which is the total number of contact of a particle; this table is modified from Santamarina & Cascante (1996)

<table>
<thead>
<tr>
<th>Packing</th>
<th>$G_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple cubic, $C = 6$</td>
<td>$\frac{[1.5(1-\nu)p'/G_{\text{grain}}^2]^{1/3}}{(2-\nu)}$</td>
</tr>
<tr>
<td>Body-centered cubic, $C = 8$</td>
<td>$\frac{9[(1-\nu)p'/G_{\text{grain}}^2/G_0]^ {1/3}}{(6-5\nu)}$</td>
</tr>
<tr>
<td>Face-centered cubic, $C = 12$</td>
<td>$\frac{(4-3\nu)[3p'/G_{\text{grain}}^2]^{1/3}}{2(2-\nu)^2}$</td>
</tr>
</tbody>
</table>

determined uniquely by knowing the current stress state, since a unique relationship between density and stress is assumed e.g. demonstrated in Figure 2.5 on page 33 (Jovicic & Coop (1997); Viggiani & Atkinson (1995); Shibuya et al. (2001); Coop (1999)).

However, more complex analytical models struggle to deal with more realistic situations, for example involving friction between particles, particle surface roughness and other elements of the soil fabric (Santamarina & Cascante (1998); Slade & Walton (1993); Zhao (2008)) gives additional review on these difficulties.

2.3.1.2 The existence of quasi-elastic behaviour

The existence of quasi-elastic behaviour at very small strains for natural deposits are well acknowledged (Burland (1989); Jardine (1995); Atkinson (2000)). Lightly- to heavily-cemented clean Touyora sand has a clear quasi-elastic boundary (Shibuya et al. (2001)). Some authors thus conclude that structured soils possess quasi-elasticity (e.g. Shibuya et al. (2001)). For clean lab sands, evidence also suggests existence of quasi-elasticity, although the threshold strain below which it operates is very small, being less than $10^{-5}$ to $10^{-6}$ strain (Chaudhary & Kuwano (2003); Jardine et al. (2001)). Thus it is important to know the strain level introduced by any dynamic tests so that its effect may be untangled from that of stress.

2.3.1.3 Stress-strain relationship

Historically, dynamic element testing techniques provided the bulk of the early small-strain data (Ishihara (1993); Ishihara (1996)). For example Hardin & Drnevich (1972), studied soil behaviour during simple shear using torsional shear equipment. They identified important factors influencing stiffness, being: strain amplitude, effective stress and void ratio, among others. They stated that: “$G_0$ varies with $p’$ to the power of 0.5. However, at large strain the modulus depends mainly on the strength of the soil,
which is more nearly a function of $p'$ to the first power. Thus, the power of $p'$ with which modulus varies increases from 0.5 at zero-strain amplitude to 1.0 at large strain amplitudes.” This view is now well established (Viggiani & Atkinson (1995); Jovicic & Coop (1997); Shibuya et al. (2001)).

Hardin & Drnevich (1972) stated that the concept of normalized strain and stress is natural and necessary: “A given strain does not have the same effect on all soils, nor on the same soil under different pressures.” Hardin & Kalinski (2005) performed tests using 12 different granular materials, demonstrating that, for a particular stress path, in this case simple shear at constant $p'$, the plot of normalized stiffness vs. normalized strain yields a single curve for a particular granular material, be it sand, silt, gravel or their mixtures; their results are reproduced in Figure 2.6 on page 34. In this figure the reference stiffness and strain are defined as: $G_{max} = G_0$ and $\gamma_r = \tau_{max}/G_0$ respectively and the stress-strain relationship is defined as:

$$\frac{G}{G_0} = \frac{1}{1 + \frac{\gamma_r}{a}[1 + b \exp(-b/\gamma_r)]},$$

where $a$ and $b$ are curve-fitting constants. Many practical stiffness-strain degradation models use a single equation for normalized stiffness vs normalized shear strain or stress (e.g. Fahey (1998)).

Since the developments of static element testing techniques capable of measuring small strains between $10^{-6}$ to $10^{-3}$ (e.g. Jardine (1992)) these dynamic studies were complimented by static tests exploring other important parameters such as anisotropy and rate/time effects. We now turn to these more recent developments.

### 2.3.2 Introduction to anisotropy

For both undisturbed and lab-prepared soil samples the inherent anisotropy due to deposition processes influences their yield-behaviour. Regarding this yielding behaviour, it is well known that in addition to $e$, $p'$ and $q$, $b$ and loading direction are also important, where: $e$ is the void ratio; $p'$ is the effective mean stress; $q$ is the shear stress; $b$ is a parameter that measures the relative magnitude of intermediate principle stress: $b = (\sigma_2' - \sigma_3')/(\sigma_1' - \sigma_3')$, where $\sigma_1'$, $\sigma_2'$ and $\sigma_3'$ are major, intermediate and minor principal stresses.

#### 2.3.2.1 Anisotropy beyond the quasi-elastic region in the dilative region of behaviour

Anisotropy can be attributed to the process of deposition and to the development of stress, for example due to the history of consolidation.
For the former, Chaudhary & Kuwano (2003) provides an excellent indication of the effect of loading direction on \(G_0\), G-degradation and failure, at fixed \(p'\) of 100kPa and with \(b = 0.5\) for a dense Toyoura sand. For samples initially isotropically consolidated, they find \(G_0\) decreasing by 30% when the loading direction changes from being perpendicular to the plane of deposition, to parallel with it; and the limit of the quasi-elastic behaviour varies from \(10^{-5}\) to \(3 \times 10^{-5}\) strain. Similarly at failure, \(q_{\text{max}}\) changes by up to 25%. The shapes of \(G_{\text{secant}}\) vs shear strain curves are almost the same for shearing along different stress paths.

For consolidation-induced anisotropy, Bellotti et al. (1996) quantified the variations of wave speed with transmission directions using different \(K\) values, where \(K = \frac{\sigma_2'}{\sigma_c'}\), is the coefficient of earth pressure. Figure 2.7 on page 35 reproduces some of their results for the case of \(K = 1\) and \(K = 2\). As shown in this figure, for isotropically consolidated \((K = 1)\) sample \(V_{p\theta=0} = V_{p\theta=90}\); and for consolidation with \(K = 2\) it can be seen that \(V_{p\theta=0} < V_{p\theta=90}\) and the sample is clearly anisotropic in terms of small strain stiffness.

### 2.3.2.2 Anisotropy towards failure in the contractive region of behaviour

The set of studies reviewed below are mainly undrained tests on loose to dense sands within the contractile phase of their behaviour, where octahedral shear strains are relatively small.

Symes et al. (1984) studied the effect of loading direction \(\alpha\), which is the direction of major principal stress relative to the direction of deposition, at constant \(b = 0.5\), where \(b = (\sigma_2' - \sigma_3')/(\sigma_1' - \sigma_3')\), using a medium loose Ham river sand. They constructed a 3D boundary surface in the \(p'-q-\alpha\) space. On this surface, as \(\alpha\) increases from 0 to 45 degrees, \(q_{\text{max}}\) may decrease by more than 50% at constant \(p'\). Further tests on loose Ham river sand and on glass ballotini in Shibuya & Hight (1987), again at constant \(b = 0.5\), confirmed the existence and usefulness of the state boundary surface in the \(p'-q-\alpha\) space. Symes et al. (1988) extended this surface to the 4D space of \(e-p'-q-\alpha\) and they visualized the \(p'-q-\alpha\) surfaces as constant-\(e\) surfaces. Shibuya et al. (2003) has repeated these previous studies with \(b = 0\) and \(b = 1\), and presented a complete state boundary surface in \(p'-q-\alpha-b\) space for medium dense Ham river sand. The effect of changing \(b\) from 0 to 1 can change \(q_{\text{max}}\) by up to 40%. Their results are reproduced in Figure 2.8 on page 36.

Lam & Tatsuoka (1988), using both triaxial and true triaxial tests, also studied the effect of \(b\) using very dense Toyoura sand by locating the 4D boundary surface in
The effect of changing $b$ from 0 to 1 can increase $q_{\text{max}}$ by more than 20%; whereas realigning the loading direction from parallel to the plane of deposition to perpendicular to it, can change $q_{\text{max}}$ by up to 40%.

The consolidation stress path can alter the inherent anisotropy, as already seen in the case of $G_0$. Its effect on the state boundary surfaces of loose Ham river sand is presented in Shibuya et al. (2003). The effect of changing $K$ from 1 to 0.5 is such that, $q_{\text{max}}$ may be changed by up to 40%, depending on $b$ value and loading direction.

Lade & Wang (2001) stated that the peak angle of friction when $b$ varies from 0 to 1 may be increased by 5 to 10 degrees. They believe if shear banding was prevented, the angle of friction is a maximum at a $b$ value of around 0.4 to 0.6. However plane-strain element tests are susceptible to shear banding prior to achieving the theoretical peak strength. This caveat applies to all preceding discussions.

In summary, the effect of anisotropy on stiffness and strength at small and large strains is complex, and can be substantial. Its influence on stiffness degradation behaviour can be estimated via Hardin’s reference shear strain $\gamma_r = 0.5q_{\text{max}}/G_{\text{max}}$ concept such that a change in $q_{\text{max}}$ will produce proportional change in $\gamma_r$ and will shift Hardin’s normalized stiffness degradation curves along the normalized strain axis by a proportional amount (for examples of Hardin’s degradation curves, please see Figure 2.6 on page 34). Its measurement in situ will require development of in situ static and dynamic techniques capable of complex stress path testing. This is unrealistic at present; it is still an active area of research outside the scope of this thesis.

### 2.3.3 In situ measurement of $G_0$ and stress-strain relationship

All of the above developments were due to advanced laboratory element testing; however, in order to take advantage of these theoretical developments for engineering design, suitable in situ testing methods must be employed (e.g. Schnaid (2005); Yu (2004); Fahey (1998)).

#### 2.3.3.1 Seismic methods

Schnaid (2005) gave a comprehensive review of the current repertoire of in situ testing techniques. Among them, geophysical tests, the seismic cone and the seismic dilatometer can all provide direct measurement of $G_0$, assuming direct conversion between wave velocity and $G_0$ once a material model is chosen, for example isotropic or anisotropic
models. Seismic push-in probes are convenient, and they also yield penetration resistance at the same time. These methods are necessarily restricted to simple and predefined stress paths and can only measure a small number of the parameters that have just been discussed.

Prominent examples are down-hole, cross-hole (Thomann & Hryciw (1992), seismic CPT (Schmaid (2005)) in situ; and bender elements (Fioravante (2000)), shear plates (Ismail et al. (2005)), geophones (Bellotti et al. (1996)) in the lab and in centrifuges (Fu et al. (2004)). The use of bender element arrays in the centrifuge is particularly interesting as illustrated in Figure 2.9 on page 37; a 2 dimensional ‘CT’ scan is possible with such an arrangement and eventually by expanding the 2 dimensional (2D) sensor-array to 3 dimensional (3D) one may be able to monitor the in situ changes continuously during a construction process. Most of these methods rely on the assumption of strain levels induced being within the quasi-elastic limit.

As already shown, seismic methods can be used to obtain stress-strain relationships when the dynamic strain level exceeds the bounds of the quasi-elastic region (Drnevich et al. (1995); Salgado et al. (1997a)). This approach is extensively developed in this thesis.

2.3.3.2 Pressuremeter

Self-boring pressuremeter tests (e.g. Fahey & Carter (1993)) can in theory provide both an estimate of $G_0$ and measurement of the stress-strain behaviour in one go; however uncertainty of probe insertion disturbance must be accounted for (e.g. Fahey & Randolph (1984)). It is a boundary value problem requiring back analysis; a soil behaviour model must be chosen first, rendering the process less straight-forward compared with seismic methods. $G_0$ should be confirmed independently using another method to lend confidence to the stress-strain relationship fitted to the pressuremeter curves. This approach was demonstrated in a series of publications: Fahey (1991), Fahey (1993), Fahey & Soliman (1994), Soliman & Fahey (1995) and Fahey (1998).

In this thesis the difficulties related to probe insertion are avoided by employing a wished-in (or ‘pre-placed’) pressuremeter on the centrifuge to measure static soil stress-strain behaviour.

2.3.3.3 Correlations with $q_c$

Indirect correlations are available between $q_c$, $G_0$ and sand state parameters, for example see Figure 2.10 on page 38, backed-up by FE results and semi-empirical formulations.
(Gaudin et al. (2005); Schnaid (2005)). This thesis provides additional data and support for such correlations, as seen in Figure 6.4 on page 192 where the lower bound for un-cemented soil following Schnaid (2005) is also plotted and the horizontal axis is a normalized CPT cone resistance: $q_{c1} = (q_c/p_a)\sqrt{(p_a/\sigma''_v)}$ where $p_a$ is atmospheric pressure and $\sigma''_v$ is the effective in situ vertical stress at the cone tip location prior to cone insertion. However as a predictive method, it is too imprecise to be of practical use (Schnaid & Yu (2007)), for example consider the large scatter of field in situ data in Figure 2.10 as labeled ‘J-pit’ and ‘Mildred Lake’.

### 2.3.4 Limitations

In situ field tests are boundary value problems whose interpretation requires assumptions that one can avoid in lab element tests. They are generally limited to exploring only a number of important soil properties along restricted stress paths (e.g. Schnaid (2005)). Therefore the development of more advanced numerical models based on many soil parameters characterizing many aspects of soil fabric would not be directly useful for in situ oriented engineering design at the present.

The route taken by this thesis is to target a particular representative soil stress path, using simple 1-curve stress-strain models, ignoring the effect of anisotropy wherever possible and ignoring rate/time effects all-together, and limiting discussions to clean, normally consolidated, un-aged sand. The result is a simplified approach yielding results immediately applicable to engineering design accepting some limiting assumptions.
2.4 On deep penetrating probes

Deep penetrating probes, e.g. CPTs, cone pressuremeters, pressed-in piles or driven piles, introduce great ranges of stress and strain into the surrounding soil, spanning from in situ stress right down to the normal compression line; therefore they can provide additional information which the seismic methods cannot. This is valuable especially when the design of displacement piles is in question.

White (2002) gave a thorough review of the subject. It is concluded that reliable measurement of piling-induced stresses, in controlled environments and under realistic conditions, have been lacking. This study attempts to provide new data of in situ stress during and after pile jacking.

Deep penetration is a complex boundary value problem that involves, according to traditional categories, the base, the shaft and their interaction. These different aspects of the phenomenon are reviewed in the following sections.

2.4.1 Probe size effect on $q_c$

Whether a smaller CPT cone size or different tip geometry compared to a pile makes any difference on the ultimate bearing capacity are central questions to the application of CPT-based pressed-in pile design.

2.4.1.1 Early literature evidence

Sanglerat (1972) provided a comprehensive review of pile size effect. This section will summarize his observations.

As reviewed by Sanglerat (1972), Kerisel (1961)’s data seems to indicate little size effect on $q_c$, and he concluded that pile diameter has no influence on $q_c$, if $q_c$ is less than 20MPa. Supporting evidence from other authors was also quoted.

As reviewed by Sanglerat (1972), DeBeer (1963) believed there is no size effect on $q_c$ once deep penetration is established and pointed out that a typical pile may need to be driven to a distance more than 20 times its diameter into a dense sand layer in order to achieve deep penetration; and in practice piles are only driven 2 to 3m into the bearing stratum and thus explained Kerisel’s 20MPa limit (Kerisel (1961)) as a result of piles not being driven deep enough.

As discussed by Sanglerat (1972), Kerisel agreed with De Beer on the requirement of certain critical embedment to achieve deep penetration, and drew parallels to the observations during deep punching of metal blocks (Sanglerat (1972) p152 and 153).
De Beer’s view was also supported by the Delft experience, again quoted by Sanglerat, that to use the value of $q_c$ directly in design, the embedment of a pile into the bearing stratum had to be substantial (Sanglerat (1972) p159).

In short the experience with CPT and driven piles in sand prior to 1972 is that scale effect on $q_c$ does not exist or is quite small, provided adequate embedment guaranteeing deep penetration.

### 2.4.1.2 Recent literature evidence

Jardine et al. (1993) observed a pile size effect by analyzing a large data base of 31 compression tests of 65 piles of different diameters ranging from 0.1 to 2m, giving

$$q_b = q_c \left(0.5 - 0.25 \log \left( \frac{D}{D_{CPT}} \right) \right),$$

where $D$ is the pile diameter and $D_{CPT}$ is the CPT cone’s diameter. This same data base was re-examined in White & Bolton (2005) where careful distinction of partial mobilization and partial embedment, coupled with using only test sites where CPT data is available, reduced the size of the database to 20 and led to a conclusion that $q_b/q_c$ is almost always close to 1 (or 0.9, quoted from White & Bolton (2005); as shown in Figure 2.11 on page 38.

The view that no observable size-effect exists also prevails amongst practitioners. Eslami & Fellenius (1997) reviewed 102 case histories, employing 5 current driven pile design methods and proposed one of their own. They concluded that equating $q_c$ to $q_b$, provided one takes steps to average out the peaks and troughs, gave excellent prediction of measured pile capacity, to within 20%, in widely different soil conditions. Their results are reproduced in Figure 2.12 on page 39. This is truly remarkable, considering the usual practice of using Factors-of-Safety greater than 2 in pile design.

### 2.4.1.3 $q_c$ and $q_b$

The above literature evidence strongly suggests the equivalence of the two and their independence from the usually tip geometries adopted by the industry (e.g. flat tip; conical cone tip of different apex angle; pyramid with different apex angle, sometimes used in precast driven piles). However the difference in pile shape (square vs circle) and in tip geometry (cone vs flat) must be reconciled, for complete confidence.

Allersma (1987) measured the stress distributions due to penetration by probes of different tip geometries under plane-strain boundary conditions using a photo-elasticity method. The stress levels involved in the tests were much less than for an in situ probe, due to equipment limitations, and particle-crushing was not modeled. However his
results provide a rare glimpse of the principal stresses introduced during penetration; they are reproduced in Figure 2.13 on page 40. From qualitative agreement between all three tip geometries, it is apparent that the major principal stress points radially outwards, giving support to a spherical cavity expansion mechanism in the far-field; however the mechanism close to the tip surface is difficult to observe and is outside the scope of this thesis.

Therefore, a cavity expansion mechanism appears to be applicable to all types of tip geometries in the intermediate and far field; this observation must be behind the apparent equality between $q_b$ and $q_c$.

2.4.2 Predicting $q_c$ and $q_b$

Yu (2004) has given a comprehensive review on this topic. Konrad (1998) employing a two part linear critical state line and the state parameter, demonstrated a direct correlation between normalized $q_c$ and a normalized state parameter as defined in Figure 2.14 on page 41. Similarly Klotz & Coop (2001) demonstrated a similar correlation with bearing capacity factor which is a function of $q_c$ and in situ stress for a flat-tipped displacement pile as defined and plotted in Figure 2.15 on page 42. These methods are of immense theoretical value; however their application requires knowledge of the critical state line, which is usually not available in the field.

Salgado et al. (1997a) has demonstrated that a cavity expansion method coupled with a simple stress fan, could predict calibration chamber results to within +/-30%, demonstrated in Figure 2.16 on page 43. However the cone penetration mechanism in the field can be highly influenced by local variations of strength, given the relatively small size of the cone, rendering theoretical prediction perilous.

The various correlations between $q_c$ and other soil parameters should be used to predict these parameters once a CPT test has been carried out, instead of being used to predict $q_c$. This opinion is implicit in current state-of-the-art reviews for sand, e.g. Yu (2004), Schnaid (2005).

Given the difficulty in predicting $q_c$ in the field and the widespread application of CPT in routine soil investigations, one could always assume the availability of CPT profiles and should concentrate on the prediction of pile performance, taking $q_c = q_b$ for granted.

However one aspect yet unresolved is pile shaft and base interaction, which is intimately linked to the pile diameter effect. Randolph (2003) acknowledged that: “the
effect of pile diameter on design end-bearing resistance is an area of apparent divergence between science and empiricism, which needs to be resolved."

We will review our current understanding of pile shaft behaviour in Section 2.4.3 and present evidence of shaft upon base interaction in Section 2.4.4; and demonstrate that interaction exists but it is small enough to allow the pile base and shaft to be treated independently for the purpose of this thesis.

2.4.2.1 Pile base load settlement prediction

Methods for predicting bored pile base settlement behaviour are relatively well established. Lee & Salgado (2005) demonstrated the usage of Finite Element (FE) implementing non-linear elastic Mohr-Coulomb plastic model in modeling both a bored pile and a circular footing. Their results and field validations are reproduced in Figure 2.17 on page 44. However FE is unlikely to become a routine design option at the present.

Yasufuku et al. (2001) presented a semi-empirical method, based on spherical cavity expansion and a shearing mechanism below the pile tip, to predict bored pile base stress at a large displacement (of one pile diameter). The load-settlement curve is then constructed assuming a simple parabolic relationship. The method is illustrated in Figure 2.18 on page 45. This type of approach is adopted in this thesis for displacement piles.

2.4.3 Pile shaft and sand interaction

The following discussion concentrates on the fundamental shear interaction behaviour between sand and a flat surface as studied using shear box equipment. This knowledge is a prerequisite for understanding the curved interface of pile shaft and sand.

2.4.3.1 Overview

The landmark papers of Uesugi et al. (1988), Uesugi et al. (1989) and Poulos (1989) discovered many facets of sand-structure interface behaviour under 2D monotonic, one-way and two-way cyclic movements. Since then, more advanced equipment 1 and measuring methods 2 confirmed these observations and provided very valuable information on micro-mechanisms. The following discussion will be biased towards a rough surface and dense sand, and a summary of its content is given below:

---

13D simple shear box used in Evgin & Fakharian (1996)
2e.g. digital imaging used in DeJong et al. (2003), Hu & Pu (2003)
Soil-structure interface friction follows the empirical Amontons’s Law:

\[ \text{Friction} = \text{Normal Stress} \times \text{Coefficient Of Friction} \]

For the case of dense sand and a rough solid surface, after a sufficient amount of total slippage, a peak angle of friction is reached; then a shear band starts to strain-soften, the friction angle begins to fall until reaching a “critical state” with constant friction angle and shear band fully formed, exhibiting zero dilation; this describes the process of “friction angle degradation”. Experimental evidence suggests a good correlation between total resultant slippage and friction angle degradation; an apparently unique correlation between total particle breakage and total plastic work due to slippage. The governing mechanism of shear band formation is still an active area of research.

2.4.3.2 Survey of past experimental work

The Early Days: A more scientific understanding of solid friction is a recent affair. The landmark is the book of Bowden & Tabor (1950). They showed that the empirical Amontons’s Law (discovered in 1699 France), i.e. existence of coefficient of friction, is truly empirical; its roots lie in the shear strength of junctions between surfaces (metal and nonmetal); its theoretical justification is very complicated for realistic scenarios; however Amontons’s Law appears to be valid for all surfaces under common conditions on earth. This is also true for soil structure interfaces as shown by Lehane et al. (1993). The difficulty here lies with the changing values of the angle of friction with shear deformation.

The study of soil-structure friction follows closely behind that of continuum solids. Potyondy (1961) is one of the first to initiate a systematic scientific examination of factors affecting the angle of friction. A comprehensive group of data highlighted the important balance of clay and sand and other soil fractions, with peak angle of friction decreasing very quickly once the clay fraction exceeds 15% (for a well graded sand matrix of \( D_{50} \approx 0.6mm \); \( D_{50} \) is the sieve opening size below which 50% by mass of the sample passes during a standard particle grading test). He identified four major factors affecting skin friction: the moisture content of soils; the roughness of surface; the composition of soils; and the intensity of normal stress.

Kishida & Uesugi (1987) developed a simple shear interface (SSI) testing apparatus. They found that the simple shear apparatus is suitable because it can measure interface slippage (please refer to Figure 2.19 on page 46 for a definition of interface slippage).
independently from the bulk shear deformation. They proposed and demonstrated the usefulness of a normalized measure of surface roughness: $R_n = \frac{R_t}{D_{mm}}$; please refer to Figure 2.20 on page 46 for a definition of relative roughness.

Uesugi et al. (1988) and Uesugi et al. (1989) fitted a transparent window to their SSI which allowed sand particles to be tracked individually during monotonic shearing. It was found for a rough surface that the sand mass will deform and dilate uniformly until peak friction is attained. Post peak, the friction angle degrades as shear deformation localizes inside a narrow band. Within the band the particles move randomly; eventually zero dilation is established with a constant angle of friction. The vital role of the shear zone is identified. They also found that when the interface is smooth, the coefficient of friction increases depending on the degree of particle crushing due to changes of $R_n$. One of their most interesting discoveries was that the change of the coefficient of friction with cumulative slippage during monotonic loading or one-way cyclic shear or two-way cyclic shear is very similar and the behaviour under monotonic loading forms the envelope of the others.

Jardine et al. (1993) summarized shear box results of 12 different granular soils and concluded that the critical state interface friction angle is independent of relative density; it increases linearly with normalized roughness until a threshold equal to the soil’s critical state friction angle; tentatively it also reduces with increasing normal stress level.

