Experimental and numerical investigations into the behaviour of deep foundations in sand

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This thesis is presented for the degree of Doctor of Philosophy

School of Civil, Environmental and Mining Engineering
(Geotechnical Engineering)

2017
Declaration

I, Ahmad Bagbag, certify that:

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Ahmad Bagbag

Date: 20/09/2017
Abstract

A key task of a geotechnical engineer is to design foundations so that they undergo sufficiently small deformation in order to satisfy serviceability criteria for the superstructures they support (e.g. buildings, bridges, roadways, conveyors, pipelines etc.). Therefore, designing safe and cost-effectiveness foundations relies on accurate methods for forecasting foundation settlements.

Numerous methods for predicting the settlement of foundations on sand have been published over last few decades. Despite this, a series of foundation settlement prediction exercises has demonstrated that engineers cannot predict foundation settlement to an acceptable level of accuracy. This may be attributable to the fact that the vast majority of settlement prediction methods require an estimate of a “homogenised operational” stiffness representing the soil mass, which is relevant to foundation settlement; and most methods attempt to derive this value using empirical correlations to penetration tests (e.g. CPT and SPT). The stiffness of soil, however, is well known to depend on the combined influence of many factors such as strain level, stress level, density, stress history, anisotropy and ageing.

The need to estimate the “homogenised operational” stiffness can be avoided by using the finite element method incorporating a suitable soil constitutive model. This is a more rational approach, as the response to different stress and strain conditions (as well as density, stress history, anisotropy and ageing) within the soil mass can be modelled. The challenge then becomes choosing an appropriate constitutive model and deriving its parameters. This is often not an easy task and cannot usually be done with confidence because soil models are invariably developed from triaxial data and very few studies have examined the ability of soil constitutive models to extrapolate from triaxial data to more complex boundary value problems (e.g. foundation response).

In this thesis a series of laboratory-scale footing tests is conducted at typical in-situ stress levels to explore the full non-linear pressure-settlement relationship of deep footings in sand. One of the key distinguishing elements of this study is that the footing experiments were performed on reconstituted fine silica sand that was recreated in an identical fashion to that employed for a parallel series of triaxial, cone penetration tests (CPTs) and
pressuremeter tests. The comparison between these tests and the footing experiments is therefore not hindered by differences between the performance of the in-situ sand and that inferred from tests on sand reconstituted in the laboratory. It also enables the ability of soil constitutive models to extrapolate from triaxial element test data to more complex boundary value problems, involving very different stress paths under ideal conditions. Studies such as this are rare, but are a fundamentally important way of assessing the predictive capability of constitutive models and the influence various components of the models have on predicted responses.
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Firstly, I am very grateful to my supervisors Dr James Doherty and Professor Barry Lehane who have generously provided their guidance, support and time for the entire period of my study. My experience has been fantastic and I appreciate all the effort they have put into assisting me to completion.

I would like to thank the structures laboratory technicians (Jim Waters and Bradley Rose), for their assistance in the workshop. They also, assisted me in forming the new footing load transferring frame and the construction of the reaction frame for the pressuremeter and the footing testing. They also, assisted me in operating heavy machinery including heavy dead load on the chamber when it was necessary.

I would also like to extend my thank to the soil laboratory technicians (Usha Mani, Claire Bearman and Yaurel Guadalupe-Torres) for assisting me to set-up the triaxial test and they also helped me to do calibration for the triaxial testing when it was necessary.

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I would like to extend my thanks to my parents, my wife and my wonderful children who helped me relieve from the stress of the work I did.

Finally, and most importantly, I would to thank the Government of Saudi Arabia (King Abdullah scholarship program) who granted me such an opportunity to study at great University like UWA.
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Chapter 1  Introduction

1.1  Background and aim

There is considerable evidence to show that the geotechnical engineering profession is unable to predict ground deformations satisfactorily. This is clearly demonstrated by the results from a recent foundation settlement prediction competition conducted by Lehane et al. (2008). The competition involved loading four square concrete footings, whilst digitally recording the vertical settlement. Prior to the load-tests, engineers from around the world were invited to submit predictions of the load-settlement response. All participants were provided with high quality in-situ test data, as well as triaxial test data on reconstituted sand from the site. Figure 1.1 shows a comparison of the predicted and measured settlements for two (typical) cases. The predictions range from less than 1mm to 85mm for a measured settlement of 13mm, and around 1mm to 87mm for a measured settlement of 31mm. Clearly current prediction capabilities, for what is arguably the most elementary problem in geotechnical engineering, are very poor.

![Figure 1.1 Summary of prediction results](image)

On detailed review of the 26 submissions, it was found that over 15 different predictive methods were employed; this reflects the absence of a well-accepted method for this problem. The most popular methods were the Cone Penetration Test (CPT)-based Schmertmann (1970) method and the Standard Penetration Test (SPT) based method of Burland and Burbidge (1985), both of which use penetration test data to deduce a single “homogenised” stiffness value for the entire soil mass. The popularity of the Burland and
Chapter 1

Burbidge method is remarkable, given that no SPT data were available! A minority of participants made use of the available shear wave velocity, triaxial data and dilatometer test (DMT) data to deduce a best-estimate “homogenised operational” linear elastic stiffness value. Only three submissions (# 1, 6 and 8) used the finite element method, employing either linear elastic or elastic perfectly plastic soil models. As can be seen from Figure 1.1, these submissions varied significantly, with # 6 and # 8 being amongst the poorest. This clearly demonstrates subjectivity associated with soil parameter evaluation.

Given the relatively simple site stratigraphy and the availability of a comprehensive set of high quality in-situ and laboratory test data, the poor predictions cannot be attributed to site variability or insufficient information. Indeed, Briaud and Gibbens (1997) conducted a similar settlement prediction competition, and similarly poor results were reported. The poor performances are attributed to the fact that all methods require an estimate of a “homogenised operational” stiffness representing the entire soil mass, which is (hopefully) relevant to foundation settlement. Most methods attempt to derive this value using empirical correlations to tests measuring the ease with which a conical object penetrates the ground (e.g. CPT and SPT). The stiffness of soil, however, is well known to depend on the combined influence of many factors such as strain level, stress level, density, stress history, anisotropy and ageing. Therefore, it is not at all surprising that empirical relationships between operational stiffness and penetration tests are not reliable. Similar foundation prediction exercise have also been conducted on clays and similarly poor results have been found (Doherty et al., 2017; Lehane, 2003).

The need to estimate the “homogenised operational” stiffness can be avoided by using the finite element method incorporating a nonlinear elasto-plastic soil constitutive model. This is a more rational approach, as the response to different stress and strain conditions (as well as density, stress history, anisotropy and ageing) within the soil mass can be modelled. The challenge then becomes choosing an appropriate constitutive model and deriving its parameters. The quality of the prediction will largely depend on the quality of the test data that is available, how well the constitutive model is calibrated using the data, and the ability of the constitutive model to extrapolate from the test data to more general boundary value problems.

In this thesis a series of laboratory-scale footing tests were conducted at typical in-situ stress levels to explore the full non-