Tabucanon et al. (1995) conducted cyclic direct shear box tests at constant normal stiffness of sand-on-sand and sand-on-metal interfaces. They observed a sudden change in post-cyclic monotonic shear behaviour if the cyclic amplitude exceeded a threshold for a prescribed number of cycles. They concluded that during cycling the normal and shear stress can drop to values close to zero, but on re-shearing to failure significant stress recovery can occur due to dilation, provided the shear zone is not fully formed during the cycling process.

**2.4.3.3 Survey of more recent experimental works**

Evgin & Fakharian (1996) developed a 3D simple shear apparatus. They concluded that the magnitude of peak and residual friction angles are independent of stress paths and the peak friction angle is dependent on the normal stress measured at the time of peak stress ratio.
Evgin & Fakharian (1997) performed cyclic constant normal stiffness tests with a rough interface ($R_n = 40 \times 10^{-3}$) using a normal stiffness of 400kPa/mm and initial normal stress of 300kPa. By plotting the normal and shear stresses after 50 cycles against the cycling amplitudes, they noticed a sudden and large dip in shear stress during the test with 0.75mm cyclic amplitude; this was attributed to the failure of the interface at the 11th cycle - when the peak friction angle was mobilized. This result agrees with Tabucanon et al. (1995).

Evgin & Fakharian (1998) extended their 1996 3D monotonic shear tests by performing constant normal load 3D cyclic testing, following elliptical displacement paths. They reaffirmed the previous findings about a unique correlation between slippage and the interface friction angle, for example in Uesugi et al. (1988) and Uesugi et al. (1989).

2.4.3.4 Inside the shear band mechanism

DeJong et al. (2003) used digital image analysis on a direct shear box, giving them the ability to monitor the shear zone independently from the sand mass. Shear zone thickness was identified to be influenced by particle grading and shape, with a thicker zone for an angular well-graded sand (calcareous) compared to a sub-rounded uniform one (silica). Using a similar technique, Hu & Pu (2003) found that the shear zone thickness was influenced by relative roughness, with a thicker zone for a rougher surface.

These observations seem to agree with observations on sand-sand shear bands e.g. Oda and Kazama’s excellent paper (Oda & Kazama (1998)) in which they wrote: “in a strain-hardening process of a granular soil, the main micro-structural change is the setting-up of columns parallel to the major principal stress direction. The columns start buckling at failure (peak stress), and the buckling columns gradually concentrate to shear bands in a strain-softening process, which causes the growth of large voids and particle rotation.” They further postulated that the controlling factors are density, rotational resistance at contacts and mean pressure, from the analogy to the buckling theory for an elastic column. Many of these factors are also important in the interface shearing problem and the rotational resistance may be strongly influenced by particle damage.

Zeghal & Edil (2002) reanalyzed data of N. Hoteit and demonstrated that the cumulative plastic work correlated well with particle breakage. Since plastic work is

\[1\] These values are within range for large-diameter bored piles in sand

\[2\] Where 1.4mm monotonic displacement is required to mobilize peak friction for this pair of interfaces
due to slippage, they postulated a fundamental role of particle breakage on shear zone formation.

2.4.3.5 Constitutive modeling of an interface

Gomez et al. (1999) presented a concise survey of literature prior to 1998 on the subject. They concluded that the Clough and Duncan hyperbolic model developed in 1971 had been most widely used due to its simplicity; however it took no account of the actual mechanism.

ElSakhawy & Edil (1996) used an elasto-plastic model to fit axis-symmetric rod pull-out test data; the parameters have little physical meaning. Ghionna & Mortara (2002) presented another elasto-plastic model to fit constant normal load (CNL) direct shear box data; the parameters again have little physical meaning.

Hu & Pu (2003) developed a damage model based constitutive relationship. The idea is based on the disturbed state concept proposed by Desai in 1992 and they summarized its essentials as: “during deformation, a material element can be considered to be a mixture of the material in two reference states: the intact state and the critical state. The micro-structural changes result in a transition of the intact state into the critical state”. The idea of this damage model may be appealing since it mirrors the growth of the interface shear band. However its parameters lacked physical meaning.

Zeghal & Edil (2002) developed an elasto-plastic model with slippage plastic work in mind. They idealized the shear band dilation by replacing the structural surface with a sinusoidal wave. Their model required parameters of soil stiffness, empirical grain breakage parameters and geometrical parameters for the “equivalent” sinusoidal surface. The model parameters require advanced laboratory testing and its assumptions are semi-empirical.

2.4.3.6 Physical modeling of a rough interface

Irsyam & Hryciw (1991) stated that soil-structure interface interaction may have contributions from both friction and passive resistance. Working with tall rectangular ribs glued on flat surface at different spacings, they demonstrated the interplay of the two factors. They observed that for small rib spacing ($< 2H$)\(^1\) the failure surface approached a plane parallel to the plate with friction being the only resistance, and

\(^1\)\(H\) is the height of the asperities, measured from base to tip.
for large spacing ($\sim 6H$) the failure surface exhibited a pronounced curvature and the passive resistance began to dominate.

Complementing Irsyam and Hryciw’s work, Dove & Jarrett (2002) examined in more detail the effect of surface topology of a triangular ribbed surface. They concluded that the asperity height, spacing and angle control the behaviour of the interface and recommended that to achieve 100% interface efficiency (i.e. an interface as strong as unbroken soil), one needs an asperity height $H = D_{50}$, an asperity shape of an equilateral triangle and an asperity spacing of between 1 and $3 \times D_{50}$. This approach of using regular asperities to model a rough surface is adopted in this thesis.

2.4.3.7 Summary

It is clear that the coil mechanics community does not possess the insight to model shaft-soil interface dilation and slip behaviour using fundamental soil grain properties. Since the shear band dilation is a crucial boundary condition for a cylindrical cavity of the pile body, if one chooses to visualize the problem as such, then one can not relate pile movement to the in situ stress changes due to cavity expansion or contraction at the present. The interplay between slippage, stress and friction angle is complicated for a flat interface and the interaction becomes even more complicated for a curved interface, where the dimension of the structure becomes important too and the scaling rule for centrifuge model piles to prototype scales is difficult to predict for shaft load-settlement behaviour; these aspects will be discussed further in later part of this thesis.

Therefore any discussion on shaft base interaction will necessarily stay qualitative and based on the strength of experimental measurements, unless a full suite of interface shear tests is performed to allow proper analysis of pile shaft settlement data. This is discussed in more detail in section 6.4.

2.4.4 Pile shaft upon base interaction

Early direct evidence of the interaction (Sanglerat (1972)), consistently observed at 5 sandy sites, came as the difference between two classic Dutch (Delft) Cones (36mm diameter) being advanced by two different procedures: in one the sleeve is driven together with the tip, in the other the sleeve is kept stationary; the cone tip resistance is somewhat higher (10 to 20%) when both the sleeve and tip were driven together, indicating the existence of a positive benefit of shaft friction upon base resistance. According to
Sanglerat (1972), Raedschelders suggested this is due to increased overburden by the down-drag of shaft friction.

Borghi et al. (2001) made similar observations when they performed centrifuge experiments of a similar nature. Two piles identical in dimensions but with different surface roughness were pressed into dense sand. However, due to the limited capacity of the piling actuator, the rough piles were only installed to a shallow depth, almost certainly above the critical depth required to achieve deep penetration. Hence the result is more akin to a shallow footing than deep penetrating probes, and the result is inconclusive.

This thesis adopts a similar modeling approach as Borghi et al. (2001) using centrifuge model piles with different surface roughness. And the difference in $q_b$ between a smooth and a rough shaft is shown to be 12% in a very dense sand (see Figure 5.1 on page 144). Keeping this 12% influence in mind, this thesis adopts the conventional approach that pile shaft and base load-settlement behaviour are treated separately and their interaction not modeled.
Figure 2.1: An active total earth pressure cell; the cross sectional and plan view of the instrument; and the active computer feed back air pressure control system that is used to re-zero the diaphragm deformation induced by a change of the measured pressure; this figure is modified from Talesnick (2005)
Figure 2.2: Concept of large strain cross-hole seismic method; showing the physical arrangement of the sensors. $V_s$ is the shear wave speed as calculated from time-of-travel between adjacent pairs of sensors; and typical source signal generated by the hammer-anchor impact and recorded signals by the sensors. The time delay between arrivals and the reduced signal strength at different sensors can be see; this figure is modified from Salgado et al. (1997a)
Figure 2.3: Typical analysis results of the large strain cross-hole seismic signals of Figure 2.2, page 30; showing travel time or the time of arrival of the source signal at different sensors. $V_{max}$ can be obtained from the gradient of the linear part of this plot; curve fitting result which generates two possible fit to raw data; this figure is modified from Salgado et al. (1997a)
Figure 2.4: Theoretical prediction of the effect of void ratio on $G_0$; the Effective medium shear modulus is the small strain stiffness $G_0$ or $G_{max}$; the solid lines are theoretical results making use of two different micro-macro homogenization assumptions: Kinematic hypothesis requires that $u_i = \epsilon_{ij} L_j$, where $u_i$ is the displacement vector between two particle centers of particle $i$ and $j$, $\epsilon_{ij}$ is the continuum strain tensor and $L_j$ is the distance vector linking the centers of the particles; the Static hypothesis is an alternative to the simper Kinematic hypothesis, its details can be found in Liao et al. (2000); this figure is modified from Liao et al. (2000)
Figure 2.5: Normalized stiffness following critical state soil mechanics concepts for Dog’s Bay sand, decomposed granite and Ham river sand; NCL stands for Normal Compression Line, CSL stands for Critical State Line; $G_0$ is the small strain stiffness, $G_{0(nc)}$ is the small strain stiffness of a soil element whose current state lies on the NCL; $p'_e$ is a reference pressure as defined in this figure; the power-scaling relationship between small strain stiffness and stress is also illustrated, where $p'$ is the confining stress and $p_a$ is the atmospheric pressure; this figure is modified from Coop (1999)
Figure 2.6: Normalized stiffness degradation curves obtained from resonant column tests for different granular materials of different gradings; the horizontal axis is normalized shear strain by a reference strain, $\gamma/\gamma_r$, where $\gamma_r$ is defined as $\gamma_r = \tau_{\text{max}}/G_{\text{max}}$ and $\tau_{\text{max}} = 0.5\sqrt{(\sigma'_1 + \sigma'_3)^2 \sin \phi'_{\text{peak}}} - (\sigma'_1 - \sigma'_3)^2$; the vertical axis is secant shear stiffness, $G$, normalized by the small strain stiffness $G_{\text{max}}$, $G/G_{\text{max}}$; the corresponding particle grading curves are shown in the top righthand corner of each plot; $e$ is the void ratio, $\phi'_{\text{peak}}$ is the peak angle of friction, $S$ is the saturation and $W$ is the water content of the sample; $a$ and $b$ are curve fitting constants defined as: $G/G_{\text{max}} = \frac{1}{1 + \gamma/\gamma_r[1 + a \exp(-b\gamma/\gamma_r)]}$; this figure is modified from Hardin & Kalinski (2005).
Figure 2.7: Anisotropy of small strain stiffness due to consolidation; on the left hand side of this figure the sample is isotropic-consolidated with the earth pressure coefficient, $K = 1$; on the right hand side of this figure the sample is anisotropic-consolidated with $K = 2$. $V_pθ$ is the propagation velocity of a compression wave traveling along an axis rotated by an angle $θ$ away from the vertical axis of the sample; this figure is modified from Bellotti et al. (1996)
Figure 2.8: State or local boundary surfaces of isotropically consolidated loose Ham river sand, constructed from undrained triaxial compression / extension tests; where the z-axis is \( t = q/2 \), the x-axis is \( p' \) which is the mean confining pressure \( ((\sigma'_{1} + \sigma'_{2} + \sigma'_{3})/3) \) and y-axis is \( \alpha \) which is the angle between the major principle stress direction and the vertical axis of the sample measured in degrees; \( b = (\sigma'_{2} - \sigma'_{3})/(\sigma'_{1} - \sigma'_{3}) \); this figure is modified from Shibuya et al. (2003)
Figure 2.9: Bender elements employed in a centrifuge; the schematic diagram of the centrifuge, and the Laminar box with bender element array are shown; three transmitters and three receivers are mounted on to oppositely placed vertical columns such that any transmitter's signal can be picked up by all three receivers on the opposite side and they results can be used to produce a 2 dimensional 'CT' scan of the soil volume situated between the transmitters and receivers; this figure is modified from Fu et al. (2004)
Figure 2.10: Correlations of \( q_c \) with \( G_0 \); \( q_c \) is the cone penetration resistance; the state parameter, \( \psi \) is defined as the difference in void ratio, \( e \), between a soil element at a current state of \((e, p')\) and another on the critical state line with a state of \((e_{\text{crit}}, p')\), hence \( \psi = e - e_{\text{crit}} \); the solid symbols are lab test results of different type of sands under the confining pressure of \( P = 50\,\text{kPa} \) or \( P = 500\,\text{kPa} \), the type of sands are indicated in the Key of that figure. ”J-Pit” and ”Mildred Lake” data are from in situ field tests, their data shows considerable scatter compared with the lab results; this figure is modified from Schnaid (2005).

Figure 2.11: CPT vs pile base resistance; \( q_b \) is the pile base resistance measured at ”plunging failure” and \( q_c \) is the cone penetration resistance; different symbols represent different pile test sites and they are identified in the Key box of the figure; this figure is modified from White & Bolton (2005).
Figure 2.12: CPT-based pile base capacity design methods; 5 different methods are shown and indicated in the top left-hand corner of each plot; the dashed lines indicate a $\pm$ 20% error of estimated pile capacity with respect to the measured value; this figure is modified from Eslami & Fellenius (1997).
Figure 2.13: Principal stress directions below penetrating probes of different tip geometries, as shown in the individual plots; this figure is modified from Allersma (1987)

Computer plot of the principal stress trajectories

(1 scale = 1cm)
Figure 2.14: State parameter and CPT resistance; the parameters are defined in the top half of this figure; the solid lines represent a bi-linear critical state line where C is a turning point at which $p' = p_c$ for a soil element with initial state at point O; $p'_c$ and $\psi$ are defined as shown in this figure; $e_{min}$ and $e_{max}$ are minimum and maximum void ratio of the soil; in the bottom half of this figure $q_c$ is the cone penetration resistance and the normalized state parameter, $\psi_N$ is defined as: $\psi_N = \psi / (e_{max} - e_{min})$; the solid symbols represents cone penetration tests carried out in different sands, their type are shown in the Key of this plot; this figure is modified from Konrad (1998)
Figure 2.15: State parameter-based pile capacity design method; the parameters: $p'_0$ and $p'_{cs}$ are defined for a soil element at its current state as indicated in the right hand side plot; the vertical axis of the right side plot is $v$, which is the specific volume; and the solid curve is the Critical State Line (CLS); $N_q$ is the bearing capacity factor and is defined as: $N_q = q_b/\sigma_v'$ and $R_s = p'_0/p'_{cs}$ is a state parameter; the symbols represent centrifuge pile penetration data and the solid line is a best fit trend of the centrifuge data; this figure is modified from Klotz & Coop (2001)
Figure 2.16: Cylindrical cavity expansion modeling of CPT penetration; the geometry of the CPT cone tip is represented by the triangular area Q; ABC represents a slip-fan, across which the major principle stress direction rotates towards horizontal and it value equals the in situ horizontal stress, $\sigma'_r$; in bottom half of the figure the symbols represents calibration chamber CPT penetration test data with different boundary conditions where BC1 represents constant vertical and lateral confining stresses and BC4 represents zero vertical boundary displacements and constant lateral confining stress; this figure is modified from Salgado et al. (1997b)
Figure 2.17: FE predictions of circular footing settlement; following Lee & Salgado (2005)
Figure 2.18: Spherical cavity expansion modeling of bored pile base capacity mobilization; the important parameters are defined in the figure; however the model is complex and for other details of the model please refer to Yasufuku et al. (2001); this figure is modified from Yasufuku et al. (2001)
Figure 2.19: Interface shear box; Uesugi et al. (1988)

Definition of relative roughness $R_n$

Figure 2.20: Definition of relative roughness, $R_n$; Fioravante (2002)
Chapter 3

Triaxial testing
3.1 Introduction

The triaxial test series was aimed at quantifying static soil stiffness and its non-linearities with strain level for relevant simple stress paths, using samples that were prepared in the same way as the centrifuge model. The results will be used in later sections for comparison with centrifuge in situ soil testing techniques and in back analysis of centrifuge piling data.

The soil testing procedures and apparatus are described first, followed by the results. All the tests were performed in the Geotechnical Research Laboratory of the Department of Engineering at Cambridge.

The tests were conducted using dry Fraction E Leighton Buzzard silica sand (referred to as Fraction E sand) with local and external strain instruments; hence all stresses presented herein are effective stresses. The sand had the following properties: $D_{10} = 0.067$ to $0.095$mm, $D_{50} = 0.12$mm, $e_{max} = 1.014$, $e_{min} = 0.613$ and $G_s = 2.65$ which is the specific gravity of the sand; following Coelho (2007); it is produced by David Ball Group plc and conform to BS-EN-ISO-9002:1994. Tan (1990) has also determined $D_{10} = 0.095$, $D_{50} = 0.14$mm, $e_{max} = 1.014$, $e_{min} = 0.613$ with $G_s = 2.65$.

The triaxial tests were aimed at producing data from which a suitable material model could be derived so that its predictions could be used as a basis for comparison with other experimental methods employed in this research. More specifically, it was required to obtain values of:

- Very small strain stiffness.
- Stiffness reduction with strain for conventional stress paths of isotropic consolidation followed by standard triaxial compression or constant $p'$ compression.

3.2 Apparatus, sample preparation and testing procedures

A GDS Instruments Ltd. triaxial system was used (GDS Instruments Ltd). A schematic diagram and a photograph of such a system are given in Figure 3.1 on page 66 and Figure 3.2 on page 67. It consisted of:

- Classic Bishop and Wesley hydraulic stress path cell (Wykeham Farrance, UK), equipped with one internal load cell and one external LVDT for measuring axial load and displacement.
• Three digital pressure controllers (GDS Instruments Limited) for controlling the cell, back and ram pressures independently.

• Two local axial strain LVDTs and their mountings; one radial LVDT and a radial strain belt (GDS Instruments Limited).

• Data acquisition systems capable of a maximum logging frequency of 1Hz (which will limit its applicability in dynamic tests but proved sufficient for static, low loading rate tests discussed herein) and its computer control software which is capable of performing a number of factory-defined stress path tests (GDS Instrument Limited).

The triaxial testing techniques adopted are best described by the following bullet points:

• Multiple sieves sand raining for dry pluviation of sand.

• Standard sample ends.

• Axial LVDTs glued directly onto the external surface of the rubber membrane.

• Modified pin-through connections for the radial LVDT belt.

• “Knob-on-plate” docking.

Careful triaxial sample preparation technique is critical to the success of the test. The procedures are now described in detail.

3.2.1 Sample former

A picture of the bottom platen before sample preparation is shown in Figure 3.3 (a) on page 68. A spirit level was first used to check that it was perfectly horizontal. A semi-transparent latex rubber membrane was wrapped around it and a filter paper grid was wrapped around the membrane before a Perspex split former was put in place around the membrane as shown in Figure 3.3 (b). Sets of horizontal marks at 5mm spacing were drawn onto the inside face of the split former to facilitate sand pluviation. The air gap between the membrane and the inside face of the split former was evacuated using a vacuum pump via a port opening through the former, as shown in Figure 3.3 (c). The filter paper grid was to facilitate this vacuum operation.
3.2.2 Sand pluviation

Dry pluviation of sand is a standard technique employed by many laboratories to prepare model tests. At Cambridge, an automatic sand pourer is widely used to prepare centrifuge models. A detailed description of this particular system can be found in Zhao et al. (2006a). They concluded that sand flow rate is the strongest controlling factor on density, while drop height and sample dimensions, or container boundary, have secondary effects.

It is also widely acknowledged that the sample preparation technique has important implications on the inherent anisotropic behaviour of the sample (e.g. Miura & Toki (1982)). Therefore it is necessary to adopt the same sample preparation technique for centrifuge and triaxial tests, in order to make valid comparisons of their results. With this aim in mind, a new manual triaxial sand pourer was developed for this project to replicate the essential details of the automatic sand pourer which was used during the centrifuge model preparation.

3.2.2.1 The Triaxial Sand Pourer

The Triaxial Sand Pourer is seen in operation in Figure 3.4 on page 69, and a schematic diagram of its set up is shown in Figure 3.5 on page 70. It consisted of a sand container, analogous to the sand hopper of the Automatic Sand Pourer. Sand was stored in it and discharged through a 2mm diameter nozzle; this nozzle diameter could be varied to alter the flow rate so as to achieve different resultant relative densities (Zhao et al. (2006a) and Miura & Toki (1982)). Due to the limited scope of this test series, only the 2mm diameter nozzle was used to create a dense sand sample in a manageable preparation time.

This single jet of sand was then passed through a stack of 17 sieves of 2mm-grid-size, placed 1cm apart, forming a cylinder of 50mm diameter. Upon exiting the sieve stack, the single jet was uniformly spread-out into a circular shape of 50mm diameter. It then fell under gravity into the sample former.

The whole assembly sat on pivoted supports, allowing free motion in the horizontal plane while the drop height was maintained. The distance between the last sieve and the top surface of the bottom platen was approximately 220mm and this was fixed during the sand pouring. The drop height therefore varied linearly from 220mm to 110mm as the sample was gradually formed. The precise effect on local density variation within the sample due to this change in drop height was not investigated. A 5% increase in
relative density when doubling the drop height was observed using the Automatic Sand Pourer (Zhao et al. (2006a)).

During sand pluviation the flatness of the sand surface was ensured by constantly adjusting the horizontal position of the pourer while monitoring local surface elevations against horizontally drawn marks on the split sample former. A strong light was used to shine through the membrane to facilitate observation, as indicated in Figure 3.5 on page 70. Constant adjustment of the sand pourer position was required to ensure flatness of the sand surface, even though the flow exiting the sieves was uniform; this is partly attributed to static electricity generation, causing unpredictable micro-turbulence in the sand flow and partly to the inherent turbulent nature of the air flow.

Using the 2mm nozzle, the total sand flow rate was kept at approximately 0.02kg per minute and the pluviation was always completed in one attempt lasting up to 40 minutes. Between 0.6 and 0.8 kg of sand was used in total, i.e. almost half was poured outside the sample former due to the adjustment required to keep the sand surface flat.

3.2.2.2 Density of the samples

Calibration of sand density was carried out using a dummy sample former; see Figure 3.5 on page 70. A sharp knife edge was machined on its upper edge to prevent piling up of sand on its upper fringe. Its volume was obtained by filling it with water and measuring the weight; this volume was used for density calculations. Three tests were conducted using the same settings as the triaxial tests. The densities obtained were 1550.1 kg/m$^3$, 1539.8 kg/m$^3$ and 1545.7 kg/m$^3$, giving an average of 1545 kg/m$^3$, or 70% relative density, and a maximum scatter of 3% relative density.

3.2.3 Friction ends

The triaxial test is traditionally considered an element test—by this it is required that the stress and strain to be uniform throughout the entire sample. However, sample bulging and formation of shear planes are clear signs of non-uniformity.

Prominent technical solutions to reduce bulging include greased ends and oversized platens. More recently, especially with the advance of CT (computed tomography) scanning techniques, it is possible to continuously monitor the change in uniformity across the entire sample and to observe shear bands. It is now quite well established that sand specimens in standard triaxial compression tests, i.e. without greased ends
etc, do bulge more or less uniformly in the vicinity of peak stress and only develop complex multiple shear bands at higher axial strains (Alshibli et al. (2004)). Theoretical support for this post-peak strain localization is available (Lade & Wang (2001); Wang & Lade (2001)).

Standard end platens were used in this test series. The problem of not using greased ends and enlarged platens was the visible bulging of the sample, most pronounced at the mid-height while being restricted to zero at the platens; the direct effect was a proportional difference in stresses. Since the stiffness data to be derived from the local strain instruments in use for this thesis are restricted to radial strain levels of less than 2% due to physical limitations of the local instruments, non-uniformity of the stress field is expected to be no more than 4%.

3.2.4 Placing of the top cap and applying a suction

To avoid bedding errors the following procedure was used, also illustrated in Figure 3.6 on page 71. The sample former was firstly overfilled before the top cap was pushed across the opening of the sample former, scraping it flat, while being guided by a spirit level (sensitivity of 2mm / 5degrees); good contact was achieved by rotating the platen relative to the sample. Then the rubber membrane was pulled gently over the platen and a vacuum suction was applied to the sample via the bottom drainage hole before the O-rings were put into place and the split sample former was removed. With practice, the top platen was always to within 2 to 3 degrees of perfect horizontality.

3.2.5 Installing the instrumentation

A sample before and after instrumentation is shown in Figure 3.7 (a) and (b) on page 72. Instruments to be attached were two sets of axial LVDTs and one radial LVDT on a radial strain belt. The individual components are shown in Figure 3.8 on page 73. The radial belt was installed first, followed by the axial LVDTs. A schematic diagram of the procedure is illustrated in Figure 3.9 on page 74, and photos of a trial installation using dummy samples are shown in Figure 3.10 on page 75.

3.2.5.1 The radial belt

As illustrated in Figure 3.9 (b) and in Figure 3.10 (b), the radial belt had two rigid arms which rotated about a spring-loaded zero-backlash hinge. Each of the two arms was attached to the sample at a seat, positioned mid-length along the arms. The seats
were approximately 1.5cm wide and 2cm high, with a curved surface to match a 50mm
diameter cylinder. Two holes on the seat allowed two pins to pass through, as shown
in Figure 3.10 (c).

As illustrated in Figure 3.9 (a) and in Figure 3.10 (b), a fast-acting super glue was
applied on opposite sides of the sample at mid height. A roughened round-headed pin
was pushed through a blob of glue, and penetrated the membrane, and embedded fully
into the sand. The glue filled any gap between the pin and the membrane and hardened
rapidly to minimize further penetration into the sample by the pin under pressure.

The radial belt was wrapped around the sample, with the first seat resting onto the
round pin, making a point contact, whilst the second seat was glued directly to the
membrane by the second blob of super glue. Before the glue hardened, two roughened
carpenter’s steel nails/pins were pushed through the holes on the second seat and
penetrate the sample by 10mm, as shown in Figure 3.9 (b) and Figure 3.10 (c). More
glue was applied at the pin heads to ensure a rigid link between the pins and the seat.
Thus this seat was fully coupled to the sample mechanically.

A blob of super glue was applied at the tip of the arm of the radial belt where the
LVDT pin’s attachment would sit and the LVDT pin was gently guided into position
before the glue hardened as shown in Figure 3.9 (b), (c), (d) and Figure 3.10 (d) and
(e).

**Radial belt calibration**

The original design called for both of the seats to be rigidly glued to the sand sam-
ple, via the long pins that penetrated into the sample. However this design implied
the inevitable stretching of the spring-loaded hinge and ran the risk of distorting it.
The mechanism was also indeterminate making theoretical prediction of its calibration
coefficient impossible.

The current arrangement represents an effective modification to the original design,
by changing one of the rigid couplings between the seat and sample to a point sliding
contact. The other fully-coupled contact was necessary, so that the unavoidable forces
generated by the LVDT data-cable (or simply, wire) during testing were fully resisted
by this joint and were not transmitted to the hinge. The mechanism of the current
arrangement is shown in Figure 3.11 (a) on page 76, where the dimensions shown in
Figure 3.11 (c) were measured using a ruler.
As indicated in Figure 3.11 (a), the red colored section of the LVDT pin is the active section, its position tracked by the LVDT. It is represented by point D; while its neighboring point on the inside surface of the LVDT barrel is named point C. The axis of rotation of the hinge is named point H. Point O is the centre point of contact area between that arm of the radial belt and the sample surface. On the opposite side of the sample, the round pin is in point contact with the seat, and the point belonging to the round pin being named A, whilst the other belonging to the seat being named B. Therefore, HOC forms a rigid body and HBD forms a second rigid body; they can only rotate w.r.t. each other about an axis through point H, coming out-of-the-page. The deformation of OA (radial strain) is related to CD (LVDT measurement); as OA elongates or contracts, points A and B will separate by sliding past each other.

The velocity diagram of this mechanism is shown in Figure 3.12 on page 77. Assuming the LVDT is sensitive to the displacement between C and D along the direction of OA only, then the ratio of this to the displacement between O and A is 1.86. This is the theoretical calibration factor of the radial belt mechanism. A calibration was performed by propping-open points O and B on the knife edges of a digital calliper, as illustrated in Figure 3.11 (b), giving a value of 1.88, and this value was used in the calculations.

**Errors and limits of the radial belt measurement**

**The first difficulty** of the above mentioned mechanism, as seen in Figure 3.12, is the obvious separation of point C and D, which is 2/3 of the displacement between O and A. In practice this means that the LVDT’s pin would eventually scrape the inside surface of the LVDT barrel, generating unwanted friction forces. The mounting procedure of the LVDT pin described above was designed to place the pin at the optimum position, where the maximum amount of gap between it and the barrel existed. The play between the pin and the barrel was around 0.5mm, corresponding to a displacement of 0.75mm between O and A, and a corresponding radial strain of 1.5%. Therefore the radial belt mounting method became unreliable at larger strains. The radial belt in this test series was limited to radial strains of less than 1%.

The first method to avoid the 1% strain limitation was proposed by Menzies (1976) and illustrated in Figure 3.13 (a) on page 78. Instead of rigidly fixing the LVDT pin to the radial belt’s arm, two pivots are introduced, so that while the sample bulges towards failure, and the arm swings open, the pivots allow the pin to remain aligned.
with the LVDT barrel. A modern version adopting this solution is the commercially available local strain belt system manufactured by Wykeham Farrance, UK., illustrated in Figure 3.13 (b). However the pivots themselves, not being backlash-free, inevitably introduce uncertainty at small strain levels at the start of shearing or during stress cycles.

**A second difficulty** of the mechanism used in this thesis is related to the curvature of the seat as indicated in Figure 3.11 (c), which had a radius of 25mm, as it was originally designed to hug a 50mm diameter sample perfectly. If the point B in Figure 3.11 (c) is the symmetric center of that seat, it is conceivable that during preparation the actual contact point may be positioned on either side of B. A worst case offset of 2mm either side of B is considered here. The effect is indicated on the velocity diagram in Figure 3.12 by points b1, b2, d1 and d2. This translates to a maximum uncertainty of radial belt calibration factor, or calculated secant stiffness, of +/- 6%.

**A third difficulty** is also related to the curvature of the seat. As the test progressed, the separation of points A and B increased and at 1% radial strain it was approximately 0.25mm. In effect, this introduced the same type of uncertainty as has just been described above. It corresponds to a change in radial belt calibration factor of +/- 0.75%.

These quantifiable uncertainties amount to +/- 6.75% in the calculated secant shear stiffness value.

**An additional uncertainty** is to do with the friction force generated during the slippage between contact points A and B and its possible effect on the hinge. With reference to Figure 3.11 and Figure 3.12, a first-order estimate involves taking moments about the hinge at point H of the forces in the spring and at the contact; this gives a force ratio of more than 8 in favor of the spring. Assuming the coefficient of friction at the hinge is similar to that at the round pin/seat contact point, one arrives at a much larger limiting friction at the hinge compared to the friction generated at the sliding contact. Therefore the hinge is not dislocated by this force. However, this force is transmitted to the sample via point O and this influence is expected to be secondary, and was not quantified.

It is expected that by developing a non-contact method of local strain measurement technique, for example using digital image analysis (e.g. in White (2002) for measuring
sand displacement around laboratory model piles and in Take (2003) for centrifuge model clay embankment deformations), the difficulties discussed above may be avoided. However due to time limitations this route was not pursued for the purpose of this thesis.

Radial belt’s lower strain response limit

Visualizing the radial belt seats as being linked in series with the sample via a ‘contact spring’ representing the contact stiffness at the seat/sample junction, then a restricting force within the radial belt system can stretch or compress the springs by an amount that translates to an equivalent radial strain. This means that before this restriction force is overcome, the radial belt mechanism was stuck whilst the sample deformed up to this amount. It is important to quantify this strain to lend confidence to the data at small strain levels at the start of the test and during stress cycles.

The mounting method just now described was able to respond to a change in radial strain of approximately $6.3 \times 10^{-6}$, as will be demonstrated. Figure 3.14 on page 79 (top figure) indicates a null response during the stress cycle shown in Figure 3.14 (the stress path is shown in the bottom figure), whilst the axial LVDT registered approximately $1.3 \times 10^{-5}$ change in strain (middle figure). An estimate of the corresponding radial strain change is $6.5 \times 10^{-6}$ strain.

3.2.5.2 Mounting the axial LVDTs

The mounting of the axial LVDT was a much simpler affair. As illustrated in Figure 3.9 (e) to (h) on page 74 and in Figure 3.10 (f) on page 75, the LVDT body was super-glued to the sample body first and care was taken to ascertain verticality, generally better than 1-in-20. The LVDT pin was then inserted prior to super-gluing the reaction stand/seat. The gauge length was recorded, as the distance between center points of the LVDT attachment and the reaction stand.

3.2.6 Docking

3.2.6.1 Dimple and knob arrangement

Traditionally, the top cap has a dimple or recess in the middle, and the internal load cell has a hemispherical knob at its tip. During docking, the knob is lowered into the

---

1This estimate was calculated based on the observation that, as seen in Figure 3.14, between time 5400s and 5600s a change of radial strain of $7 \times 10^{-6}$ was registered whilst a change of axial strain of $1.4 \times 10^{-5}$ was registered. By assuming constant poisson’s ratio, thus it is obtained that:

$$
1.3 \times 10^{-5} \times \frac{7 \times 10^{-6}}{1.4 \times 10^{-5}} = 6.5 \times 10^{-6}.
$$

---
dimple and subsequently during shearing of the sample, the dimple is forced upwards against the knob. The top cap adjusts until the dimple fits the knob perfectly and the sample is perfectly aligned, as illustrated in Figure 3.15 on page 80. This process of alignment adjustment cannot be observed by the eye since it occurs at small axial strain levels and it is not monitored in standard triaxial set-ups. Therefore its effect on the measured soil stiffness is unknown.

This standard knob-dimple docking procedure was used in essentially the same GDS Instruments Ltd. triaxial set-up of Kok (2006). He observed ‘humps’ in Young’s modulus vs. axial strain plots, occurring in many of his tests at different axial strain levels, as shown in Figure 3.16 on page 81. He eliminated bedding error as a cause, and attributed them to the alignment adjustment of the knob-dimple docking.

The problem of knob-dimple arrangement arises because the contact force between them points well away from the vertical\(^1\), thus introducing a large bending moment even for small contact forces. This moment remains large until perfect alignment is enforced, leaving data prior to this point shrouded in uncertainty.

### 3.2.6.2 Knob-on-flat plate arrangement

In an attempt to reduce this out-of-alignment-induced-moment during docking, the dimple in the top cap was eliminated in this series of triaxial tests. The knob of the load cell came into contact with a flat top surface of the top cap during docking, as shown in Figure 3.17 on page 82. Thus the contact force pointed vertically downwards perfectly, provided that the top cap was perfectly level. This was ensured to within 2 to 3 degrees of the horizontal using a spirit level of an approximate sensitivity of 2mm per 5 degree tilt.

At the end of the consolidation stage, after a period of creep of up to a few hours, and just before shearing the sample, the load cell was brought into contact with the top cap by slowly turning a large threaded nut, as illustrated in Figure 3.18 (a) and (b) on page 82. Docking was completed when the load cell registered a small load. The sample was then allowed to creep for at least another 15 minutes before being sheared.

This modification meant no alignment adjustment would occur during shearing, provided that the contact between the knob and top cap did not slip, which was very unlikely. Therefore any misalignment introduced during sample preparation remained unchanged, and any small bending moment associated with it acted on the sample.

\(^1\) Typically 60 degrees from vertical, if the contact is frictionless
throughout the experiment with its magnitude increasing proportionally with the applied axial force.

This knob-on-flat plate type of docking arrangement was also adopted by Clayton & Heymann (2001), however humps were observed for chalk in these tests, as shown in Figure 3.19 on page 83, which was attributed to micro-crack closure when tested at low confining stress levels. More recently, no such anomaly was found by Heymann et al. (2005) who used both laser interferometry and LVDTs on a similar chalk sample with visible fissures at low confining stresses. While in Kok (2006) the residual soil samples are known to be fissured, the reason behind these humps are still unclear.

3.3 Anomaly during shearing

3.3.1 Identification of anomalies

As shown in Figure 3.20 on page 84, an improperly centred triaxial compression test induced bending moments in addition to axial compression. This behaviour was expected to show up as the two LVDTs on either side of the sample registered different axial strains due to sample bending, and this difference must increase almost linearly with the axial force while the axial strain remained small and the material behaviour was close to linear-elastic behaviour.

A sensitive way of observing this trend is by plotting one LVDT reading against the other; this way any deviation from the expected smooth line relationship cannot be attributed to the misalignment, and is an anomaly if no other satisfactory reason can be found.

In general, the data supports the simple bending model, for example in Figure 3.21 for test TC500-4 (top half of the figure) and TC500-5 (bottom half of the figure), on page 85. However for the majority of the tests, anomalies were observed at very small strain levels for example in Figure 3.22 on page 86 for these two tests; and Figure 3.24 on page 88 on page 88 for all the tests of this series.

The anomalies can be grouped under two categories:

- One of the two LVDTs sticks, i.e. reads zero until a sudden start of reading.

  This appears to be the typical stick/slip behaviour of a frictional contact and

\footnote{The test identifiers will be explained in a later section; here ‘TC500-4’ means a simple traxial compression test with sample consolidated to a mean confining stress of 500kPa; ‘-4’ means it’s the fourth test following the same stress path.}

\footnote{During tests TC500-4 and TC500-5 small unload-reload cycles were conducted. However, these data were omitted from these plots for clarity, hence the jerky appearance of strain-time curves.}
is attributed to the friction force between the LVDT barrel and its pin. This situation was observed in three of the tests.

- A visible sudden change in gradient of the LVDT1 vs. LVDT2 plot, or a ‘kink’. These are marked by circles, see Figure 3.22 and Figure 3.24. These ‘kinks’ are clearly discernable in some tests whilst they appear less in the rest. Although it might be suspected that the triaxial membrane was deforming relative to, and slipping over the soil, causing mis-registration of the LVDT mountings, this could not be proved.

3.3.2 Strain rate effect

For tests TC500-4 and TC500-5, the anomalies happened at a large increase of loading rate, as shown in Figure 3.22 on page 86, suggesting the possibility that they are manifestations of loading rate changes.

Similar increases of loading rates were introduced purposefully in test TC400 to examine this very rate effect on material stress-strain response. On examining the axial LVDT responses at these sharp changes, however, no visible kinks were found, as seen in Figure 3.23 on page 87. Loading rate effect was therefore discounted as a reason for the anomalies.

3.3.3 Effect of the anomalies on stiffness data

Figure 3.25 on page 89 and Figure 3.26 on page 90 show where the anomalies are on the tangent and secant Young’s modulus plots, using vertical solid lines. In two out of nine tests, a ‘hump’ is observed in the Young’s modulus plot, where an anomaly is found. Where a clear ‘hump’ can be observed in the secant stiffness plot, no clear distortion is observed in the corresponding tangent stiffness plot, emphasizing the importance of studying both plots together.

However for the majority of the tests, the apparent consistent trends of the data are not broken by the existence of the anomalies. It is not possible to suggest a link between the anomaly and the ‘hump’. As already discussed previously, the physical mechanism behind the ‘hump’ is still unclear.

3.3.4 Other considerations

For the axial LVDTs, one detail of contention is that a force that may act on the LVDT attachments may deform the membrane in shear, introducing erroneous axial strains.
This force is the bending moment and pulling force exerted by the relatively rigid wire which sticks out of the LVDT body and pinned to the bottom pedestal. Precautions to minimize this force were taken, similar to Cuccovillo & Coop (1997), however it was not possible to measure this force.

3.4 Test Data

3.4.1 Test summary

In total 14 triaxial compression tests were performed. All of the samples were tested dry and the pore pressure was kept at atmospheric. Therefore all the stresses reported below are effective values. All of the tests followed one of two different stress paths:

- Isotropic consolidation to \( p'_0 \), followed by a standard triaxial compression to failure, i.e. the stress path during shearing has a slope of 3 in a \( q \) vs. \( p' \) plot. This is a uniaxial compression test, during which only the major principal stress changes.

- Isotropic consolidation to \( p'_0 \), followed by a constant \( p' \) triaxial compression to failure, i.e. the mean stress \( p' \) is kept constant while \( q \) increases monotonically to failure.

They are referred to as Standard TC and Constant-\( p' \) TC test paths. The test stress path type, value of \( p'_0 \) of all the tests are reported in Table 3.1 on page 60. The tests are grouped under three categories:

- Preliminary tests: 5 tests were conducted to validate the sample preparation technique, during which different radial belt mounting methods were trialed; therefore no radial strain data will be presented for these tests. They follow the Standard
TC path, with the consolidation stress fixed at 500kPa. These are tests TC500-1, TC500-2, TC500-3, TC500-4 and TC500-5.

- Standard TC tests: 4 tests were conducted following the Standard TC path, with different consolidation stresses from 100kPa to 400kPa. These are tests TC400, TC300, TC200 and TC100.

- Constant-p TC tests: 5 tests were conducted following the Constant-p TC path, with different consolidation stresses from 100kPa to 500kPa. These are tests pTC500, pTC400, pTC300, pTC200 and pTC100.

All the data presented are calculated from local LVDT measurements, unless stated otherwise. The local strain data presented is limited to below approximately 1% radial strain. Axial stress and shear stress $q$ were calculated using the undeformed cross section dimension, introducing less than 2% error in stiffness at 1% radial strain.

### 3.4.2 Axial strain rates

The current GDS Instruments Ltd. control software only allows the external applied strain rate to be specified manually when carrying out the Standard TC tests. Strain rate was determined automatically by the control software during the Constant-p TC tests and varied during a test and between tests. The monitored internal axial strain rates in the tests ranged between approximately $10^{-5}$ per minute and $5 \times 10^{-4}$ per minute as shown in Figure 3.27 on page 91.

The effect of varying strain rate is assessed in tests TC400 and TC300. The applied external applied axial strain rates varied between $2 \times 10^{-5}$ and $2 \times 10^{-4}$ per minute in test TC300 and between $2 \times 10^{-4}$ and $2 \times 10^{-3}$ per minute in test TC400. The local stress-strain plots for both tests are shown in Figure 3.28 on page 92. The effect of increasing the applied strain rates by 10 times is an increase in shear strength and secant stiffness of less than 5%. Therefore, for the range of strain rates encountered within this series of tests, the effect of strain rate on stiffness measurement is less than 5%.

### 3.4.3 Preliminary tests and repeatability

For the preliminary tests, the behaviour during consolidation is shown in Figure 3.29 on page 93. The axial strain behaviour during standard TC compression is shown in Figure 3.30 on page 94 and the corresponding secant and tangent Young’s modulus
shown in Figure 3.31 on page 95. During both consolidation and shearing stages good agreements exist between the tests. At an axial strain level of $10^{-4}$, the secant Young’s moduli agree to within 13%; at $10^{-5}$ axial strain they agree to within 20%; noting the anomaly in test TC500-4.

3.4.4 Standard TC tests

For the Standard TC tests, the behaviour during consolidation is shown in Figure 3.32 on page 96. The stress and strain behaviour during standard TC compression is shown in Figure 3.33 on page 97; the corresponding secant and tangent Young’s modulus are shown in Figure 3.34 on page 98. Corresponding plots of shear modulus are shown in Figure 3.35 on page 99, deduced assuming equivalent linear elastic isotropic behaviour between relevant strain states, as defined by axial and lateral strain measurements. ‘Humps’ are observed in three out of the four tests shear modulus plots, but in only one in their Young’s modulus plots.

3.4.5 Constant-p TC tests

For the Constant-p TC tests, the behaviour during consolidation is shown in Figure 3.36 on page 100. The stress and strain behaviour during Constant-p TC compression is shown in Figure 3.37 on page 101; the corresponding secant and tangent shear modulus are shown in Figure 3.38 on page 102. Clear ‘humps’ are observed in two tests out of the five shear modulus plots.

3.5 Soil material model parameters

3.5.1 Critical state angle of friction

Limited by the functional range of the local strain instruments, none of the tests described above managed to load the sample to its critical state. One undrained standard triaxial compression test was carried out as part of the 5R5 Research Module under collaboration with a group of researchers at Cambridge University. This test may serve to provide an estimate of the critical state friction angle of the Fraction E sand. The sample was 107mm tall, 50mm diameter and prepared by air pluviation to a dry density of 1570 $kg/m^3$, corresponding to a relative density of approximately 80%. No local strain instrumentation was used. The stresses were corrected for the change in cross-sectional area. The test stress paths are shown in Figure 3.39 on page 103. It is believed that the pore pressure transducer failed to respond due to cavitation under negative pore
pressure. The last meaningful data is taken at zero pore pressure, as marked by the red cross, and the data point corresponds to a mobilized angle of friction of 35.7 degrees. This value is taken to be an estimate of the critical state angle of friction.

### 3.5.2 Peak angle of friction

Most of the tests did not reach a conclusive peak in their $q$ vs. axial strain plots, except tests TC500-1, pTC200, TC100 and pTC100. The mobilized angles of friction at the end of tests or at the peak are shown in Figure 3.40 on page 104; calculated using the undeformed cross-sectional area. The peak strength were achieved at between 2% and 5% axial strain measured by an external LVDT; at this strain range the internal strain instruments were already out of range. For comparison with typical triaxial data, the predictions of peak angle of friction for silica sands by Bolton (1986) is plotted in the the above figure as a solid line; it lies just above the corrected peak angle of frictions of Fraction E sand.

The deformed cross-sectional area of the samples were estimated by extrapolating the dilation rate measured from the internal LVDTs to the end of test; these corrections are between 5 and 13%. The mobilized angles of friction calculated using these corrected cross-sectional areas are also plotted in Figure 3.40.

### 3.5.3 Comparison with Lee

Lee (1992) performed extensive triaxial tests on uniform Leighton Buzzard silica sand of two different gradings: “7/14” (passing between British standard sieve sizes of 1.18 and 2.3mm; $D_{50} = 1.7mm$) and “52/100” (passing between British standard sieve sizes of 0.15 and 0.3mm; $D_{50} = 0.21mm$). These sands are different from Fraction E sand ($D_{50} = 0.12mm$) only by their different grading. The corrected peak and the critical state friction angles of Fraction E sand are superimposed onto Lee’s data in Figure 3.41 on page 104, using diamond-shaped symbols. The good agreement is encouraging.

### 3.5.4 Small strain stiffness, Poisson’s ratio and stiffness degradation

#### 3.5.4.1 Fahey-Carter model

A simple model capable of representing many realistic non-linear stress-strain relationships, is proposed by Fahey & Carter (1993). The model is compared with other similar methods in Fahey (1998). To characterize non-linear elasticity behaviour, this model
employs four parameters: $C$, $n$, $f$ and $g$, in addition to Poisson’s ratio, $\nu$. They are defined as:

$$\frac{G_0}{p_a} = C\left(\frac{p'}{p_a}\right)^n \quad (3.1)$$

where $p_a$ is the atmospheric pressure.

$$\frac{G}{G_0} = 1 - f\left(\frac{\tau}{\tau_{max}}\right)^g \quad (3.2)$$

where $G$ is the secant shear stiffness and $\tau_{max}$ is the maximum shear strength \(^1\); $f$ and $g$ are soil-specific parameters that govern the stiffness degradation relationship.

By varying their values, different modulus degradation curves can be produced (see Figure 3.42 on page 105) which are of similar form to the experimental results of others (see Figure 3.43 on page 106).

This model has been implemented into the 1D finite element program CAMFE (Carter (1978)), which was kindly made available to the author. Therefore the Fahey-Carter model was chosen here for its convenience.

### 3.5.4.2 Curving-fitting results

Figures 3.44 to 3.47 on page 107 to 108, show the results of curve-fitting using the Fahey-Carter model (Fahey & Carter (1993), assuming both the Young’s and shear modulus degrade according to the same relationship, with the model parameters of $C = 2200$ for Young’s modulus or 1000 for shear modulus, with $\nu = 0.1$ at small strain, $n = 0.5$, $f = 0.8$ and $g = 0.5$.

The choice of $\nu = 0.1$ is shown to be reasonable by inspecting the good quality data of test TC100 in Figure 3.45 and Figure 3.46. $E_{secant}$ for test TC100 at small strain is of the order of 200MPa, while $G_{secant}$ approaches approximately 100MPa at small strain and the corresponding tangent stiffness values approaches the secant ones at small strain. Given the relationship between them being $G = E/2(1+\nu)$, the choice of 0.1 is within the range of measurements. It is also a value recommended in Fahey (1993). The choices of $C = 2200$ and $n = 0.5$ are shown to be reasonable by inspection of the good quality data of the preliminary tests in Figure 3.44 on page 107, and of test TC200 and TC100 in Figure 3.45 on page 107. The small-strain secant and tangent

\(^1\tau_{max} = \frac{1}{2}\sqrt{\left(\sigma'_1 + \sigma'_3\right)^2 \sin^2(\phi'_{peak}) - \left(\sigma'_1 - \sigma'_3\right)^2}
Young’s modulus corresponding to consolidation stresses of 100, 200 and 500kPa are of the order of 200, 300 and 480MPa; the corresponding Fahey-Carter model predictions are 220, 310 and 490MPa. Parameters $f = 0.8$ and $g = 0.5$ were chosen to allow both the trend and magnitude of stiffness degradation to be optimally captured.
Figure 3.1: Schematic diagram of a GDS Instruments Ltd. triaxial system; following Kok (2006)
Figure 3.2: Photo of a GDS Instruments Ltd. triaxial system
Figure 3.3: (a) Bottom platen; (b) split former and filter paper grid; (c) ready for sand pouring
Figure 3.4: Sand pouring set-up
Figure 3.5: Schematic diagram of the triaxial sand pourer
Figure 3.6: Top cap placement
Figure 3.7: (a) A typical sample before and (b) after instrumentation
Figure 3.8: Instrumentation
Figure 3.9: (a) to (h) Installation procedures
Figure 3.10: (a) to (f) Installation photos using dummy samples
Figure 3.11: (a), (b) and (c) Radial belt mechanism

For calibration, the seats are propped open on knife edges and the separation is measured by a digital caliper.
Figure 3.12: Velocity diagram of the radial belt mechanism

\[ \frac{cd}{oa} = 1.99 \]
\[ \frac{cd1}{oa} = 1.88 \]
\[ \frac{cd2}{oa} = 2.09 \]
\[ \frac{ab}{oa} = 0.5 \]
Figure 3.13: (a) Menzies’ modifications; following Menzies (1976) (b) Wykeham Farrance’s design
Figure 3.14: Radial belt response during a stress cycle
Figure 3.15: Standard docking procedure; following Kok (2006)
Figure 3.16: Anomaly in secant Young’s modulus; following Kok (2006)
Figure 3.17: Knob-on-flat plate docking arrangement

Figure 3.18: (a) and (b) Docking procedures
Figure 3.19: Anomaly in secant Young’s modulus; Clayton & Heymann (2001)
Figure 3.20: Bending mechanism of the sample
Figure 3.21: Strains from axial LVDTs: top half of the figure is for test TC500-4 and the bottom half of the figure is for test TC500-5.
Figure 3.22: Anomaly on axial LVDT plots: top half of the figure is for test TC500-4 and the bottom half of the figure is for test TC500-5
Figure 3.23: Strain rate effect and anomaly; data from test TC400
Figure 3.24: Anomalies identified on axial LVDT plots
Figure 3.25: Effect of the anomalies on stiffness data
Figure 3.26: Effect of the anomalies on stiffness data
Figure 3.27: Measured local axial strain rates
Figure 3.28: Strain rate effect for test TC400 (top half of the figure) and TC300 (bottom half of the figure)
Figure 3.29: Consolidation stages of preliminary tests
Figure 3.30: Shearing stages of preliminary tests
Figure 3.31: Young’s modulus of preliminary tests
Figure 3.32: Consolidation stages of standard TC tests
Figure 3.33: Shearing stages of standard TC tests
Figure 3.34: Young’s modulus of standard TC tests
Figure 3.35: Shear modulus of standard TC tests
Figure 3.36: Consolidation stages of constant-p TC tests
Figure 3.37: Shearing stages of constant-p TC tests
Figure 3.38: Shear modulus of constant-p TC tests
Figure 3.39: Stress paths of undrained simple TC test
Figure 3.40: Mobilized angles of friction

Figure 3.41: Comparison with Lee (1992)
Figure 3.42: Modulus degradation curves of Fahey-Carter model; following Fahey (1998)
Figure 3.43: Modulus degradation curves for various geomaterials; following Tatsuoka & Shibuya (1991)
Figure 3.44: Young’s modulus of the preliminary tests; smooth solid line represents the Fahey-Carter fitting result

Figure 3.45: Young’s modulus of the standard TC tests; smooth solid lines represent the Fahey-Carter fitting results
Figure 3.46: Shear modulus of the standard TC tests; smooth solid lines represent the Fahey-Carter fitting results.

Figure 3.47: Shear modulus of the constant-p TC tests; smooth solid lines represent the Fahey-Carter fitting results.
Chapter 4

Experimental methods
4.1 Introduction

The centrifuge test series was designed to trial equivalent in situ field methods and to obtain pile foundation performance data for subsequent back analysis. Observations fall into four groups:

(1) The seismic method: its ability to measure small strain stiffness and stiffness degradation up to an intermediate strain, and its ability to estimate in situ stress during pile jacking.

(2) The mini pressuremeter: its design and operation, its performance and limitations.

(3) Pressed-in piles: pile base resistance during staged jacking and load settlement curves during subsequent load testing, effect of pile shaft roughness on pile base response.

(4) Wished-in piles: load settlement curves during load testing, effect of shaft roughness on pile response.

All of these experiments were performed on the Turner beam centrifuge at the Schofield Center, Cambridge University. A picture of the centrifuge is shown in Figure 4.1 on page 122. Its operation and the centrifuge scaling rules are explained in Schofield (1980).

The centrifuge model consists of three independent elements: 1) piling; 2) seismic; and 3) pressuremeter; all embedded in a tub of sand bed. Schematic diagrams of these three packages are shown in Figure 4.2 on page 123 and a photo of the assembly appears in Figure 4.3 on page 124. The design and operation of the first two are described in detail below and that of the pressuremeter in section 5.3.

4.2 Test sand and sand pouring

4.2.1 Fraction E silica sand

The sand was recycled from previous centrifuge experiments that used commercial Fraction E silica sand purchased from David Ball UK Ltd. The classification properties of new Fraction E sand has be described in section 3.1.

The difference between the recycled and new sand were studied using a sensitive laser particle sizing method that essentially measures the shadow size of free-falling particles through a laser beam (White (2003)) and the results are shown in Figure 4.4 on page 124. The recycled sand (Laser method: $D_{10} = 0.19 \text{mm}$) has less fines compared to
the new one (Laser method: $D_{10} = 0.16mm$) but its particle size distribution otherwise lies at the upper end of the sand. Section 3.5.3 and Figure 3.41 on page 104 have shown angles of friction of Leighton Buzzard sand of three different gradings; “52/100” ($D_{50} = 0.21mm$) sand has very similar grain sizes as Fraction E ($D_{50} = 0.12$ to $0.14mm$) and their peak angle of frictions differ by $< 3^\circ$. Given the excellent comparison of $D_{10}$ between new and old Fraction E sands obtained from the Laser method (0.19mm vs. 0.16mm) one may expect smaller difference between their peak angle of frictions and a simple estimate gives: $3^\circ \times (0.19 - 0.16)/(0.21 - 0.12) = 1^\circ$ which is smaller than the apparent scatter of angle of friction obtained in the triaxial test series (for example consider the data presented in Figure 3.40 on page 104). Therefore the recycled and new Fraction E sands are deemed comparable.

Another practical reason for the adoption of the recycled sand was its availability in large quantities, since almost half a ton of sand is required for one centrifuge experiment. The new fraction E sand was used in the triaxial test as discussed in Chapter 3.

### 4.2.2 Soil model preparation

Dry sand was poured into a steel tub of circular cross section, 850mm in diameter and 400mm deep, using a robotic sand pourer, shown in Figure 4.5 on page 125. The pourer operates by discharging sand from a nozzle through multiple sieves and at a controllable drop height. Its construction and calibration are described separately in Madabhushi et al. (2006) and Zhao et al. (2006a).

For most of the tests presented herein the sand bed was poured as a single uniformly dense layer using the same automatic sand pourer settings and the same sand. The densities of samples were calculated by using average sample height and they are listed in Table 4.1. Originally more than 10 test series were planned and a double digit test series ID was adopted to simplify software coding (i.e. the test ID string length is always 3), e.g. the first test series is referred to as T01 instead of T1. This notation persisted helped its abundant use in software data analysis codes.

<table>
<thead>
<tr>
<th>Test Series</th>
<th>T01</th>
<th>T02</th>
<th>T03</th>
<th>T04</th>
<th>T06</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density ($kg/m^3$)</td>
<td>1624</td>
<td>1643</td>
<td>1626</td>
<td>1640</td>
<td>1616</td>
</tr>
<tr>
<td>Relative Density (%)</td>
<td>94.2</td>
<td>100</td>
<td>94.8</td>
<td>99.1</td>
<td>91.7</td>
</tr>
</tbody>
</table>
Table 4.2: Pile surface roughness (c.f. $D_{10} = 0.095mm$)

<table>
<thead>
<tr>
<th>Pile ID</th>
<th>M (mm)</th>
<th>N (mm)</th>
<th>H (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>0.20</td>
<td>0.20</td>
<td>0.095</td>
</tr>
<tr>
<td>R2</td>
<td>0.20</td>
<td>0.25</td>
<td>0.080</td>
</tr>
<tr>
<td>R3</td>
<td>0.20</td>
<td>0.20</td>
<td>0.085</td>
</tr>
<tr>
<td>R4</td>
<td>0.15</td>
<td>0.30</td>
<td>0.110</td>
</tr>
</tbody>
</table>

The mean and standard deviation of densities between all test series are 1629 kg/m$^3$ and 10 kg/m$^3$, or 96% $D_r$ and 3% $D_r$. This deviation of 10 over 1620 kg/m$^3$ could be due to errors in sand volume measurement of 2mm over a depth of 370mm. At the depth measurement locations, the sand layer thickness was measured to within +/-1mm using rulers and a weighted-line. However the sand surface was not entirely flat. These difficulties in obtaining precise volume measurements are believed to be the major contribution to the scatter in density measurements tabulated above.

In test series ‘T01’, the sand sample contained two uniform horizontal layers of different densities. The top 95mm thickness of the sample was prepared loose at a density of 1411 kg/m$^3$ or 29.1% relative density ($D_r$), whilst the thicker bottom layer had a density of 1624 kg/m$^3$ or 94.2% $D_r$.

4.3 Piling Package

4.3.1 Model pile design

The model pile was made from straight sections of thick-walled smooth aluminium alloy or brass tube of 12.7mm outer diameter and 2mm wall thickness, with relative surface roughness $R_n \simeq 5 \times 10^{-3}$. Four piles (piles R1 to R4) had anis-symmetric (with respect to the long axis of the pile) regular ribs machined on to their surfaces using a lathe, which made the surfaces very rough along the axis of the pile. The ribs were approximately 0.1mm tall and regularly-spaced at approximately 0.4mm spacing; the pile surface profile along the shaft direction was obtained using a diamond-tipped surface profiler at the Materials Laboratory at Cambridge University Engineering Department; the results are shown in Figure 4.9 on page 128.

The ribs can be characterized by their width $M$, spacing $N$ and height $H$. The estimated mean values of these parameters for each pile are summarized in Table 4.2.

---

1There is only one pile that was made from brass and it is identified as the ‘Copper’ pile.
The pile was fitted with a strain-gauged load cell at the tip to record the unit base load, which is denoted $q_b$, and is analogous to CPT tip resistance since the pile is closed-ended, albeit with a flat rather than a conical tip, as shown in Figure 4.8 on page 127.

### 4.3.2 Manufacture and calibration of tip load cells

All the pile tip load cells were made in-house. They were strain-gauged thin-walled cylinders, with four 120Ω strain gauges forming a conventional full bridge which was sensitive to axial loading and insensitive to bending, as well as being compensated for temperature change.

Before the load cells were fixed into the piles, their yield characteristics were quantified using an Instron (Instron 5500 R with a 10kN load cell) in the Fatigue Lab of Cambridge University Engineering Department. The load cells have flat ends and they were squashed between two flat platens of the Instron, as shown in Figure 4.10 on page 129.

During the very first cycle of loading, the load was increased beyond 7000N until yielding just occurred. The yield stress thus measured was between 280 to 300 MPa and required between 7000 and 7600N of loading to achieve this. The load cells were then loaded cyclically from 0 to 7000N for 100 cycles before being strain gauged. Finally the cells were cyclic loaded to 7000N for 10 cycles, with laser strain measurement to ensure no visible yielding behaviour occurs. They were then strain-gauged and cycled a further 20 cycles to verify integrity. The maximum loading during the centrifuge tests did not exceed 5000N.

After the load cells were fitted to the pile, the final assembly was calibrated against a commercial load cell. Calibration results are plotted in Figure 4.11 on page 129. Typical non-linear and hysteretic behaviour of the load cells is shown in Figure 4.12 on page 130 for pile R2. In general, the load cell hysteresis\(^1\) is measured as approximately 2%. Since the maximum design tip load of a driven pile and a bored pile are approximately 5000N and 1500N respectively (at approximately 10% pile radius displacement during load tests respectively), it was decided to use CF measured at 5000N loading for pressed-in piles and that at 1500N for wished-in piles. Hence the load may be measured to within 2% of the actual value.

\(^1\) Defined as the maximum difference between load cell output readings for the same applied maximum load: one reading is obtained by increasing the load from zero to the maximum load and the other by decreasing the load from the same maximum load to zero.
4.3.3 Piling locations

The piling positions were located on the circumference of a circle with diameter of 430mm in a plan view of the circular container, as shown in Figure 4.14 on page 131. The minimum distance between adjacent piles was 160mm and the piles were located 210mm away from the rigid wall of the tub. Wished-in and pressed-in piles were installed to a final toe level approximately 170 to 200mm above the rigid base of the container.

4.3.4 Installation and load test of a pressed-in pile

Pile jacking was conducted using an actuator with one vertical degree of freedom. The actuator was capable of exerting 10kN of compression force while traveling at any speed up to 10mm/s. The model pile was fixed to the actuator and its movement was logged continuously using a wire potentiometer, a linear variable differential transformer (LVDT) or a laser displacement sensor. The force applied at the top of the pile was recorded using a commercial load cell, while that at the tip was recorded by the tip load cell.

At the end of installation, pile top load was reduced to approximately zero before commencing a load test. During a load test, the settlement was recorded by a separate short-range LVDT fixed to the moving actuator. Its spindle made contact with a stationary target during the load test.

The original design of this actuator is described by Silva (2005). A schematic diagram of it is shown in Figure 4.15 on page 132, reproduced from Silva (2005). The actuator was controlled using a dial on its controller box, as illustrated in Figure 4.16 (b) on page 133. The speed calibration of the actuator is shown in Figure 4.16 (a). Although there is a small difference in speed when the actuator movement direction is reversed, there is a linear correlation between the dial number and the speed.

4.3.5 Wished-in pile placement

The sand pouring was paused when the desired toe depth was reached, and the wished-in piles were hung from a taut steel wire with their toes touching the sand surface. The bowing action of the wire applied a small axial load that forced the pile into contact with the sand surface without inducing a circular footing failure. Good contact was ensured by manually twisting the pile back and forth a couple of times. A similar strategy was used to place the buried total earth pressure cells by Garnier et al. (1999),
who demonstrated the effectiveness of the twisting action in producing repeatable sand structure contact.

The pile verticality achieved using the above procedure was checked during four trial placements using a laser, and was between 1-in-20 and 1-in-60. The initial pile embedment was 160mm.

4.3.6 Load test of a wished-in pile

The pile actuator was brought into contact with the top of the wished-in pile inflight. First contact was identified as the point of sharp rise in load cell reading. A short-range LVDT was used to record the pile head settlement.

4.3.7 Calibration of the short-range LVDT

The short-range LVDT was fixed to the moving actuator via a rigid clamp and two bolts, as illustrated in Figure 4.6 on page 126. However some play existed between the LVDT spindle and its housing, and there was some concern that this may have affected its calibration. Because the total amount of movement during a pile load test is small, any such uncertainties may prove intolerable. Two series of tests were conducted to quantify this effect.

The tests were conducted using a similar fixture system with a digital caliper providing the reference. The measured calibration coefficients, CF, are plotted in Figure 4.7 on page 126. Tests were conducted with the spindle randomly placed relative to its housing, limited by the play between them. The results indicated a maximum variation in CF of 0.6%. Therefore, the load test data between tests are comparable with an uncertainty no bigger than 0.6%.

4.4 Seismic package

4.4.1 Air hammers and accelerometers

An air hammer was used as the source of vibration. The current design follows that of Arulnathan et al. (2001); it has been used successfully to measure small strain stiffness on the Turner Beam Centrifuge at Cambridge by Ghosh & Madabhushi (2002). It consisted of a metal tube, within which a piston was propelled back and forth by pressurized air. Each time the piston traveled the full length of the chamber it hit an end-stop and the impact produced a shock wave that traveled in the plane of the cross section of the cylinder and vibrated in the direction parallel to the axis of the
cylinder. The shock wave characteristics were different between the forward and the reverse impacts, and they were distinguished by an ID number of either 0 for 1.

Figure 4.17 on page 134 shows the two air hammers used and their schematic construction. Figure 4.18 on page 134 illustrates typical shock wave signals created by the hammers.

The accelerometers were calibrated using a Bruel and Kjaer Type 4291 Accelerometer Calibrator that applies +/-1g acceleration periodically to the accelerometers and measuring both the positive and negative peak voltages produced. The set-up is shown in Figure 4.19 on page 135.

4.4.2 Arrangement of accelerometer arrays

Two accelerometer arrangements were used as shown in Figure 4.20 on page 135. When the desired depth was reached during sand pouring, the automatic sand pourer was paused whilst the accelerometers and air hammers were rested on top of the sand; then sand pouring was resumed.

The first arrangement involved four vertically-spaced accelerometers which detected waves traveling vertically and vibrating in the horizontal direction. An air hammer was located vertically below the accelerometers.

The second arrangement consisted of four horizontally-spaced accelerometers which detected waves traveling horizontally and vibrating in the horizontal direction. A parallel air hammer was placed beyond the accelerometers. Figure 4.21 on page 136 shows this arrangement during model preparation. The black solid circle indicates the position which is directly underneath the path of the pile during installation.

Notice the relative size of the air hammer and accelerometers (10mm across) to their spacings (usually 20 to 40mm). Let’s assume there is only one point of entry into an accelerometer by the propagating seismic wave (this is a significant simplification, since sand surrounds the accelerometer and the vibration of the soil is transmitted to it via all the contact between the soil and the accelerometer body), then it is possible to place this point at the front or the back of the accelerometer body and this means that the effective wave travel distance between accelerometers is not known by an amount of the order of the size of the accelerometer (12mm) which is large compared to the spacing between them (20 to 30mm); this effect will be discussed further in later sections of this thesis.
The accelerometers were numbered such that the one placed closest to the air hammer was given an identification (ID) number of (1), the second closest (2) and so forth.

### 4.4.3 Accelerometers data processing

Example signals received by the accelerometers are shown in Figure 4.22 on page 136. The logging rate was between 45 and 50kHz. Raw data was not filtered or altered in any way.

Each pair of adjacent accelerometers registered a time delay in arrival of the same shock wave. This time difference was obtained by correlation of the two shock signals in the time domain. The center-to-center distance of the accelerometers was used as the travel distance and allowed the average wave velocity, \( V \), of the soil between them to be derived. Loop shear stiffness was then calculated from \( G = \rho V^2 \).

Relative shear displacement between the two accelerometers was calculated from double integration of their respective acceleration time records and taking the difference between the two. Loop shear strain was calculated from the maximum relative shear displacement divided by the center-to-center distance between the accelerometers.

The relative displacement between pairs of accelerometers was calculated by double integrating the acceleration time traces to obtain displacement traces. Manual adjustment was used to correct the signal drift as discussed in this section below.

Let us assume the following simple model for an accelerometer time trace, \( a \), such that it is composed of background noise \( C \) and a shockwave signal, \( \text{signal} \), which consists of several cycles of large amplitude motions lasting only a short time.

The simplest form of \( C \) is a non-zero constant. By double integration we obtain the displacement trace of the instrument, as:

\[
a = C + \text{signal} \tag{4.1}
\]

\[
v = Ct + \int (\text{signal})dt \tag{4.2}
\]

\[
u = 0.5Ct^2 + \int \int (\text{signal})dt \tag{4.3}
\]

If we subtract the displacements of two separate instruments, No 1 and No 2 (the numbers indicate the instrument number), we obtain:
\[ u_1 - u_2 = 0.5(C_1 - C_2)t^2 + \int \int (signal_1 - signal_2)dt \] (4.4)

The first term on the right hand side of Equation 4.4 denotes an error introduced by the non-zero offsets. This suggests that the contribution of the background non-zero signal \( C \) is a smooth function of \( t \) and the contribution of the \( signal \) should be easily distinguishable from it. This will be shown to be indeed the case. However the actual form of \( C \) is more complex than the constant signal assumption just made.

We examine a typical pair of shockwave signals in Figure 4.23 on page 137. This figure shows the complete time trace of the signal; it includes 100 data points prior to the arrival of the signal, and the entire seismic event. A smooth drift of displacement values, \( u_1 \) and \( u_2 \), with time can be seen. This is also reflected in the plot of \( u_1 - u_2 \) vs. time. At scale employed by Figure 4.23 the contribution from the \( signal \) itself can not be seen.

The data of Figure 4.23 are reproduced in Figure 4.24 on page 137 concentrating on the initial part of the shockwave signal. The contribution of \( signal \) is clearly distinguishable from a smooth background shift; by accepting that this smooth background shift is the contribution \( C \) we can make an estimate of this contribution by extending the smooth trend manually, as indicated by the green solid curve drawn in the plot of \( u_1 - u_2 \) vs. time. The shear displacement due to \( signal \) can be estimated by \( du \) as indicated in the same plot.

The error in adopting \( du \) as the shear displacement lies in the manual operation of extending the smooth contribution of \( C \). And the associated error range is indicated by the solid purple lines in the plot of \( u_1 - u_2 \) vs. time in Figure 4.24. In this case the expected error is within +/-5%.

To further test the above procedure for obtaining \( du \), accelerometer signals in the form of perfect single-cycle sine waves were generated by computer, as shown in the plot of acceleration, \( a \), vs. time in Figure 4.25 on page 138. The maximum relative displacement as shown in the plot of \( u_1 - u_2 \) vs. time is \( 5e - 5 \) (in arbitrary unit). A relatively large random noise signal (in comparison with typical seismic data obtained in this thesis) was superimposed onto the pure sine waves and the same procedure carried out as shown in Figure 4.26 on page 138 and we obtain \( du = 4.3e - 5 \) (in arbitrary unit). The error in this case is 14%.

Therefore the above procedure may introduce errors between 5% and 15% depending on the noise level of the signal. For typical signals of this thesis the noise level is small
(compare Figure 4.23 with Figure 4.26) the expected error is +/-5%; hence the shear strain values presented in this thesis will also have an expected error range of +/-5%.

Madabhushi (1990) and Chan et al. (1994) presented a more rigorous method of obtaining velocity, \( v \), from acceleration time trace, \( a \). It made use of the knowledge that velocity at the beginning and at the end of the experiment must be zero. This information can be used to pre-treat the acceleration data before integration. The suggested procedure, following Madabhushi (1990), is:

1) Compute the average acceleration
2) Offset the acceleration trace by this average value
3) Integrate the acceleration trace to obtain the velocity data

This procedure enforces zero velocity at the beginning and at the end of the data. In Figure 4.27 on page 139 this procedure is adopted to obtain an estimate of \( du \) for the acceleration data presented in Figure 4.26. The value of \( du \) obtained is \( 4.9e - 5 \) (in arbitrary unit) compared with the correct value of \( 5e - 5 \) (in arbitrary unit).

This author was not aware of this method of data correction at the time of production of this thesis. In hindsight the above procedure should always be employed and it can reduce the error of estimation. In the cases shown in Figure 4.26 and 4.27, the improvement of prediction accuracy was 10%.

4.4.4 Testing procedure

The tests were carried out during centrifuge swing-up from 1g to 60g in 10g steps; then at a constant centrifuge acceleration of 60g during pressed-in piling.

The installation of a jacked pile involved many cycles of loading. In these tests, the pile base was advanced by approximately 1.5 diameters during each cycle, before the pile head was unloaded. Each cycle consisted of the following steps: (1) continuous penetration of the pile; (2) pause of penetration whilst the load was held; (3) seismic tests before unloading; (4) unloading of the pile until the pile base load approached zero; (5) seismic tests after the unloading of the pile base load. The next cycle then followed immediately.

During the pause in penetration of step (2), before the start of seismic tests in step (3), the soil relaxed under the applied load and the pile base load reduced. This reduction in stress was in general less than 15% of the value of \( q_c \). This implies that stiffness
measured during step (3) could be up to 10% less than the value during continuous penetration.

Following the above procedures, the in situ wave velocity was monitored both before and after the unloading of the pile base at each increment of penetration. The corresponding data will be identified with a letter ‘a’, indicating that it was obtained after the base unloading step; or with a letter ‘b’, indicating measurements made with the full base load mobilized.

### 4.5 Centrifuge test series

Centrifuge test series were conducted in week-long slots, with each week’s tests being conducted in the same sample of sand and identified by their test series ID from 01 to 06. Within each test series, a number of centrifuge flights were performed. During each flight one pile test was undertaken, sometimes with seismic experiments. These tests are given individual test IDs which are listed in Table 4.3.

<table>
<thead>
<tr>
<th>Series ID</th>
<th>Test ID</th>
<th>Type</th>
<th>Pile Roughness</th>
<th>Pile ID</th>
<th>Load Test</th>
<th>$q_c$</th>
<th>Seismic Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>2</td>
<td>Non Disp</td>
<td>Rough</td>
<td>R3</td>
<td>N</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>Non Disp</td>
<td>Smooth</td>
<td>S3</td>
<td>N</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>Disp</td>
<td>Rough</td>
<td>R1</td>
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<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>7</td>
<td>Disp</td>
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<td>Copper</td>
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<td>Y</td>
<td>Y</td>
<td>Y</td>
</tr>
<tr>
<td>02</td>
<td>1</td>
<td>Disp</td>
<td>Rough</td>
<td>R1</td>
<td>Y</td>
<td>Y</td>
<td>N</td>
</tr>
<tr>
<td>2</td>
<td>Disp</td>
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<td>Copper</td>
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<td>Y</td>
<td>Y</td>
<td>Y</td>
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<td>3</td>
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</tr>
<tr>
<td>4</td>
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<td>Y</td>
<td>Y</td>
<td>N</td>
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<tr>
<td>03</td>
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<td>R3</td>
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<tr>
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<tr>
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<tr>
<td>2</td>
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<td>R1</td>
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<td>Y</td>
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<tr>
<td>3</td>
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<td>R1</td>
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<td>Rough</td>
<td>R1</td>
<td>-</td>
<td>N</td>
<td>-</td>
<td>-</td>
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</tbody>
</table>
The pressuremeter tests were conducted during test series 06 and are described separately in Section 5.3 on page 150.

4.5.1 Failed experiments

Out of a total of 13 penetration tests, 2 failed to produce data. Out of a total of 15 pile load tests, 3 failed to produce data. And out of a total of 11 seismic tests, 4 failed to produce data. They are described in detail in this section.

4.5.1.1 Failed pile tests

Tip load cell failure during penetration happened during tests 03T5 and 06T2. Damaged strain gauges, or their electrical connections, were the most likely cause. The subsequent scheduled load tests and seismic tests were aborted.

During test series 01, the short-range LVDT was not available. The long-range laser (500mm) that was used to monitor penetration displacement proved to be too noisy to distinguish sub-millimeter movements; therefore no load test data is available from tests 01T2 and 01T4.

During test 04T1, an operator error occurred during the final unloading stage prior to the load test; the pile was unloaded too much into tension and was effectively pulled-out, damaging pile-actuator connections, and rendering the subsequent load test incomparable with the other tests.

4.5.1.2 Failed seismic tests

The air hammer malfunctioned during tests 02T2 and T4. The post-test examination suggested that a small amount of sand had got inside the hammer body during assembly and effectively jammed its mechanism.

During test 04T1, a kink developed in the air supply line and prevented the air hammer from functioning.

During test series 04T3, the signal strength was so low that it is difficult to distinguish it from the background vibrations. It is believed that air leakage through the centrifuge slip ring, coupled with possible residue sand jamming the air hammer mechanism, caused the weak signals.
Figure 4.1: Turner Beam Centrifuge, Schofield Centre
Figure 4.2: Schematic set-up of centrifuge package assembly
Figure 4.3: Photo of the centrifuge package

(1) Arms of beam centrifuge  (4) Vertical actuator
(2) 850 mm diameter tub  (5) Model pile
(3) Laser displacement sensor  (6) Vertical load cell

Figure 4.4: Particle size distributions of recycled and new Fraction E sand
Figure 4.5: Automatic sand pourer
Figure 4.6: Short LVDT

Figure 4.7: Calibration of LVDT
Figure 4.8: Pile tip load cell design
Figure 4.9: Topography of rough pile surface; dimensions are indicated in (a); unit of dimension is mm
Figure 4.10: Cyclic loading of the load cells

Figure 4.11: Load cell calibration results, including the Pile Top Load cell; the solid straight lines are least-square data fitting results
Figure 4.12: Typical load cell hysteresis
Figure 4.13: Installation of the wished-in piles

Figure 4.14: Piling positions in plan view; the solid dots represent piling positions
Figure 4.15: Pile actuator; following Silva (2005)
Figure 4.16: (a) Actuator speed control calibration and (b) controller panel, following Silva (2005)
Figure 4.17: Air hammers

Figure 4.18: Shock waves
Figure 4.19: Calibration of accelerometers

Figure 4.20: Sketch of ACC arrangements
Figure 4.21: ACC array

Figure 4.22: ACC signals
Figure 4.23: Plots of acceleration, $a$, vs time; velocity, $v$, vs time; displacement, $u$, vs time and relative displacement, $u_1 - u_2$, vs time; ‘1’ and ‘2’ denotes data from accelerometers no 1 and no 2 respectively.

Figure 4.24: Data from Figure 4.23 are re-plotted for the initial part of the shockwave signal.
Figure 4.25: Computer generated single cycle sine wave signals.

Figure 4.26: A random noise was superimposed onto the signals of Figure 4.25 and the procedure of manually predicting $du$ was carried out as illustrated in the plot of $u_1 - u_2$ vs time.
Figure 4.27: Data from Figure 4.26 was reanalyzed using a procedure for integrating acceleration trace recommended by Madabhushi (1990).
Chapter 5

Centrifuge in situ testing
5.1 Introduction

The results of the centrifuge tests are presented in model scale and in the following order:

2. Mini pressuremeters: their design, calibration and operation; their performance in sand.

All the experiments were carried out in dry sand and all stresses are effective stresses. All the centrifuge data are presented in model scale (without up-scaling to prototype) These data will later be used to calibrate predictive analytical methods in Chapter 6. Centrifuge scalings laws, for example as described in Schofield (1980), to convert data from model to prototype scale.

5.2 Results of centrifuge pile tests

The base resistances of all the pressed-in piles recorded during in-flight penetrations are summarized in Figure 5.1 on page 144. Red-colored symbols represent piles with smooth shafts; black-colored ones represent rough shafts. The corresponding shaft resistances during penetrations are summarized in Figure 5.2 on page 145. The pile jacking was conducted in stages, each involving monotonic loading followed by an unload-reload cycle. The data of the unload-reload cycles were omitted from these figures for clarity, thus the jerky appearance of the load-toe level curves. During the subsequent load tests, the pressed-in pile base load-settlement curves are summarized in Figure 5.3 on page 146; and shaft load-settlement curves are summarized in Figure 5.4 on page 147.

For the wished-in piles, base load-settlement curves are summarized in Figure 5.5 on page 148; that of shaft load-settlement curves in Figure 5.6 on page 149. The initial embedment of the bored piles is 160mm. Red-colored symbols represent piles with smooth shafts; black-colored symbols represent rough shafts.

5.2.1 Sample uniformity and test repeatability

As seen in Figure 5.1 the smooth and consistent penetration profiles within each test and between tests testify to the uniformity within each sample and the repeatability
between them. Further confidence is provided by Figure 5.2 on page 145 where piles of comparable surface roughness and embedment mobilize comparable amounts of shaft resistance during pile jacking. This is also seen from the good agreement of pile tip settlement plots in Figure 5.3 on page 146 and Figure 5.5 on page 148.

5.2.2 Influence zone

Tests 01T5 and T7 are different from the others. As stated previously, the sand bed had a loose layer from ground level to 95mm below ground level (bgl) instead of being uniformly dense throughout. As seen in Figure 5.1 on page 144, a clear change of trend of the penetration profile is evident when the pile tip approaches 50mm bgl, or approximately 3.5 pile diameters away from the top of the dense stratum; thus the influence zone size below the pile tip is $>3.5D$ in this case. Similarly, the tip resistance becomes fully governed by the dense layer when it penetrates beyond approximately 130mm bgl, indicating a influence zone of $3D$ above it.

The empirical values commonly used in averaging CPT data for use in pile designs range from 4 to $8D$ above and below the pile tip. This data does not conflict strongly with these values.

5.2.3 Mobilized base resistance

It is often observed that a comparable displacement pile mobilizes a lot more base resistance than a non-displacement pile at the same amount of settlement (e.g. Randolph (2003); Deeks (2004)). Deeks (2004) presented back analysis of a sizable pile load test database and found secant pile base stiffness, measured at a settlement-diameter ratio of 2%, of a pressed-in pile is as much as 10 times that of a non-displacement pile. This is clearly demonstrated by the comparison of Figure 5.3 and Figure 5.5. The pressed-in piles mobilize almost 80% of the penetration resistance, $q_c$, after only 1mm of settlement; compared with only 10% of the $q_c$ for wished-in piles (see Figure 5.5). Back analysis of this contrasting behaviour will be presented in sections 6.2 and 6.3.

5.2.4 Mobilized shaft resistance

By comparing Figure 5.4 on page 147 and Figure 5.6 on page 149, one concludes that the pressed-in piles mobilize more than 2 times the maximum shaft resistance compared to the wished-in piles, for both rough and smooth shafts. This result demonstrates that much higher in situ stresses must exist around a pressed-in pile.
Comparing Figure 5.2 on page 145 with Figure 5.4 on page 147, one can observe that almost all of the shaft resistance during penetration was mobilized during the first millimeter of subsequent load tests, for both rough and smooth shafts. This indicates little change of stresses around the shaft at peak shaft capacity during static testing after pile unloading compared to that during steady penetration\(^1\).

Comparing Figure 5.4 on page 147 with Figure 5.6 on page 149, for the rough shafts, one recognizes clear peaks and post-peak softening for the wished-in piles, but no such behaviour for the pressed-in piles. This supports the analogy with simple shear boxes, as discussed in detail in the literature review chapter, that when a shear band is fully formed no further dilation is available upon immediate re-shearing (e.g. Uesugi et al. (1988) and Uesugi et al. (1989)).

Maximum shaft resistances were mobilized at 1\(mm\) displacement for the rough piles, or \(7D_{50}\), which is in qualitative agreement with typical shear box data from the literature (e.g. Uesugi et al. (1988) and Evgin & Fakharian (1996)). This result, again, supports the shear box analogy and the conclusion that the shaft mobilization behaviour is governed by relative roughness among other factors, in addition to normal stress. This will be discussed in more detail in section 6.4.

Notice in Figure 5.4 on page 147 the rough pressed-in piles of test series 04 mobilized approximately 35\% more peak shaft load compared to other rough piles. A probable explanation is that pile R1, which was used in the three tests, was more offset from the vertical than the other piles. However this can not be confirmed. With hindsight, the verticality of every pile test should be measured post-test before pile extraction.

5.2.5 Residual loads

Notice that in Figure 5.3 and Figure 5.5 that the curves do not exactly pass through the origin. This is because of base residual loads due to rebound of the pressed-in piles introduced by unloading post-penetration, and due to shaft down-drag caused by soil settlement during centrifuge swing-up of the wished-in piles. The residual loads are relatively small for the wished-in piles, being less than 60N. For the pressed-in piles, the maximum residual load is approximately 500N for the rough piles, or 20\% of the maximum shaft capacity.

\(^1\)One needs to make the assumption that the shaft-sand interface behaviour does not change; this is justified since the shear band is now fully formed and is shearing under critical state conditions; no further dilation is expected upon re-shearing, provided no ‘set-up’ is allowed to occur.
Figure 5.1: Pressed-in pile penetration tip resistance; ‘bgl’ stands for ‘below ground level’; red symbols = smooth piles; black symbols = rough piles
Figure 5.2: Shaft load during pressed-in pile penetration; red symbols = smooth piles; black symbols = rough piles
Figure 5.3: Pressed-in pile base load settlement curves; red symbols = smooth piles; black symbols = rough piles.
Figure 5.4: Pressed-in pile shaft load settlement curves; red symbols = smooth piles; black symbols = rough piles
Figure 5.5: Wished-in pile base load settlement curves; red symbols = smooth piles; black symbols = rough piles
Figure 5.6: Wished-in pile shaft load settlement curves; red symbols = smooth piles; black symbols = rough piles
5.3 In situ pressuremeter testing

The design of the miniature pressuremeter followed that of a traditional Menard-type pressuremeter. The pressure fluid used was colored de-aired water. Pressure was applied using compressed air at an air-water interface within a glass capillary. The precise position of this interface, or meniscus, was tracked using digital cameras. A schematic diagram of the concept is shown in Figure 5.7 on page 156. Probe expansion was calculated from the changes of the interface’s position. Pressure is recorded by a pressure transducer. The probe was designed to measure small to intermediate strains and avoid the problem of disturbance due to probe insertion. However, membrane penetration introduced considerable amount of uncertainty in the interpretation of the miniature pressuremeter; this important effect is discussed later in section 5.3.4.4.

Gaudin et al. (2005) developed a similar device for use on the centrifuge. Known as a miniaturized cone pressuremeter, it had an outer diameter of 10mm and a length of 20mm, giving an aspect ratio of 2 and was installed as a displacement probe at 1g. It was used to determine the cavity limit pressure of sands.

5.3.1 Design

The probes were wished-in-place instruments; they were buried during sand pluviation using the automatic sand pourer (Zhao et al. (2006a)). A total of 8 probes were placed at two different depths. Figure 5.12 on page 158 shows such a package ready for sand pluviation.

The probe was machined from aluminum tubing; a drawing of it is shown in Figure 5.8 on page 156 and a photo of it in Figure 5.9 on page 157. On it there were 4 rows of six 0.5mm diameter holes permitting water to inflate the rubber membrane; the membrane were of approximately 0.5mm thickness, which occupied the recess machined onto the outer surface of the tubing.

The Young’s modulus of the latex rubber was measured to be 1.07MPa. The effective outer diameter of the probe was on average 6.3mm and the active length of the probe was 35mm giving a length-to-diameter ratio of approximately 6, which is similar to many commercial pressuremeters (Yu et al. (2005)). Houlsby & Carter (1993) found that for a linear elastic-perfectly plastic material and for a pressuremeter with length-to-diameter ratio of 6, the back-calculated soil strengths may be 25 to 43%.

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1 This is the average taken over 6 tests, with a standard deviation of 0.09MPa
2 Accurate to within 1mm
higher than the actual shear strength of the soil. Yu et al. (2005) adopted a more realistic critical state soil model for clay (Yu (1998)) and found for a pressuremeter with an aspect ratio of 5, the back-calculated soil shear strength maybe 13% higher than the actual value. It was expected that by adopting an aspect ratio of 6 some corrections, perhaps up to 10%, would be required to convert the back-calculated soil parameters from raw pressuremeter data to their actual in situ values.

The glass capillary had an inner diameter of 0.4mm. The pipes used to transfer fluid from the glass capillary to the probes were very thick-walled copper tubing used in F1 racing cars’ brake systems.

Digital image processing was performed manually, during which the position of the meniscus was located by eye to 1 pixel accuracy (or 0.2mm).

The tests were load-controlled. The pressure control system worked by letting in or out small volumes of pressurized air in quick succession during loading or unloading. This was achieved by alternately opening and closing two solenoid valves placed in series; as illustrated in Figure 5.10 on page 157. The valves were controlled by a digital trigger, which was set to activate the switches in turn at any user-specified rate.

5.3.2 Preparation

The system was de-aired and then flushed through with colored de-aired water under vacuum. It was then sealed and made ready for burial by the robotic sand pourer. It was possible to prepare and test all the probes simultaneously. Detailed procedures are described below and the process is illustrated in Figure 5.11 on page 158.

1) The probe was enclosed inside an airtight pressure chamber; the end nut of the probe was removed.

2) The system was de-aired by applying a vacuum via the air intake at the top of the glass capillary tube.

3) De-aired water was introduced under vacuum through a water inlet on the pressure chamber and saturated the pressure chamber, the probe, the copper pipe and the capillary tube.

4) The vacuum was removed; the probe end nut, with a sealing O-ring, was screwed into place underwater via the pressure chamber’s access port.

5) The access port was sealed again and the probe was tested for system compliance by increasing air pressure simultaneously at the top of capillary tube and inside the
pressure chamber. This process also helped to dissolve any remaining air into the water under high pressure.

6) The pressure chamber was removed and the probe was then ready for burial.

Figure 5.12 on page 158 shows a photo of the probes ready for sand pluviation. Figure 5.13 on page 159 shows photos of the glass capillary tubes, the digital cameras and the completed centrifuge package with the piling actuator and air hammer controllers.

5.3.3 Raw measurement corrections

The raw data comprises the water meniscus level and the applied air pressure. These values were corrected for the following effects:

1) system compliance; 2) water head

The effect of membrane strength and changes of its thickness were estimated and ignored, whilst the effect of membrane penetration is discussed employing a simple elastic indentation model.

5.3.3.1 System compliance

If the probe membrane is restricted from any movement during a pressuremeter test, there should ideally be no change of the water meniscus’ location. In reality, water changes volume with pressure, the copper piping expands and contracts with changing pressure; both of these effects are expected to be linear and repeatable. Any air bubbles which were not flushed out during probe saturation will also change volume with changing pressure; this effect is expected to be nonlinear (since as pressure rises air becomes increasingly difficult to compress) and is repeatable. These changes were recorded as an apparent volumetric strain vs. applied air pressure, and were the measure of system compliance.

To restrict the membrane’s movement during the tests for system compliance, the probe was placed inside a pressure chamber and subjected to an air pressure that was equal to the applied cavity pressure. The measured system compliance of all the probes is shown in Figure 5.14 on page 159.

5.3.3.2 Correction for water head

In flight, the meniscus was always above the level of the probes, introducing a positive water head with respect to the probe. An initial cavity pressure of between 20 to 80kPa therefore existed at zero-input air pressure, simply due to this head of water. During
the centrifuge pressuremeter tests, the meniscus level changed by up to 180mm as water left or re-entered the capillary tube. At 60g, this was equivalent to a change in water pressure of up to 105kPa. This change in water head was taken into account when the raw data was processed. The location of the meniscus was measured to an accuracy of 0.2mm; so the correction should be accurate to 0.1 kPa.

5.3.3.3 Correction for membrane thickness

As the cavity expanded or contracted, the membrane stretches or shrinks along the hoop direction, acting as a long, thin cylindrical shell under internal pressure. Since rubber is incompressible this leads to a decrease or increase of rubber membrane thickness. Since the water meniscus level change only reflects the change of inner diameter of the membrane, the change in membrane thickness needs to be calculated to obtain the cavity strain measured at the outer diameter. This calculation was performed assuming incompressibility of the membrane; its Young’s modulus being 1.07MPa. The correction factor is \(9.4 \times 10^{-5}\) volumetric strain per 100kPa change of cavity pressure. This is very small compared with the volumetric strain corrections required for system compliance. It is ignored in further calculations.

5.3.3.4 Corrections for membrane strength

Similar to the correction for membrane thickness, during expansion the membrane deforms elastically, therefore providing resistance to expansion. This is estimated to be approximately 1kPa of extra cavity pressure required per 1% volumetric strain during inflation. This correction is ignored in the calculations, introducing a 1% error, since the increase in cavity pressure at 1% volumetric strain is 100kPa in this series of tests.

5.3.4 Prediction and measurement

5.3.4.1 CAMFE finite element program

CAMFE \(^1\) is a 1D finite element program developed by Carter (1978) written in FORTRAN for the analysis of cylindrical cavity expansion and contraction in a continuum. It contains a facility that allows user-defined material models to be included. Fahey & Carter (1993) added the Fahey-Carter soil model to the material library of CAMFE. A detailed description of CAMFE can be found in Carter (1978).

\(^1\)CAMFE is a shorthand for Cambridge 1-D Finite Element Program
CAMFE coupled with the Fahey-Carter model has been extensively used to perform
curve-fitting of pressuremeter data with multiple unload-reload loops (Fahey & Carter
Fahey (1998)). CAMFE and the Fahey-Carter material model will be used to predict
cavity expansion behaviour of the centrifuge tests. In section 3.5.4.1 Fahey-Carter
model parameters for the Fraction E sand was obtained and was employed in this
thesis.

5.3.4.2 CAMFE prediction
CAMFE was used to predict cavity expansion plots for the probes buried at two different
depths, using the Fahey-Carter soil parameters obtained from the triaxial tests (see
section 3.5.4.1), and assuming $K_0 = 1 - \sin(\phi_{\text{peak}})$, where $\phi_{\text{peak}}$ was obtained from
triaxial tests (Jaky (1948)); Gaudin et al. (2005) demonstrated Jaky’s relationship to
be accurate to within 5% during their centrifuge test, which had a very similar set-up.
The predictions are plotted in Figure 5.18 on page 162.

5.3.4.3 Measurements
Five probes survived the proof test and showed no signs of leakage. Among them,
probes 1, 2 and 4 were buried at shallow depth (10.6m bgl at mid-height, prototype
scale), while probes 7 and 8 were at a deeper depth (15m bgl at mid-height, prototype
scale). A total of 11 pressuremeter tests were carried out during 5 separate flights, using
these 5 pressuremeter probes. The raw data were corrected for system compliance and
for water head. The results of the deeper probes are plotted in Figure 5.16 on page 160
and the results of the shallower probes are plotted in Figure 5.17 on page 161 (the solid
and dashed smooth curves within these figures are numerical predictions and will be
discussed in later sections).

The tests at the shallow depth, showed excellent consistency, demonstrating the
repeatability of pressuremeter tests and the uniformity of the sample. Interestingly,
repeated testing between flights of the same probe did not appear to affect the re-
results, indicating that the swing-down-swing-up unloading-reloading process between
centrifuge tests completely erased any memory of past loading. However, differences
up to 0.01 cavity strain exist between the prediction and the measurement.
5.3.4.4 Membrane Penetration

The 0.5mm thick rubber membrane is relatively thick compared to the \( D_{10} \) grain size of 0.1mm, giving a size ratio of 5. Therefore, a simple indentation model is called for to model sand grain penetration (Molenkamp & Luger (1981)). This model suggests that the indentation depth, normalized by the sand grain diameter, is proportional to a function of void ratio and cavity pressure normalized by membrane stiffness. The proportionality constant has probable values between 0.6 and 1.4; this value is assumed to be 1 for this illustrative calculation. The void ratio used is \( e_{\text{min}} = 0.613 \), and the sand particle size is taken to be 0.1mm. The Young’s modulus of the rubber membrane is 1.07MPa. The results are plotted in Figure 5.15 on page 160, showing a volume strain error up to 0.03 at 250kPa which corresponds coincidentally with a general membrane penetration of 0.1mm. It may well be that cavity pressures in excess of 250kPa would have generated no further membrane penetration. Gaudin’s work on limit pressures could equally have been unaffected by membrane penetration. Unfortunately, the small-strain and intermediate strain data required in the present study fell precisely in the pressure range during which significant membrane penetration errors must be present: 0.25% error in volumetric strain over 100kPa pressure change, as shown by comparing CAMFE predictions with miniature pressuremeter data in Figure 5.16 and 5.17. The membrane penetration error is sufficiently large to render the data unreliable. No further analysis of the data is undertaken for the purpose of this thesis.

The pressuremeter tests were designed to produced in situ measurement of soil stiffness and stiffness degradation behaviour and produce model parameters for back analysis of pile load-settlement behaviour. Due to membrane penetration error this purpose could not be fulfilled. Triaxial stress-strain data will be used instead for this purpose in later sections of this thesis.
Figure 5.7: Schematic diagram of centrifuge pressuremeter

Figure 5.8: Drawing of the probe
Figure 5.9: The probe

Figure 5.10: Pressure control unit
Figure 5.11: Probe preparation procedures

Figure 5.12: Pressuremeter assembly ready for sand pouring
Figure 5.13: Completed centrifuge package

Figure 5.14: Measured system compliance
Figure 5.15: Membrane penetration-induced volumetric strain

Figure 5.16: Pressuremeter data for the deep probes; the solid smooth line represents finite element predictions of these pressuremeter tests using CAMFE corresponding to that shown in Figure 5.18
Figure 5.17: Pressuremeter data for the shallow probes; the dashed smooth line represents finite element predictions of these pressuremeter tests using CAMFE corresponding to that shown in Figure 5.18.
Figure 5.18: CAMFE predictions of pressuremeter results
5.4 In situ seismic testing

5.4.1 Interpretation of velocity measurements

As described previously, the seismic method yields a measurement of travel time between two accelerometers. This is then converted to velocity by assuming a travel distance, and this distance is not accurately known. Therefore the absolute values of \( G \) calculated from this velocity can contain a significant error.

However, the travel distance does not change during a test, unless the accelerometers are displaced by a significant amount. Therefore, by normalizing \( G \) with a reference value, one eliminates this uncertainty in travel distance. The same argument also applies to the local sand density, as follows:

\[
G = \rho V^2 = \rho \left(\frac{l}{t}\right)^2; \quad G_r = \rho V_r^2 = \rho \left(\frac{l_r}{t_r}\right)^2
\]

(5.1)

by dividing \( G \) calculated from one pair of accelerometers with a arbitrarily chosen reference \( G_r \), which, for example, may be the value of \( G \) measured at 60g centrifuge acceleration using the same pair of accelerometers, one obtains:

\[
\frac{G}{G_r} = \left(\frac{t_r}{t}\right)^2
\]

(5.2)

We also accept the following well-established relationship following Roesler (1979) expressing the dependency of small-strain stiffness on principal stresses:

\[
G_0 \propto \sigma'_1^a \sigma'_2^b
\]

(5.3)

or

\[
G_0 \propto \sigma'_1^n
\]

(5.4)

where \( n = a + b \) if \( \sigma'_1 \propto \sigma'_2 \), which is indeed the case during centrifuge swing-up, for example Garnier et al. (1999). Therefore it is expected that a unique power law relationship between \( G_0 \) and \( \sigma'_v \) exists such that:

\[
G_0 \propto \sigma'_v^n
\]

(5.5)
We may also make the commonly accepted assumption that the material model is of the form:

$$G \propto G_0 f(\gamma)$$  \hspace{1cm} (5.6)

and the form of $f(\gamma)$ will depend on the choice of a particular model. The Bolton & Whittle (1999) model is especially convenient, because it is a one-parameter power law relationship, which will allow the effect of stress and strain to be separated. The Bolton-Whittle model states that:

$$G \propto \gamma^\alpha$$  \hspace{1cm} (5.7)

where $\alpha$ is a curve fitting parameter; $\gamma$ is calculated as being the maximum simple shear strain, being:

$$\gamma = \frac{d}{l}$$  \hspace{1cm} (5.8)

where $d$ is the maximum shear displacement between adjacent accelerometers, and $l$ is their spacing. Shear displacement/time data of any accelerometer was obtained by double integration of the raw acceleration/time. Therefore, relative displacement/time data of adjacent accelerometers was calculated by subtracting their displacement/time data, and $d$ is the maximum value of the relative displacement. Given the expression of $G_0$ in terms of stress, one obtains:

$$G \propto \frac{\sigma'_v}{\sigma'_{vr}} \gamma^\alpha$$  \hspace{1cm} (5.9)

and

$$\frac{G}{G_r} \propto \left(\frac{\sigma'_v}{\sigma'_{vr}}\right)^n \left(\frac{\gamma}{\gamma_r}\right)^\alpha$$  \hspace{1cm} (5.10)

Taking logarithms of both sides, one obtains:

$$\log \frac{G}{G_r} \propto n \log \frac{\sigma'_v}{\sigma'_{vr}} + \alpha \log \frac{\gamma}{\gamma_r}$$  \hspace{1cm} (5.11)

Therefore by plotting $\log \frac{G}{G_r}$ vs either $\log \frac{\sigma'_v}{\sigma'_{vr}}$ or $\log \frac{\gamma}{\gamma_r}$, the best-fit values of $n$ and $\alpha$ can be obtained.
5.4.2 Small-strain stiffness variations with stress level

Figures 5.19 to 5.24 on page 169 to 172 present the seismic data collected during centrifuge flights at increasing acceleration; also plotted are the predicted values of $G_0$ using the triaxial results, where $G_0 \propto p'^n$, $n = 0.5$ and assuming $p' = \sigma'_{uv}$. It is apparent that the different accelerometer pairs give different absolute values for $G$, and the differences between them is quite large. This was anticipated and has been attributed to the uncertainty in wave travel path length, discussed in the previous section.

Plotted on the right-hand side of the figures are the maximum shear strains mobilized by the shock waves. For each group of data, the shear strains are mostly within narrow bands and present no visible correlations with $G$; it is therefore possible to assume that the shocks happened at approximately the same strain level for each of the test group.

By plotting $G/G_r$ vs $\sigma'_{uv}/\sigma'_{vr}$, the power relationship between $G$ and stress is recovered. The results are plotted in Figures 5.25 to 5.29 on page 172 to 176. The smooth solid lines are again the triaxial predictions. Significant scatter is still present, especially at lower g-levels, for example in Figure 5.25 on page 172. This is due to high noise-to-signal ratios, since instrument-soil coupling at low confining stresses is poor (e.g. Zeng et al. (2006)). However, the unique power law is clearly seen, and is in good agreement with the triaxial prediction.

5.4.3 Stiffness degradation with strain level

In test 01-T5-12-0 and 1, the air hammer was used to excite a range of different shear strains during centrifuge flight at increasing acceleration, as shown in the right hand side of Figure 5.22 on page 171. Therefore these results contain the effects of both strain and stress. As discussed in the previous section, by making use of the Bolton-Whittle model, these two effects can be separated by plotting $\log \frac{G}{G_r}$ vs $\log \frac{\sigma'_{uv}}{\sigma'_{vr}}$ and vs $\log \frac{\gamma}{\gamma_r}$ respectively. This is done in Figure 5.30 on page 177. Data scatter is reduced by averaging, and the result is shown in Figure 5.31 on page 177. In the right-hand side figure, the solid line represents the triaxial results of $\ln(G/G_r)$ vs $\ln(\sigma'/\sigma'_{vr})$. In the left-hand side figure, the solid line represents a best fit relationship to the seismic data between $G$ and shear strain:

$$\log(G/G_r) \propto -0.42 \log(\gamma/\gamma_r)$$

(5.12)
The Bolton and Whittle power-law model adequately describes the seismic data, with \( \alpha = -0.42 \).

5.4.4 Comparison with the triaxial stiffness degradation data

Dynamic lab tests provide measurements of the dynamic backbone, while static tests provide that of the static backbone. The secant shear stiffness measured in the seismic method is the loop stiffness as defined in Hardin & Drnevich (1972), as shown in Figure 5.32 on page 178 (following Lanzo et al. (1997)). For the first cycle of loading, the test itself is essentially a simple shear test, and the stiffness is equivalent to that of a monotonic load test. Differences between the seismic and triaxial tests will include the effects of strain rate, loading cycles and cyclic shear strain level.

5.4.4.1 Strain rate

The strain rate effect is considered generally quite small or not observable at small to intermediate strains for clean sand (for example, Matesic & Vucetic (2003); Hoque & Tatsuoka (2004); LoPresti et al. (1997); Bolton & Wilson (1989)). The seismic tests are conducted at high strain rates (100%/s) compared with the triaxial tests (between 0.00002 and 0.001%/s), giving a strain rate ratio of \( 10^5 \) to \( 10^6 \).

The strain rate effect, at very small to high strain rates (0.00003%/s upwards), on the stiffness and strength of sand has been extensively studied before (see a summary by Matesic & Vucetic (2003)). It is shown to increase approximately linearly with logarithm of strain rate. Matesic & Vucetic (2003) measured an increase of 0.2% to 6% per ten fold increase in strain rate for Toyoura, Nevada and Riverside silty sands. For these sands the potential effect of strain rate could be between \( 1.002^5 = 1.01 \) and upto \( 1.06^6 = 1.4 \). Fraction E silica sand used in this study is a clean sand that was washed during its production and is free from clay, silt or organic matter; therefore it is expected that rate effect for this sand may lie much closer to the lower limit of 1.01 instead of the upper limit of 1.4. However high strain-rate triaxial testing was not conducted for this sand and the precise effect of strain rate is not known.

5.4.4.2 Cyclic shear strain amplitude

The dynamic backbone is in general different from the static one; the difference between them depends on the type of soil, and it grows with stress or strain level. They coincide at zero shear strain (e.g. Fioravante et al. (1994); Jamiołkowski et al. (1994); LoPresti
and at strain levels smaller than the threshold strain, $\gamma_{tv}$ (Ishihara (1996); Vucetic & Tabata (2003)). $\gamma_{tv}$ has values of $10^{-5}$ for sands (Hsu & Vucetic (2004)). Since the strain range of interest is up to $10^{-3}$ in this thesis, a difference between the cyclic and monotonic backbone is anticipated.

### 5.4.4.3 Number of loading cycles

The number of loading cycles during cyclic testing is denoted as $N$. When $N$ is 1 the measured stress-strain relationship is known as a static backbone and when $N$ is large it is known as a dynamic backbone. LoPresti et al. (1997) has conducted a thorough comparison of the effect of $N$ on the shear stiffness vs strain relationship for both sand and clay at strain levels greater than the threshold strain. Their results for Toyoura and Quiou sand are reproduced in Figure 5.33 on page 179. The value of $N$ required to approach the dynamic backbone is 10 or less. For Huston sand this is around 6 to 10 (Hoque & Tatsuoka (2004)), and appears to be 10 cycles for Leighton Buzzard 14/25 sand (Bolton & Wilson (1989)). Sharma & Fahey (2003) has also produced a comparison of stiffness degradation as a function of $N$ for clean and cemented sand; see Figure 5.34 on page 180. Their results are similar to that shown in LoPresti et al. (1997); however for cemented sand, $N$ has a drastic effect due to gradual bond breaking (Sharma & Fahey (2003)).

A typical air hammer shock wave is severely damped by the soil and generally induces one cycle of shear loading. At any centrifuge acceleration level, for example test 02-T3-12-0-vh at 60g, approximately 20 consecutive shocks were fired. During the first shockwave $N = 1$, therefore the measured soil stiffness is the monotonic stiffness determined using the static backbone. With every additional shockwave cumulative plastic volumetric strains must be developing. And after 10 shockwave the measured soil stiffness must be approaching that determined from the dynamic backbone. Therefore one might expect to record a gradual change of measured shear stiffness values with increasing number of shockwave. However the large amount of scatter in the raw data made it impossible to observe. Under the circumstance a constructive approach of making use of the data is to take the average of all the data points. The resultant average shear stiffness may lie between that obtained from the static backbone and the dynamic backbone, albeit approaching that from the dynamic backbone given that $N \gg 10$ for all the tests. Hence it is expected that the stress-strain relationship constructed using the seismic data will give an estimate of the dynamic backbone.
Resonant column tests were not conducted for Fraction E sand for the purpose of comparison with the seismic data. Therefore the difference between the seismic method and the conventional dynamic element tests is not known.

5.4.4.4 Stress path and anisotropy

Additional differences may be the result of differing stress paths and anisotropy; since the seismic test is a plane strain test in $K_0$ consolidated soil, the triaxial compression tests have $b = 1$, and the major principal stress is always perpendicular to the plane of deposition. As discussed in the literature review section on anisotropy, differences up to 40% might be expected.

5.4.4.5 Comparison with monotonic triaxial data

Figure 5.35 on page 181 and Figure 5.36 on page 181 reproduce the triaxial test data on a log-log scale in accordance with the Bolton Whittle model. The best fit relationship to the seismic data of: $\log(G/G_r) = -0.42 \log(\gamma/\gamma_r)$ is also plotted as a solid line. The difference between the seismic data and the standard TC path tests data are clearly seen. Therefore, for any problem the designer must judge whether the monotonic or the cyclic stiffness or something in between should be used in making predictions. Where a rotary-bored pile is concerned, one may expect the monotonic backbone should be used to predict its load/settlement behaviour under working load. For a diesel-hammer driven pile, the dynamic backbone should be more appropriate.
Figure 5.19: Seismic data from test 01-T5
Figure 5.20: Seismic data from test 02-T3

Figure 5.21: Seismic data from test 01-T7
Figure 5.22: Other seismic data from test 01-T5

Figure 5.23: Seismic data from test 02-T2
Figure 5.24: Seismic data from test 06-T1

Figure 5.25: Normalized seismic data from test 01-T5
Figure 5.26: Normalized seismic data from test 02-T3
Figure 5.27: Normalized seismic data from test 01-T7
Figure 5.28: Normalized seismic data from test 02-T2
Figure 5.29: Normalized seismic data from test 06-T1
Figure 5.30: Other normalized seismic data from test 01-T5

Figure 5.31: Averaged normalized seismic data from test 01-T5
Figure 5.32: Cyclic shear stiffness; Lanzo et al. (1997)
Figure 5.33: Cyclic and monotonic stiffness degradations; LoPresti et al. (1997)
Figure 5.34: Cyclic and monotonic stiffness degradations; Sharma & Fahey (2003)
Figure 5.35: Shear modulus of the standard TC tests

Figure 5.36: Shear modulus of the constant $p'$ TC tests
Chapter 6

Back analysis using cavity expansion methods
6.1 Estimation and prediction of in situ stiffness and stress during pressed-in piling

6.1.1 Introduction

This section attempts the measurement of in situ stiffness and stress below and around the pile tip during staged jacking and their prediction using a simple spherical expansion analysis.

As described in section 4.4.4 on page 119, seismic wave velocities were recorded during the staged jacking installation of both rough and smooth pressed-in piles. As discussed in section 2.2 on page 9, seismic wave velocity may be used to derive in situ shear stiffness and give estimates of in situ stresses. This section will assess and exploit this capability to measure the in situ stiffness and stress changes during the staged jacking process.

As discussed in section 2.4.2 on page 21, the spherical cavity limit pressure has been used in predicting pile capacity. In the following sections a spherical cavity expansion solution using a simple linear-elastic Mohr-Coulomb model will be used to match the cavity limit pressure to pile penetration resistance, \( q_c \), by varying the elastic shear stiffness. Using this best-fit value of shear stiffness, a prediction of in situ stresses during pile jacking and after pile unloading is made using spherical cavity solutions.

6.1.1.1 Stress-induced anisotropy and limitations on prediction

Hongnam & Koseki (2005) gave a comprehensive review of the relationship between stress anisotropy and shear modulus. An important empirical observation has been the independence of shear modulus from the out-of-plane normal stress and the associated equation:\(^1\)

\[
G_0 \propto \sigma_a^{na} \sigma_b^{nb}
\]

which is well established (Roesler (1979); Ishihara (1996); Stokoe et al. (1997); Fioravante (2000); Fioravante et al. (1998); Ismail et al. (2005)).

Bellotti et al. (1996), Fioravante et al. (1998) and Fioravante (2000) gave a thorough review of the values of \( na \) and \( nb \) for sands from the literature. Their values range from 0.22 to 0.3, and it is shown that \( na \approx nb \) with \( na + nb \approx 0.5 \) on average. This equation

\(^1\)\( G_0 \) is the small-strain shear stiffness associated with principal stresses \( \sigma_a \) and \( \sigma_b \); \( \sigma_a \) and \( \sigma_b \) are along the direction of wave travel and of wave polarization; \( na \) and \( nb \) are soil specific parameters.
is used in this section to compute values of in situ stress from the measured values of $G_0$. However, this equation is only valid when the fabric of the soil is not drastically disturbed by loading, which happens if a large shear stress is applied or at a large ratio of major to minor principal stresses. We will study the error of prediction using this equation at large stress ratios by reviewing the literature.

Yu & Richard Jr (1984) observed such an effect of large stress ratio, $K = \sigma'_1/\sigma'_3$, with values up to 5. They recorded over-prediction of $G_0$ by up to 30%, if the stress ratio effect is ignored completely. To account for it, they proposed the formula:

$$G_0 \propto \sigma'_a^{\alpha} \sigma'_b^{\beta} (1 - \alpha R_n)$$ (6.2)

where

$$R_n = \frac{K - 1}{K_{\text{max}} - 1} = \frac{\sigma'_1/\sigma'_3 - 1}{(\sigma'_1/\sigma'_3)_{\text{max}} - 1}$$ (6.3)

and where typical values of $\alpha$ and $\beta$ were 0.2 and 2. At the limiting stress ratio, $R_n = 1$, thus giving $G_0 \propto 0.8 \sigma'_a^{\alpha} \sigma'_b^{\beta}$.

Hoque & Tatsuoka (1998) studied this effect for Toyoura sand using precise local strain instrumentation on a rectangular-shaped sample inside a triaxial cell. They found that when $K < 2.5$ during triaxial compression, there is no observable error of prediction; beyond 2.5, over-prediction builds up linearly to almost 30% close to failure. Hoque & Tatsuoka (2004) presented triaxial data of 4 different sands. Their results are reproduced in Figure 6.1 on page 190. The departure point between prediction and measurement is at $K \approx 3$ for all three cases, and over-prediction at larger stress ratios is 40%. Kuwano et al. (2001) and Chaudhary et al. (2004) studied the same effect using a hollow cylinder apparatus. Their results confirm the above opinion that prediction becomes poor at high stress ratios where the point of departure is $K = 2.5$ to 4 depending on loading direction. Over-prediction of up to 40% was recorded approaching failure. Hongnam & Koseki (2005) conducted hollow cylinder torsional loading tests using Toyoura sand; their results are seen in Figure 6.2 on page 191 to indicate that the effect of stress ratio is small if $K < 4$.

Therefore, it is expected that the prediction of stresses from stiffness measurement will be especially accurate at smaller stress ratios. At larger stress ratios, the prediction of stresses gradually becomes worse and underestimates the true value by 40% at the limiting stress ratio of failure.
6.1.2 Measurement of in situ stiffness

As discussed in section 2.2 on page 9, the seismic velocities provide direct measurements of shear stiffness. It was shown in section 5.4.3 on page 165 that the Bolton and Whittle model (Bolton & Whittle (1999)) can fit the seismic stress-strain data well, such that:

\[ \log \left( \frac{G}{G_r} \right) \propto -0.42 \log \left( \frac{\gamma}{\gamma_r} \right) \]

following Equation 5.12 on page 165. Normalizing \( \frac{G}{G_r} \) by \( (\gamma/\gamma_r)^{-0.42} \) allows the effect of seismic signal magnitude to be canceled out, since it varies somewhat between tests as it proved difficulty to keep the centrifuge air supply pressure absolutely constant given the difficulties with slip-ring leakage. This allows the effect of minor seismic signal strength variations to be filtered out. The Bolton and Whittle model for Fraction E sand may be expressed as the following:

\[ \left( \frac{G}{G_r} \right) \left( \frac{\gamma}{\gamma_r} \right)^{-0.42} = \left( \frac{\sigma'_v}{\sigma'_{v0}} \right)^{av} \left( \frac{\sigma'_h}{\sigma'_{h0}} \right)^{ah} \]  \hspace{1cm} (6.4)

where \( av + ah = 0.5 \), as shown in section 5.4.2 and 3.5.4.2. We may assume that \( av = ah = 0.25 \), as discussed in the previous section.

Figures 6.5 to 6.10 on page 193 to 197 plot this normalized stiffness against normalized distance (distance between pile base and the accelerometers, \( h \), divided by the diameter of the pile, \( D \)) during pressed-in piling tests, using open symbols (the solid lines within these figures are analytical predictions which will be discussed in later sections of this thesis). The test procedures were described in section 4.4 on page 115; Figure 4.20 on page 135 summarized the definition of terms. Clear trends of increasing in situ stiffness with proximity to the pile tip can be seen; also there is very little observable decrease of in situ stiffness after pile unloading, for example, compare Figure 6.6 with Figure 6.5, noticing the relatively large amount of data scatter.

Notice large erratic jumps in data presented in Figures 6.7 to 6.8 in comparison to more reasonable ones in Figures 6.5 to 6.6. This is due to the difficulty with measuring every short impulse signal such that, even at 50kHz logging rate, the largest peak in the acceleration-time record is characterized by only 5 data points, rendering the process of cross-correlation erroneous. With much higher logging rates afforded by more elaborate data acquisition systems in the future, data scatter can be reduced.

6.1.3 Predictions using the cavity expansion method

Zhao et al. (2006b) demonstrated that if cavity expansion limit pressures are made to fit the measured pile base resistance, even with significant simplifying assumptions, one
can provide an excellent fit to the estimated stress fields below a pressed-in pile in dense sand. This method is further developed herein by reducing the number of assumptions. The method involves two steps:

1. A simple linear-elastic Mohr-Coulomb model is used to calculate cavity limit pressure, $p_L$. $p_L$ is matched to the pile penetration resistance, $q_b$, by varying the elastic shear stiffness, $G_{cav-pL}$, of the model. The input of this exercise is the pile penetration record of: $q_b$ vs depth, $d$. The object is to match the cavity limit pressure, $p_L$, to that of $q_b$ at every depth. The output is a list of best-fit values of shear modulus, $G_{cav-pL}$ vs $d$.

2. Using these values of, $G_{cav-pL}$, calculations of cavity-induced stresses are made using a large-strain spherical cavity expansion solution for cavities at different depth, $d$. These stresses are then superimposed onto virgin in situ stresses to form a prediction of in situ stresses induced by pile installation.

### 6.1.3.1 Step 1: Matching the cavity limit pressure to $q_b$

Carter et al. (1986) presented an elegant closed-form solution of the spherical cavity limit pressure, $p_L$, for a linear-elastic Mohr-Coloumb soil:

$$G_{cav-pL} = \frac{p_0'}{2N + k} \left[ Tp_{L}^{1+\alpha} - Zp_{L}^{1-\beta} \right]$$

where:
- $\psi$ and $\phi_{peak}$ are dilation angle and peak angle of friction;
- $G_{cav-pL}$ is the linear-elastic shear modulus;
- $p_L$ is the spherical cavity limit pressure;
- $p_0'$ is the initial cavity pressure prior to expansion;
- $k = 2$ for a spherical cavity;
- $T = (k + 1)(1 + \frac{kN}{\alpha+\beta})$;
- $Z = (k + 1)\frac{kN}{\alpha+\beta}$;
- $\alpha = k/M$; $\beta = 1 - k\frac{N-1}{N}$;
- $M = \frac{1+\sin(\psi)}{1-\sin(\phi_{peak})};$
- $N = \frac{1+\sin(\phi_{peak})}{1+\sin(\psi)};
- \chi = \frac{\nu k(1-\nu) - k\nu(M+N) + (k-2)\nu+1}{(k-1)^2\nu+1\nu+1}\frac{MN}{[k(1-\nu)+1]MN}$.

Poisson’s ratio, $\nu$, is taken in what follows to be 0.25. To determine $p_0'$, which equals the initial in situ horizontal stress at pile-toe level, let’s make use of Jaky’s relationship that:
\[ K_0 = 1 - \sin(\phi_{\text{peak}}) \]

Hence: \[ p'_0 = \sigma'_h = K_0 \sigma'_v = K_0 \gamma d. \] We require that \( p_L = q_b \), where \( q_b \) is the pile base resistance during jacking at a given depth, \( d \). Hence \( G_{cav-p_L} \) may be calculated using Equation 6.5.

The input to Equation 6.5 is the pile penetration record of \( q_b \), which equals \( p_L \), vs depth, \( d \); as shown in Figure 6.3 on page 192. The output of Equation 6.5 is a set of linear-elastic shear stiffness \( G_{cav-p_L} \) vs \( d \); which is normalized by \( G_0 \) and plotted, in the right-hand side of Figure 6.4 on page 192, where \( d \) is normalized by pile diameter, \( D \).

Notice there are two different \( G_{cav-p_L} \) vs \( d \) curves for rough and smooth piles because the rough piles experienced higher penetration resistance, due to a positive shaft-upon-base interaction effect. The effect is seen as giving higher values of \( G_{cav-p_L} \) for a rough pile compared to a smooth one.

### 6.1.3.2 Step 2: Large-strain spherical cavity expansion-contraction

First, on the ground of simplicity, let’s make the assumption that at any depth of pile-toe penetration, \( d \), the change of in situ stress around the pile is due to a spherical cavity at depth \( d \), expanding from zero radius to the pile-radius, \( R \). The soil is assumed to be linear elastic-Mohr-Coloumb, as in Step 1 above. The input to this exercise is the values of \( G_{cav-p_L} \) vs \( d \). The rigorous large-strain analytical solution of Yu (2000) will be used to calculate the stress distribution, \( \sigma'_{cav} \), away from the pile toe. For the pile unloading stage, the cavity is modeled to contract elastically until the original virgin in situ horizontal stress is reached.

It is assumed that the stress field introduced by the cavity, \( \sigma'_{cav} \), superimposes upon the initial in situ stresses, \( \sigma'_0 \), to give the current stress field:

\[ \sigma'_v = \sigma'_{v0} + \sigma'_{r-cav} \]  \hspace{1cm} (6.6)

and

\[ \sigma'_h = \sigma'_{h0} + \sigma'_{\theta-cav} \]  \hspace{1cm} (6.7)

This assumption ensures that when the cavity is far away, the in situ stress stays unaltered.

187
**Predicted stiffness distributions** The stress ratio effect was not quantified for Fraction E sand in this thesis. Therefore the following predictions are made without taking the stress ratio into account. Predictions of \((G/G_r)/(\gamma/\gamma_r)^{-0.42}\) were made by evaluating Equation 6.4:

\[
\frac{\sigma'_v^{0.25} \sigma'_h^{0.25}}{\sigma'_{v_0}^{0.25} \sigma'_{h_0}^{0.25}}
\]  

(6.8)

where \(\sigma'_v\) and \(\sigma'_h\) are given by Equation 6.6 and 6.7.

These predictions are plotted as solid lines in Figures 6.5 to 6.10 on page 193 to 197, taking corresponding colors of the solid symbols of which they were making a prediction of. Figure 6.5 shows reasonable agreement of predicted and measured values of in situ stiffness for locations directly beneath the pile during penetration. In this figure the predictions show some sensitivity to the depth of instruments; accelerometers pair 12 is located at approximately 80mm below that of pair 34; the increase in stress measured by pair 12 is higher than that of pair 34 at a fixed normalized distance of \(h/D\) (comparing the black smooth line prediction for pair 12 against the red line for pair 34). However this trend could not be observed in the seismic data due to the relatively large data scatter. Both the prediction and measurement indicates a sharp increase in stress and stiffness when the pile is within 10 diameters away, this observation fits the known boundary condition that as \(h/D \to 0\) in situ stress approaches that of \(q_c\) which is 40 to 50MPa for the tests presented herein.

In Figure 6.6 seismic measurements and predictions are made of in situ stiffness after unloading of pile base load, \(q_c\), to zero. Comparing this figure to the previous figure of Figure 6.5 there is little observable drop in stiffness measurements over the majority range of values of \(h/D\). This observation is supported when we compare Figures 6.7 and 6.8, where significant data scatter and difficulty in data analysis existed as discussed previous, however the trend of stress increase is clearly seen and little stiffness reduction can be observed. However for the data group measured at \(h/D \approx 3\) a small noticeable drop in in situ stiffness measurement can be observed. It is unclear what is behind this drop. One hypothesis may be that it is known that close to failure small strain stiffness is known to decrease by upto 30% from prediction in a triaxial simple compression test (Hoque & Tatsuoka (1998), Hoque & Tatsuoka (2004) and Yu & Richard Jr (1984)); this decrease may continue beyond 30% close to the pile base as shearing progresses beyond peak strength and moves towards strain localization and/or critical state. However
there is no study of small strain stiffness behaviour in this zone of soil state known to
the author.

For the locations to the side of the pile centreline and above the toe level, the
prediction is in worse agreement with data as expected since the process around a pile
shaft is far from a spherical cavity expansion mechanism; however, the prediction is
qualitatively correct in capturing the sharper increase in normalized stiffness as the pile
approaches and \( h \) changes from negative to 0, followed by a slower decrease as the pile
passes by and \( h \) becomes increasingly positive, as seen in Figure 6.9 and Figure 6.10.

6.1.4 Discussions

Clear trends of increasing in situ stiffness with proximity to the pile tip can be seen
during pile penetration; there are only small decreases of in situ stiffness after pile
unloading. This suggests a stiffer pile load-settlement response than could be expected
from an undisturbed soil bed. This increase of in situ stiffness and the stiffer pile
response is believed to be the result of large locked-in residual stresses.

The cavity expansion method is very promising for predicting in situ stiffness and
stress during penetration along a path directly under the pile tip, keeping the stress
ratio effect in mind. The method captures the trend at positions around the pile shaft
and above its toe level, but is unable to provide good quantitative prediction. This
is to be expected, since cylindrical cavity contraction was observed to have happened
around the pile shaft (White (2002)). Its effect on the stiffness and stress fields is not
modeled.

Since in the field \( K_0 \) is generally unknown, a useful assumption in this circumstance
is that \( K_0 = 1 \). The predictions were repeated with this assumption and the results are
presented in Figures 6.11 to 6.14 on page 198 to 201. Good agreement with measured
data is evident. Therefore this method is not especially sensitive to the choice of \( K_0 \).
Figure 6.1: Effect of stress ratio on small-strain stiffness; Hoque & Tatsuoka (2004)
Figure 6.2: Effect of stress ratio on small-strain stiffness; Hongnam & Koseki (2005)
Figure 6.3: Penetration resistance; $d =$ pile toe level bgl; $D =$ pile diameter; red symbols = smooth piles; black symbols = rough piles.

Figure 6.4: Shear modulus and $q_b$.
Figure 6.5: Vertical seismic array data before pile unloading; with $K_0 = 1 - \sin(\phi_{peak})$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.6: Vertical seismic array data after pile unloading; with $K_0 = 1 - \sin(\phi_{peak})$; the smooth lines are predicted values using spherical cavity expansion solutions.

Figure 6.7: Vertical seismic array data before pile unloading; with $K_0 = 1 - \sin(\phi_{peak})$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.8: Vertical seismic array data after pile unloading; with $K_0 = 1 - \sin(\phi_{\text{peak}})$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.9: Horizontal seismic array data before pile unloading; with $K_0 = 1 - \sin(\phi_{\text{peak}})$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.10: Horizontal seismic array data after pile unloading; with $K_0 = 1 - \sin(\phi_{peak})$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.11: Vertical seismic array data before pile unloading, with $K_0 = 1$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.12: Vertical seismic array data after pile unloading, with $K_0 = 1$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.13: Horizontal seismic array data before pile unloading, with $K_0 = 1$; the smooth lines are predicted values using spherical cavity expansion solutions.
Figure 6.14: Horizontal seismic array data after pile unloading, with $K_0 = 1$; the smooth lines are predicted values using spherical cavity expansion solutions.
6.2 Pressed-in pile base load settlement curve

6.2.1 Cavity expansion

Under working conditions, the pile has already been unloaded from the $q_b$ state. Therefore, the pile load test is similar to the re-expansion of a cavity. We will also make the assumption that $q_b = q_c$ for the jacked piles. In the previous section a simple spherical cavity expansion model which can predict the stress distribution below a pressed-in pile was proposed. This is achieved by employing a suitable value of shear stiffness, $G_{cav-p_L}$ as part of its linear elastic-plastic model parameters in order to match the prediction of the cavity limiting pressure, $p_L$, to the pile base resistance, $q_b$. We have implicitly made an assumption that the pile tip resistance, $q_b$ equals the internal pressure of a spherical cavity of a radius equal to the pile radius, $R$.

6.2.2 Base hemisphere

It is observed that in front of a penetrating pile, a zone of crushed soil moves downwards together with the tip (e.g. White (2002)). Its shape is close to a hemisphere, and it represents a boundary between highly-crushed sand and relatively intact sand. Therefore it is reasonable to consider a pile capped by a hemisphere as shown in Figure 6.15 (a) on page 208; the assumption that $q = p$, where $p$ stands for cavity pressure, equates to the assumption that there is no shear stress within the hemisphere and it deforms only by volumetric straining. This assumption simplifies the calculations and avoids the need of a shear-transfer mechanism between the pile base and the cavity. Prominent examples where such a shear-transfer mechanism was postulated are Yasufuku & Hyde (1995) and Yasufuku et al. (2001) for predicting bored pile capacity and settlement behaviour, and Salgado et al. (1997a) for predicting $q_c$ in sand.

6.2.3 Pile base settlement model

We shall denote pile base settlement as $W$; it has two contributions:

1) From the expansion of the cavity.

2) From the volume compression of the hemisphere.

6.2.3.1 Cavity expansion

This contribution is illustrated in Figure 6.15 (b) on page 208. Cavity pressure $p$ causes the cavity to expand by a displacement of $u$, which results in a pile settlement of $W_{cav}$, by assuming volume conservation of the hemishpere, i.e. ignoring the volume
compression at this stage. We now attempt to express this settlement, \( W \), as a function of mobilized pile base stress, \( q \).

For a spherical cavity of radius \( R \) undergoing linear-elastic expansion with shear stiffness \( G_{cav} \), then:

\[
\frac{\pi R^2}{2} = \frac{p}{4G_{cav}} - \frac{q}{4G_{cav}}\frac{R}{2} \tag{6.9}
\]

The conservation of volume of the hemisphere requires that:

\[
u A_{hemi} = W_{cav} A_{pile} \tag{6.10}
\]

or

\[
u 2\pi R^2 = W_{cav} \pi R^2 \Rightarrow u = \frac{W_{cav}}{2} \tag{6.11}
\]

Together they give:

\[
\frac{W_{cav}}{R} = \frac{1}{2} q \frac{R}{G_{cav}} \tag{6.12}
\]

6.2.3.2 Hemisphere compression

This contribution is illustrated in Figure 6.15 (c) on page 208. The hemisphere compresses under the applied pressure \( q \) elastically, governed by the shear stiffness \( G_{hemi} \) and the Poisson’s ratio, \( \nu_{hemi} \), since the bulk modulus \( K \) is related to \( G \) and \( \nu \) by

\[
K = \frac{2(1+\nu)}{3(1-2\nu)} G
\]

Therefore the volume change of the hemisphere is given by:

\[
\Delta V_{hemi} = \frac{q}{K_{hemi}} V_{hemi} \tag{6.13}
\]

where \( V_{hemi} = \frac{2}{3} \pi R^3 \);

Now let’s assume that all this change in volume is translated to a pile settlement of \( W_{hemi} \), so that:

\[
W_{hemi} A_{pile} = \Delta V_{hemi} \tag{6.14}
\]

Then equating Equation 6.13 and 6.14 for \( \Delta V_{hemi} \);
\[ W_{hemi} \pi R^2 = \frac{2/3q\pi R^3}{2(1+\nu_{hemi})G_{hemi}} \]  

(6.15)

which simplifies to:

\[ \frac{W_{hemi}}{R} = \frac{1-2\nu_{hemi}}{1+\nu_{hemi}} \frac{q}{G_{hemi}} \]  

(6.16)

### 6.2.3.3 Total settlement

Now assuming superposition of the two contributions from \( W_{cav} \) of Equation 6.12 and \( W_{hemi} \) of Equation 6.16, giving:

\[ \frac{W}{R} = \frac{W_{cav} + W_{hemi}}{R} = 0.5 \frac{q}{G_{cav}} + \frac{1-2\nu_{hemi}}{1+\nu_{hemi}} \frac{q}{G_{hemi}} \]  

(6.17)

normalizing by \( q_c \):

\[ \frac{W}{R} = (0.5 \frac{q_c}{G_{cav}} + \frac{1-2\nu_{hemi}}{1+\nu_{hemi}} \frac{q_c}{G_{hemi}}) \frac{q}{q_c} \]  

(6.18)

If one defines a non-dimensional pile base stiffness being:

\[ E_{base} = \frac{q/q_c}{W/R} \]  

(6.19)

Then an expression for \( E_{base} \) may be written as:

\[ E_{base} = \frac{1}{(0.5 \frac{q_c}{G_{cav}} + \frac{1-2\nu_{hemi}}{1+\nu_{hemi}} \frac{q_c}{G_{hemi}})} \]  

(6.20)

### 6.2.3.4 The value of \( G_{cav} \)

We have demonstrated in section 6.1 that a substantial residual stress and higher values of small-strain stiffness exist after pile jacking. Therefore, on pile reloading, on average, a much stiffer soil (\( G_{cav-p_L} \)) is at work resisting pile settlement compared to what the virgin ground prior to pile installation can offer (\( G_0 \)): with \( G_{cav-p_L} \gg G_0 \). We assume \( G_{cav} = G_{cav-p_L} \).

\( G_{cav-p_L} \) is obtained by fitting the limit pressure prediction of a spherical cavity to match that of \( q_0 \) or \( q_{c1} \), as was performed in section 6.1. It was shown to vary from
\( G_0 \) at shallow depth to more than \( 3G_0 \) at deep penetration (see the right-hand side of Figure 6.4). \( G_{cav-p_L} \) is calculated using Equation 6.5 on page 186 which is rewritten as:

\[
G_{cav-p_L} = \frac{p'_0}{2} \frac{N - 1}{N + k} [Tq_c^{1+\alpha} - Zq_c^{1-\beta}]
\]  

(6.21)

\( G_{cav-p_L} \) is a function of \( q_c \), in situ stress, soil friction angle and Poisson’s ratio, \( \nu \), which is taken to be 0.25.

### 6.2.3.5 The value of \( G_{hemi}, \nu_{hemi} \)

In the triaxial tests it was found that \( \nu = 0.1 \) is applicable for normally-consolidated sand at small strains; at intermediate strains a value of 0.25 is generally assumed, as was used in the cavity expansion fittings in previous sections. Given the lack of direct measurement, \( \nu \) could take either value. We will show that prediction assuming \( \nu = 0.25 \) is the most satisfactory.

The material inside the hemisphere has been subjected to a very large mean stress and then unloaded to something closer to its in situ stress level. The effect is a reduction in voids ratio, hence an increase in small-strain stiffness. To be able to predict this increase, one would need to measure the critical state line of the material in question following the methodology of Coop (1999). However, one can anticipate the likely change by comparison with the well-studied Ham River sand (Coop (1999)).

For Ham River sand the effect of over-consolidation is expressed as (Figure 3.41 on page 104):

\[
\frac{G_0}{p_r} = A_0 (\frac{p}{p_r})^{n_0} (\frac{pe}{p})^c
\]  

(6.22)

where \( c = 0.13 \) (see Figure 2.5 on page 33).

Therefore the ratio of stiffness before \( (G_0) \) and after \( (G_{hemi}) \) pile insertion becomes:

\[
\frac{G_{hemi}}{G_0} = \left( \frac{p_{hemi}}{p_{init}} \right)^{n_0} \left( \frac{pe-hemi}{pe-init} \right)^c
\]  

(6.23)

where \( p_{init} \) and \( p_{hemi} \) are the mean stress of the virgin ground and its value after pile unloading respectively. After unloading the mean stress \( p \) is assumed to return to its in situ value for the purpose of calculation, i.e. \( p_{hemi} = p_{init} \), therefore:
Given the lack of information on the precise location of the normal compression line, the pile base stress during jacking, $q_c$, is a good estimate of $p_{e-hemi}$, assuming during jacking that the stress state remains on the normal compression line. Making this substitution, gives:

$$\frac{G_{hemi}}{G_0} = \left(\frac{q_c}{p_{e-init}}\right)^c$$

(6.25)

The value of $p_{e-init}$ is strongly influenced by the crushing strength and has a possible value of 10,000kPa as indicated in Figure 3.41 on page 104, where the dilation becomes zero. $q_c$ at the depth of load tests has a value of approximately 40,000kPa. $c$ has the value of 0.13 for Ham River sand, as shown in Figure 2.5 on page 33. Therefore:

$$\frac{G_{hemi}}{G_0} = \left(\frac{40,000}{10,000}\right)^{0.13} = 1.20$$

(6.26)

Hence the increase in small-strain stiffness due to over-consolidation is $\sim 20\%$\(^1\), giving:

$$G_{hemi} = 1.2G_0$$

(6.27)

where $G_0$ is determined from the triaxial data:

$$\frac{G_0}{p_a} = C \left(\frac{p}{p_a}\right)^n$$

(6.28)

where $C = 1000$, $p_a = 100kPa$ and $n = 0.5$.

### 6.2.4 Prediction of pile base stiffness

Recall the expression for pile base stiffness of Equation 6.20 being:

$$E_{base} = \frac{q/q_c}{W/R} = \frac{1}{(0.5 \frac{q}{G_{cav}} + \frac{1-2\nu_{hemi}}{1+\nu_{hemi}} \frac{q}{G_{hemi}})}$$

(6.29)

with the following substitutions:

\(^1\)For illustration purposes, suppose if let’s assume $p_{e-init} = 1MPa$, which must be far too small to be correct, then $\frac{G_{hemi}}{G_0} = 1.60$; therefore producing only a 60% increase in small-strain stiffness.
\[ G_{cav} = G_{cav-p_L} \]  

(6.30)

\[ G_{hemi} = 1.2G_0 \]  

(6.31)

\[ \nu_{hemi} = 0.25 \]  

(6.32)

The normalized stress vs deformation plots of eight pile load tests of similar embedment are shown in Figure 6.16 on page 209, where the measurement was corrected for the pile length shortening; predictions are made with Equation 6.29; the solid green line is for \( \nu_{hemi} = 0.25 \); the black line represents \( \nu_{hemi} = 0.1 \) and the red line for \( \nu_{hemi} = 0.49 \).

6.2.5 Discussions

\( \nu_{hemi} = 0.49 \) models an almost incompressible hemisphere, and the foundation stiffness is over predicted, indicating the importance of hemisphere compression in pile base settlement prediction. \( \nu_{hemi} = 0.25 \) provides a good fit to experimental data.

Because \( G_{cav-p_L} \) increases approximately linearly with depth; so does \( q_c \); \( G_{hemi} \) is proportional to \( G_0 \), hence \( G_{hemi} \propto \sqrt{\text{depth}} \). Therefore \( E_{base} \) is predicted to reduce with depth. This trend is indeed observed in Deeks & White (2006), who have conducted centrifuge tests using a very similar test set-up.
Figure 6.15: Soil hemisphere
Figure 6.16: Pile load test plots and prediction
6.3 Wished-in pile base load settlement curve

We have made use of spherical cavity expansion to model the load settlement behaviour of a pressed-in pile in the previous section. This methodology is adopted below to perform similar calculations for a wished-in pile. The results serve to highlight different foundation stiffness as a consequence of the different installation processes and present a simple method to assess relative performances of pressed-in and wished-in piles.

6.3.1 Introduction

Adopting the same mechanism as proposed for displacement piles presented in the proceeding section, let’s write: \( W = W_{\text{cav}} + W_{\text{hemi}} \) (follows Equation 6.17, 6.16 and 6.11), where

\[
W_{\text{hemi}} = \frac{1 - 2\nu_{\text{hemi}}}{1 + \nu_{\text{hemi}}} \frac{q}{G_{\text{hemi}}} R
\]  

(6.33)

and

\[
W_{\text{cav}} = 2u = \frac{q}{2G_{\text{cav}}} R
\]  

(6.34)

For wished-in piles, the parameters of these equations are different from those of pressed-in piles.

6.3.2 Modeling of cavity expansion

Loading of a bored pile for the first time is akin to the initial expansion of a cavity in virgin ground which is highly non-linear. A rigorous calculation of the expansion can be best achieved using FE and a suitable non-linear material model.

We have used CAMFE with the Fahey-Carter model to predict cylindrical cavity expansion in section 5.3.4, where the non-linear elastic soil stiffness is characterized by \( G_0 \) and a degradation rule. It is proposed here that an equivalent linear-elastic Mohr-Coulomb material, with an equivalent elastic stiffness of \( G_{eq} \), can adequately model cylindrical cavity expansion of the non-linear Fahey-Carter soil.
6.3.2.1 Equivalent model for cylindrical cavity expansion in Fahey-Carter material

For a Fahey-Carter material, it will be shown that the pressure-expansion relationship of a cylindrical cavity can be reproduced very well using a simple linear-elastic Mohr-Coulomb plastic model, by replacing the non-linear elastic part of Fahey-Carter model with a linear-elastic one. We term this linear-elastic Mohr Coulomb model as the equivalent model, and the shear stiffness as the equivalent shear stiffness, \( G_{eq} \).

This equivalent model requires a value for the shear stiffness and the Poisson’s ratio:

1. Poisson’s ratio, \( \nu \), of 0.25 has been used in previous sections and is widely accepted as a first approximation when measurements are unavailable; it is used in the calculations below.

2. The equivalent shear stiffness, \( G_{eq} \), is chosen to give a good curve-fit to the FE result calculated using CAMFE and the Fahey-Carter soil model of Fraction E sand.

The equivalent model makes use of the rigorous large-strain solution given by Yu (2000) for a cylindrical cavity inside a linear-elastic Mohr-Coulomb material.

Solutions using Yu’s formulation and using CAMFE are computed for cylindrical cavity expansions at two different initial cavity pressures, with \( G_{eq} = 0.35G_0 \), which gives the best curve-fitting result. Figures 6.17 and 6.18 on page 215 and 216 show the results with \( p_0' = 37 kPa \); Figures 6.19 and 6.20 on page 217 and 218 for \( p_0' = 137 kPa \) (37 kPa and 137 kPa are arbitrarily chosen, one at a shallow depth and one at a deeper depth). Very good agreement is evident both for stress and strain at the cavity wall, and at one pile radius away from the cavity.

The equivalent linear-elastic shear stiffness, \( G_{eq} \), in the case of cylindrical cavity expansion, is only 35\% of \( G_0 \). This is due to stiffness degradation of the Fahey-Carter material. The precise value of 0.35 is a function of the stiffness degradation property of that material and is expected to be different for different soils; it is also particular to the cylindrical cavity expansion mechanism, and a different value should be expected for other problems, for example, a footing or spherical cavity expansion.

It is unclear how well this equivalent model (\( G_{eq} = 0.35G_0 \)) will perform for a spherical cavity without conducting 3D FE analysis. There is no reason why, for a spherical cavity, the best fit should still remain at \( G_{eq} = 0.35G_0 \). However, developing non-linear FE material models for solving spherical cavity expansion problems is outside the scope of this thesis. The value of 0.35 from the cylindrical expansion result is used in the following calculations, such that:
\[ G_{eq} = 0.35G_0 \]  \hspace{1cm} (6.35)

**6.3.2.2 Prediction of model pile base stiffness**

During the initial stage of expansion, following Equation 6.11, then:

\[ u = \frac{q}{4G_{cav}} R \]  \hspace{1cm} (6.36)

and

\[ \frac{W_{cav}}{R} = \frac{1}{2} \frac{q}{G_{cav}} \]  \hspace{1cm} (6.37)

where \( G_{cav} = G_{eq} = 0.35G_0 \), as Equation 6.35.

For the soil hemisphere, let’s assume \( \nu_{hemi} = 0.25 \) and \( G_{hemi} = G_0 \). Therefore one can predict the initial pile base stiffness using Equation 6.17:

\[ \frac{W}{R} = \frac{W_{cav} + W_{hemi}}{R} = \frac{1}{2} \frac{q}{G_{cav}} + \frac{1 - 2\nu_{hemi}}{1 + \nu_{hemi}} \frac{q}{G_{hemi}} \]  \hspace{1cm} (6.38)

where \( G_{cav} = 0.35G_0; \nu_{hemi} = 0.25 \) and \( G_{hemi} = G_0 \).

This prediction is plotted as a solid green line in Figure 6.21 on page 218. It overpredicts the measured behaviour. The black line is a prediction for an equivalent rough pressed-in pile with:

\[ G_{cav} = 0.2G_0 \]  \hspace{1cm} (6.39)

which provides the best fit to the experimental data. This suggests the equivalent linear shear stiffness in the case of a spherical cavity is less than that of a cylindrical cavity, for a Fahey-Carter material.
6.3.3 Discussions

6.3.3.1 At the limiting design load

As shown in Figure 6.21 on page 218, the pile base settlement behaviour is almost linear until $W/R \approx 0.3$; this is rather similar to the behaviour of shallow footings (Carter (2006)). Pile limiting settlement is usually taken at a settlement ratio of $W/R = 0.05$ or $W/D = 10\%$, which locates on the very beginning of the straight-portion of the curve on Figure 6.21; therefore the slope of it is useful in predicting pile base settlement at its limiting design load.

6.3.3.2 Comparison of pressed-in and wished-in piles

Summarizing the expressions for pile base settlement, following Equation 6.29, 6.38 and 6.39 for both wished-in and pressed-in piles:

$$\frac{W}{R} = \frac{W_{cav} + W_{hemi}}{R} = 0.5 \frac{q}{G_{cav}} + \frac{1 - 2\nu_{hemi}}{1 + \nu_{hemi}} \frac{q}{G_{hemi}}$$ (6.40)

where:

$\nu_{hemi} = 0.25$;

$G_{cav} = 0.2G_0$ for a wished-in pile and $G_{cav} = G_{cav-pl}$ for a pressed-in pile;

$G_{hemi} = G_0$ for a wished-in pile and $G_{hemi} = 1.2G_0$ for a pressed-in pile.

or, written separately for wished-in and pressed-in piles:

$$\frac{W_{wished-in} G_0}{qR} = 2.5 + 0.4$$ (6.41)

$$\frac{W_{pressed-in} G_0}{qR} = \frac{0.5}{G_{cav-pl}/G_0} + 0.33$$ (6.42)

For a wished-in pile, hemisphere compression’s contribution to base settlement is relatively small compared with the cavity expansion mechanism; however for a pressed-in pile it’s comparable with that of the cavity expansion contribution. The large difference between the cavity expansion contribution for the two types of pile accounts for the major difference in performance between them. For example, for rough piles of 160mm embedment, $G_{cav-pl}/G_0 = 2.5^{1}$, giving a prediction of their performance ratio as $2.9/0.53 = 4.7$ in favor of the pressed-in pile. Also Equations 6.41 and 6.42 predict an upper limit of the performance ratio as being approximately $2.9/0.33 = 9$, for comparable wished-in and pressed-in piles.

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1From the right-hand-side of Figure 6.4 on page 192
6.3.3.3 Impact of installation procedure

Contrary to common conception, over-consolidation or soil compaction due to pile driving is by no means the controlling factor behind the performance difference between a pushed-in and a wished-in pile. The important factor is the large locked-in stress and change in soil fabric due to pile jacking, which has increased in situ stiffness to values much larger than $G_0$.

The pile load settlement process can be understood via the simple cavity expansion/contraction mechanism proposed herein. This proposed mechanism is the result of a number of simplifying assumptions and it cannot replace rigorous FE when precise prediction is required. However, it captures the core mechanism of the problem, and is capable of assessing the relative performances between displacement and non-displacement piles. The method should be judged against the following statement:

“Far more than a method of prediction, the designer needs a descriptive mechanism which embodies the working of any geotechnical facility.” Bolton (1993)
Figure 6.17: Comparison of theoretical solution with FE: 1

Stress evolution during expansion and contraction
$G_{eq}/G_{max} = 0.36, \nu = 0.25, p_0 = 37 \text{kPa}$
Figure 6.18: Comparison of theoretical solution with FE: 2
Figure 6.19: Comparison of theoretical solution with FE: 3
Figure 6.20: Comparison of theoretical solution with FE: 4

Figure 6.21: Wished-in pile load tests: base load settlement plots and prediction
6.4 Pile shaft load-settlement curve

6.4.1 Mechanism of pile shaft friction mobilization

The peak shaft load for the rough wished-in piles of this thesis was achieved at $\sim 1$mm displacement, which is in the range of other model test results (Lehane et al. (2005); Fioravante (2002); Garnier (2002); Foray et al. (1998); Latotzke et al. (1998); Fellenius (2002); Gwizdala & Klosinski (1993); Garnier & Konig (1998)). However, field-scale piles mobilize their peak shaft capacity at larger displacements (for example from 2 to 10mm, Krabbenhoft et al. (2006); Mandolini et al. (2002); Zhang et al. (1993)). The ratio between these different values of displacements is the scaling factor required to project centrifuge model piles to their prototype scale equivalents. However, the calculation of the scaling factor is difficult. We will explore these difficulties in the following discussion.

As discussed in the literature review section 2.4.3, the sand-structure interface shear behaviour includes interface dilation and slippage, both are influenced by stress level and interface roughness. For theoretical modeling of shaft friction mobilization, slippage must be accounted for. We will now assess its significance in pile shaft settlement.

Uesugi et al. (1988) has defined the interface slippage, $\delta_1$, as the sum of the shear zone distortion and net sliding, as shown in Figure 2.19 on page 46. He then studied the magnitude of slippage using a simple shear box, dense Seto sand ($D_{50} = 1.82$mm) and a very rough interface with $R_n = 71 \times 10^{-3}$. Their data is reproduced in Figure 6.22 on page 225. By comparing Figure 2.19 and page 225, at peak angle of friction, the slippage accounts for approximately 1/3 of the total displacement; while post-peak at point 3, the proportion has increased to approximately 1/2.

Evgin & Fakharian (1996) have also performed similar experiments using a simple shear box which is specially adapted for interface testing. They used a dense crushed quartz sand ($D_{50} = 0.6$mm) and a medium-rough interface of $R_n = 42 \times 10^{-3}$. Their results for three tests using different normal stresses are reproduced in Figure 6.23 on page 226. Note that the total displacement is the sum of the sliding displacement or slippage, and the shear deformation. The proportion of slippage over total displacement is approximately 1/2 for all three tests. Therefore, interface slippage is large during virgin loading.

\footnote{Definition of $R_n$ is shown in Figure 2.20 on page 46}
6.4.2 Analysis of pile shaft friction mobilization

A well-established model for understanding pile shaft behaviour follows Fioravante (2002) and is illustrated in Figure 6.24 on page 227. Adopting this view, Lehane et al. (2005) presented a semi-empirical method utilizing interface shear box data, non-linear soil models and FE cavity expansion analysis. Their centrifuge test set-up and typical results are shown in Figure 6.25 on page 228. They have concluded that for centrifuge model piles, slippage at the pile-sand interface has an overriding influence on pile settlement. Interface shear tests are required to measure it. Lehane et al. (2005) also demonstrated that the interface dilation in dense sand accompanying the interface slippage is very important for model piles, \(^1\) and represents a significant contribution for field-scale piles \(^2\). Lehane et al. (2005) showed that this increase can be qualitatively reproduced using non-linear soil cavity expansion FE solutions, coupled with shear box data.

Therefore, the task of predicting pile shaft load mobilization may be performed using an iterative procedure following these steps:

1) given a small increment of slippage calculate the value of incremental shaft friction from shear box data;

2) calculate incremental interface dilation w.r.t. the increment of slippage using shear box data;

3) calculate the change of in situ stress w.r.t. the incremental interface dilation using FE cavity expansion analysis;

4) calculate the tangent shear stiffness of the soil for the current in situ stress and shear strain level; and calculate the incremental shear deformation for the incremental shaft friction as calculated in step 1). And the total incremental settlement of this step is simply the sum of the settlements of step 1) and 4).

Given the importance of both slippage and interface dilation for model piles, extensive shear box testing is required, in addition to advanced triaxial testing, in order to allow such a procedure to be employed.

In the field, no sophisticated interface shear technique capable of measuring both interface dilation and slippage is yet available. Therefore, the above prediction method is irrelevant for day-to-day engineering design of large-diameter foundation piles, which

\(^1\) \(\Delta \sigma'_h/\sigma'_{h0}\) ranges between 2 to 14; see Figure 6.26 on page 229

\(^2\) \(\Delta \sigma'_h/\sigma'_{h0} = 40\%\) for a 800mm diameter pile in dense sand and \(\sigma'_{h0} = 55kPa\)
are the concern of this thesis. Its value lies in the back analysis of small-scale model pile tests and small-diameter field installations, e.g., the mini-pile and soil nail.

It is also clear that for centrifuge model pile tests, the scaling factor for pile shaft settlement is a function of pile dimension, sand particle size, surface roughness and stress level.

Interface shear tests were not performed for this thesis. However, simple analytical solutions are available which allow the shear distortion of the far-field soil to be estimated; this serves as a comparison with the total measured settlement, and affords an estimate of the magnitude of slippage. We will examine such a method as proposed by Bolton (1993) below.

**6.4.3 Simple theoretical prediction of shear distortion**

Ignoring slippage, shaft friction acting on the interior of a circular hole produces shear deformation in the continuum which is zero at infinity and increasing to a maximum at the pile shaft. Given a material stress-strain model, for any value of shaft friction, one can calculate the shear deformation inside the continuum and the settlement at the shaft. The simplest case is to assume linear-elastic behaviour, as was done in Randolph (1977). This thesis take into account the soil non-linearity using a method proposed by Bolton (1993).

Randolph (1977) has pioneered a simple axis-symmetric analysis of pile shaft load-settlement model, as depicted in Figure 6.27 on page 230, for a linear elastic material. The same concept is adopted by Bolton (1993) for a non-linear material, and is used to predict pile settlement behaviour that mimics the non-linear stress-strain relation of the material, by assuming a simple power law relationship (e.g., Bolton & Whittle (1999)).

We will follow Randolph (1977) to obtain the governing equation; then solve it by substituting into it the power law stiffness-strain relationship as proposed by Bolton (1993) to obtain a closed-form expression of continuum shear distortion as a function of shaft shear stress.

**6.4.3.1 Governing equation**

Following Randolph (1977), consider the vertical equilibrium of an element of soil in Figure 6.27 on page 230. This yields
\[
\frac{\partial}{\partial r} (r \tau) + r \frac{\partial \sigma'_z}{\partial z} = 0
\]  
(6.43)

where \( \tau \) is the shear stress increment and \( \sigma'_z \) is the vertical total stress increment.

Randolph (1977) has made the simplifying assumptions that \( \tau \gg \sigma'_z \) and simplifies the above equilibrium equation to

\[
\frac{\partial}{\partial r} (r \tau) = 0
\]  
(6.44)

and integrates w.r.t. \( r \) to

\[
\tau = \frac{\tau_R R}{r}
\]  
(6.45)

where \( \tau_R \) is the shear stress increment at the pile shaft and \( R \) is the pile radius. Also by definition:

\[
\gamma = \frac{\tau}{G} = \frac{\partial u}{\partial z} + \frac{\partial w}{\partial r}
\]  
(6.46)

where \( u \) is the radial and \( w \) is the vertical displacement of the soil.

Randolph (1977) has made the simplifying assumptions that the primary displacement will be vertical and the radial movement can be ignored, thus leaving

\[
\gamma = \frac{\tau}{G} = \frac{\partial w}{\partial r}
\]  
(6.47)

and integrating to

\[
W_s = \int_R^\infty \gamma dr
\]  
(6.48)

where \( W_s \) is the shaft settlement.
6.4.3.2 Power law relationship for stiffness and strain

Following the method of Bolton (1993) and assuming a power law stiffness degradation rule following Bolton & Whittle (1999), then:

\[
\frac{G}{G_r} = \left(\frac{\gamma}{\gamma_r}\right)^\alpha
\]

Also, since \(\gamma = \tau / G\), giving:

\[
\frac{\gamma}{\gamma_r} = \left(\frac{\tau}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}}
\]

Substituting this expression of \(\gamma\) for \(W_s\)

\[
W_s = \int_R^\infty \gamma dr = \int_R^\infty \gamma_r \left(\frac{\tau}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}} dr
\]

Since \(\tau = \frac{\tau_r R}{r}\), then:

\[
W_s = \int_R^\infty \gamma_r \left(\frac{\tau_r R}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}} dr = \gamma_r \left(\frac{\tau_r R}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}} \int_R^\infty r^{-\frac{1}{1+\alpha}} dr
\]

and:

\[
W_s = \gamma_r \left(\frac{\tau_r R}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}} \left(\frac{1 + \alpha}{\alpha}\right) \left[\frac{1}{r^{\frac{1}{1+\alpha}}}ight]_R^\infty
\]

6.4.3.3 Evaluating the definite integral

The typical value of \(\alpha\) for Fraction E sand is approximately -0.5 from the seismic and the static triaxial tests; thus \(\frac{\alpha}{1+\alpha}\) is negative, giving:

\[
r^{\frac{\alpha}{1+\alpha}} |_{r \to \infty} = r^{-1} |_{r \to \infty} = 0
\]

and the definite integral may be evaluated as:

\[
W_s = -\gamma_r \left(\frac{\tau_r R}{G_r \gamma_r}\right)^{\frac{1}{1+\alpha}} \left(\frac{1 + \alpha}{\alpha}\right) R^{\frac{1}{1+\alpha}} = \frac{1}{R^{\frac{1}{1+\alpha}}} [\gamma_r R \left(\frac{1}{\alpha} - \frac{1 + \alpha}{\alpha} (G_r \gamma_r)^{\frac{1}{1+\alpha}}\right]
\]
6.4.3.4 Estimated magnitude of shear deformation

For an order of magnitude estimate, let’s assume the pile to be rigid and the distribution of shaft shear stress is uniform. Then:

\[ \tau_R = \frac{Q_s}{2\pi RL} \]  \hspace{1cm} (6.55)

where \( Q_s \) is the shaft load and \( L \) is the length of the pile. This gives:

\[ W_s = Q_s \sqrt{\frac{1}{\pi R} \left[ \frac{1+\alpha}{-\alpha} \frac{(2\pi L G_r \gamma_r)}{(1+\alpha)^2} \right]} \]  \hspace{1cm} (6.56)

Estimated parameter values of Fraction E sand are: \( G_r = 10 \text{MPa}, \gamma = \gamma_r = 0.01, \) and \( \alpha = -0.5 \) (see Figure 3.38 on page 102). The pile geometry dictates that: \( R = 6.7 \text{mm} \) and \( L = 160 \text{mm} \).

Therefore, under \( Q_s = 500 \text{N} \) of loading, one obtains \( W_s = 0.037 \text{mm} \). This value is 1/8 of the measured settlement of \( \approx 0.29 \text{mm} \), as shown in Figure 6.28 on page 231 for a rough bored pile loading test. Bearing in mind the approximate nature of this calculation slippage does appear to dominate, in agreement with the observations of Lehane et al. (2005).

6.4.4 Discussions

We have shown that the contribution of slippage to total shaft settlement is significant for rough model piles and the scaling rule of shaft settlement is a complex function of many factors. This poses difficulties when shaft-base interaction is to be studied.

However for very smooth shafts, no interface shear band is expected; the relative motion of pile and soil post-peak must be entirely due to net sliding, whilst the shaft settlement before the peak is due to shear deformation of the continuum far-field, which can be scaled as usual (i.e. by the factor of the centrifuge acceleration to \( g \)). Therefore, the use of a smooth shaft in the centrifuge may be required if unequivocal scaling to prototype is essential; however, almost all field-scale piles are far from being smooth. This discussion raises the question of how to model shear interfaces correctly in the centrifuge for granular materials.
Figure 6.22: Measurement of interface slippage; Uesugi et al. (1989)
Figure 6.23: Measurement of interface slippage; Evgin & Fakharian (1996)
Figure 6.24: Pile shaft interface model; Fioravante (2002)
Figure 6.25: Centrifuge pile pull-out tests; Lehane et al. (2005)
Figure 6.26: Increase of horizontal in situ stress during pile pull-out; Lehane et al. (2005)
Figure 6.27: Elastic deformation around a pile shaft; Randolph (1977)
Figure 6.28: Wished-in pile shaft load displacement plot
Chapter 7

Conclusions and future work
7.1 Conclusions

This thesis has (1) conducted advanced triaxial testing on a well-used laboratory sand\textsuperscript{1}; (2) developed and tested new dynamic\textsuperscript{2} and static\textsuperscript{3} in situ centrifuge soil testing techniques; (3) made in situ measurement of stiffness and estimated stress fields during pressed-in piling\textsuperscript{4}; (4) made predictions of these fields using a new approach based on spherical cavity expansion theory\textsuperscript{5}; (5) proposed and tested a spherical cavity expansion-contraction mechanism for base load mobilization of pressed-in\textsuperscript{6} and wished-in\textsuperscript{7} piles; and (6) demonstrated the importance of interface slippage in understanding pile shaft load-settlement behaviour\textsuperscript{8}. We will look at each of these areas in turn before summarizing them in a discussion of piling design.

7.1.1 Triaxial testing techniques

The standard use of a radial belt, whereby both seats are rigidly attached to the sample, was shown to be inadequate and a modified radial belt attachment method was proposed and proved effective. The new method has a determined mechanism whose velocity diagram may be used in conjunction with physical calibration tests to provide the calibration factor.

The effect of adopting the “knob-on-flat-plate” docking method was investigated and the sample was shown to follow the expected bending behaviour due to a constant offset of axial load. However, unexpected “kinks” were observed that could not be explained satisfactorily, and should be documented in any triaxial test reports.

7.1.2 Centrifuge mini-pressuremeter

A new miniature pressuremeter was developed for the centrifuge, along with saturation, system calibration, installation, operation and data correction procedures. System compliance was shown to be significant and its calibration mandatory. The design successfully avoided any installation disturbance by adopting a wished-in installation process, and produced repeatable results. However, membrane penetration errors proved

\textsuperscript{1}see Chapter 3
\textsuperscript{2}section 5.4
\textsuperscript{3}section 5.3
\textsuperscript{4}section 6.1
\textsuperscript{5}section 6.1.3
\textsuperscript{6}section 6.2
\textsuperscript{7}section 6.3
\textsuperscript{8}section 6.4

233
severe for the current design at small strains (say from $10^{-6}$ to $10^{-3}$). For future designs, “Chinese lanterns” of the Cambridge In-Situ design, as assessed by Cunha & Campanella (2005), should be adopted to reduce the penetration problem. However, the design as it stands is expected to provide accurate measurement in fine-grained soils or at large strain (say $> 5\%$) in fine sand.

### 7.1.3 Centrifuge seismic method

The large-strain cross-hole method of Salgado et al. (1997a) was successfully implemented in the centrifuge. Uncertainties in wave travel path length are accounted for by normalization. Small-strain stiffness and its dependency on mean stress level were successfully measured and compared well against data obtained from triaxial tests. The dynamic shear stiffness-strain relationship was also estimated assuming a power law constitutive model of Bolton & Whittle (1999), and gave reasonable comparison with the static data obtained from triaxial tests. The design of the seismic test followed closely of the large-strain cross-hole field testing technique of Salgado et al. (1997a) and Drnevich et al. (1995). The results provide a comprehensive validation of this type of field method (measurement of stiffness at strain levels $> 10^{-6}$ by generating high-amplitude seismic waves in situ) in a controlled laboratory environment.

The in situ shear stiffness field around a penetration pile was measured directly using the seismic method both during pile jacking and after pile unloading. An estimate of the corresponding in situ stress field was made from the stiffness field, and possible errors due to the large stress ratio were assessed based on literature evidence (e.g. Yu & Richard Jr (1984), Hoque & Tatsuoka (1998) and Hoque & Tatsuoka (2004) as discussed in section 6.1.1.1). Stress and stiffness increase was first detected when the pile toe was between 15 and 20 pile diameters away from the measuring station. They increased sharply when the distance was within 10 pile diameters. When the separation was 5 pile diameters, the measured shear stiffness rose to between 1.4 to 1.7 times the virgin in situ value; for distances smaller than 5 pile diameters the trend is expected to continue, because mean stress must reach that of $q_c$ at zero distance. During the staged jacking process, after every pile unloading stage, the shear stiffness reduced by a small amount, only just noticeable given the large scatter of the data. This gives direct evidence of stiffer soil behaviour after pile jacking and demonstrates the existence of large locked-in stresses. These higher values of shear stiffness (approximately 2 to 3 times of virgin in situ value at 3 to 5 pile diameters away from pile base) and large
locked-in stresses (approximately 5 or more times of virgin in situ value at 3 to 5 pile diameters away from pile base) are the main reason behind the stiffer load-settlement behaviour of a displacement pile vs a non-displacement pile.

7.1.4 Prediction of in situ stress during pile jacking

A new approach was developed to predict in situ stresses directly below the pile tip during monotonic jacking in dense silica sand. It is based on a simple spherical cavity expansion solution assuming a linear-elastic Mohr-Coulomb material model. The measured pile tip resistance, \( q_b \), is the input to the model, and values of the linear-elastic shear stiffness are chosen so that the theoretical cavity limit pressure matches that of \( q_b \) at a certain depth. Once the shear stiffness is found, in situ stress and stiffness can be calculated using large spherical cavity expansion solutions and the standard relationship between small-strain stiffness and stress. The predicted in situ stiffness field compared well with the measured values from the seismic method for locations directly below the pile toe. This method also predicts large locked-in stiffness and stress after pile unloading.

The method is not sensitive to the choice of \( K_0 \), making it an attractive design method to provide first-order estimates in uniform dense silica sands. The method is semi-empirical and its applicability to other types of soils has not been investigated. However, the pile-driving induced stress is expected to be the highest, and the influence zone the largest, in dense non-crushable sand, compared with carbonate sands or clays.

7.1.5 Pile base load-settlement prediction

A simple base load mobilization mechanism was proposed for both displacement and non-displacement piles. Pile base settlement is separated into two components: (1) compression of a hemispherical soil tip cone; (2) spherical cavity expansion outside the tip cone. For a displacement pile, the cavity expansion process is visualized as the reloading curve of an unload-reload loop; in contrast for a non-displacement pile, the process is seen as the initial expansion of virgin ground. The model is able to account for the much larger mobilized pile base resistance of a pressed-in pile over a wished-in pile at the limiting design settlement of 10%D. The difference is attributed to higher soil stiffness due to high locked-in stresses introduced by the pile jacking and unloading processes. The effect of soil densification is shown to be small.
The method is semi-empirical in nature, and its applicability to other types of soil was not investigated. While the method in its current state should not be used to provide a rigorous prediction of pile base stiffness, it may be used to estimate pile performance or evaluate relative pile performance of different installation processes, provided \( q_b \) can be estimated during construction, such as by equating \( q_b \) to CPT resistance \( q_c \).

### 7.1.6 Evaluating model pile shaft data and shaft-upon-base interaction

Pile shaft settlement was shown to consist of interface slippage and the continuum shear deformation of the far field. The method of Bolton (1993) was used to estimate the magnitude of the continuum shear deformation, taking account of soil non-linearities. The estimate was compared with the measured total settlement in the centrifuge tests. It was found that for rough wished-in model piles employed in this thesis, the contribution of slippage was overriding. This supports the opinion that the scaling factor of a model pile shaft is a complex function of both interface properties and in situ stress.

Using rough shafts, compared with smooth ones, a 10% increase in pile base resistance during pile jacking was measured. This suggests that the effect of shaft-upon-base interaction is small for the model piles considered in this thesis and lends support to the current design practice where the pile shaft and base are treated independently. However this result may not be valid in other type of soils. The ambiguity of scaling laws governing shaft settlement also makes direct generalization of the above statement to prototype scales difficult.

### 7.2 Discussions and recommendations for future work

#### 7.2.1 Development of centrifuge soil testing techniques

The air hammer is not a precise wave generator; the magnitude of signals cannot be easily controlled. More precise and controllable seismic sources should be employed in future applications. The scatter of results is expected to improve significantly once an optimum wave form can be generated and its magnitude controlled precisely. Higher logging frequency with respect to the frequency of the signal should be used to improve data quality.
The size of the accelerometers used in this thesis is excessive compared to their spacings. Miniature accelerometers (< 5mm) are now available; their use will allow more confident measurement of $G_0$.

Dense arrays of accelerometers and seismic wave sources can be employed in many situations, in addition to pressed-in piling investigated in this thesis. For example, it is expected that they will prove to be a sensitive tool for lower stress applications, to study soil fabric development during (1) dynamic soil compaction, (2) loading of footings and embankments, (3) dynamic/cyclic foundation loading, (4) tunneling or tunnel collapse, (5) excavations, and (6) during and after earthquakes. However, the method is most versatile in sand, where rate effects are small. Future researchers are encouraged to conduct cyclic simple shear tests to further validate this method.

The miniature pressuremeter has a robust and flexible design. It was configured for use at small to intermediate strains in this thesis. However, it is very easy to replace the glass capillary tubes with ones of larger internal diameter so that it can operate up to the cavity limiting pressure. In its current form, there is virtually no limit on the number of probes placable in a single sample of sand. Future researchers do need to overcome the membrane penetration problem in sand, however.

### 7.2.2 Large-strain seismic field testing

This thesis provided a comprehensive validation of the approach first published by Salgado *et al.* (1997a). It is hoped that this will encourage companies to adapt their cross-hole, down-hole and SCPT equipments to make available large-strain dynamic measurements to their clients without resorting to expensive non-disturbed sampling and lab element testing.

The application of the seismic method, coupled with self-boring pressuremeter testing, would be able to measure many of the soil parameters currently deemed important for foundation design, offering a complete in situ-based soil investigation strategy. This thesis has studied such a scenario for piled foundations; the suite of in situ tests recommended are:

1. CPT to obtain $q_c$, providing soil classification, stratification and indirect estimate of soil state and strength parameters etc.

2. Self-boring pressuremeters, measurement of soil strength parameters and stiffness.
(3) Large-strain seismic method, direct measurement of small-strain stiffness and dynamic stiffness degradation curve with strain.

Geotechnical processes that involve many cycles of loading, for example, pile driving or pile jacking with numerous short strokes, should be examined using both dynamic and static soil properties so to ascertain whether the dynamic or the static behaviour, or something in between, must be used in design. And the same suite of tests can fulfill site investigation requirements of a wide range of foundation design scenarios.

7.2.3 Assessing the impact of geotechnical processes on pile performance: THINK STRESS!

We have demonstrated the major effect of locked-in stress and stiffness on pile load-settlement behaviour. Therefore whether a construction process creates or destroys locked-in stress and stiffness has an important influence on foundation performance. Therefore, the effectiveness of a geotechnical process in creating/destroying locked-in stress needs to be assessed.

Given the importance of in situ stress to foundation performance under working load, different long and short-term scenarios that may alter it should be investigated in the lab using in situ stress and stiffness monitoring. Examples are (1) tunneling-induced pile bending and settlement, (2) ground heave due to deep excavations, (3) effect of piling at small spacings, (4) pile cyclic loading and (5) earthquakes; all of these processes have the potential to reduce in situ stress or destroy existing locked-in stress, inducing unexpected settlement.

The impact on in situ stress and void ratio related to different geotechnical processes can be visualized using diagrams similar to Figure 7.1 on page 240. We have demonstrated the difference in stiffness of a material under monotonic loading versus that under cyclic loading via the comparison between the results of large strain seismic method and monotonic triaxial test. The practical implication in the appropriate choice of design parameters for different geotechnical processes remains to be studied.

7.2.4 Strategy for the reuse of foundations

With the emphasis on the reuse of piled foundations, a promising approach is to investigate geotechnical processes that can enhance in situ stress and create locked-in stress, for example by compaction grouting or re-driving; both approaches can be investigated in the centrifuge as well as in the field. Pile base grouting and pile driving can be idealized as spherical cavity expansions, and their effect on in situ stresses can
be estimated assuming cavity expansion solutions. In the field, the statnamic testing method of a pile is especially convenient for studying performance improvements of piles by different re-driving efforts, since the test itself simulates pile re-driving and at the same time provides a measure of its performance.

7.2.5 Implications to piling design and construction practices

It is unlikely that, in the immediate future, sophisticated in situ testing techniques, such as the self-boring pressuremeter or the large-strain cross-hole method will be employed purely to facilitate piling design, unless extraordinary prestige or novelty is attached to a project. Such projects are rare. Therefore it is likely that during the short term the application of the seismic CPT in routine engineering projects will become widespread. Thus the designer will have at his hand $q_c$ and $G_0$ profiles, in addition to the useful suite of lab results using remoulded samples, such as the angle of frictions and triaxial sample stiffness at large strains. Hence the designer will not be in a position to determine soil stiffness degradations accurately. This single drawback will prevent him from utilizing much of the recent theoretical advances, such as those presented in this thesis, to accurately predict pile performances. The status-quo of piling design is empiricism which will not be broken until the above stated sophisticated in situ testing techniques become affordable.

Meantime, the theoretical advances presented will serve as a guide to piling engineering practices. One striking observation of this thesis is the significance of the locked-in stress and stiffness due to displacement piling method, which does not disappear upon pile unloading. The working load-settlement behaviour of a pile depends on its installation process enormously. The obvious implication is whether one can quality-control the important aspects of our installation procedures and the need to clarify our understanding of what aspects of these procedures are most crucial.

One class of piling technique comes to mind, which can serve as a perfect example, being the full-displacement cast-in-situ technique, e.g. screw piles. Modern piling rigs can record the penetration rate of the auger as well as the rate of rotation, and both parameters can be automatically controlled. Originally the industry found great benefits of advancing the auger-head at such a rate with respect to the rate of auger rotation so to displace the soil sideways instead of excavating them, to produce zero spoil, thus having to pay nothing for spoil removal. With what was discussed in this thesis, one may expect better load-settlement behaviour of a screw pile compared to a
rotary-bored one, due to its soil-displacement installation process which creates locked-in stress and stiffness which does not disappear upon auger retrieval. The question remains whether one can predict the magnitude of this improvement given a particular penetration/rotation ratio and whether one can, in practice, achieve such a ratio with our rigs and current practices and three: given the unavoidable deviation from this ratio from time to time, can the engineer propose a workable quality-control program to reassure our clients? Same question applies also to the driven-cast-in-situ and continuous-flight-auger techniques.

Figure 7.1: Geotechnical processes
References


248


SAGLERAT, G. (1972). The penetrometer and soil exploration. 19, 20, 27, 28


251


Zhao, Y., White, D.J. & Bolton, M.D. (2006b). In-situ seismic monitoring during centrifuge tests of jacked piles. In *International conference on physical modeling in geotechnics, ICPMG 06’,* Taylor and Francis, Hong Kong. 185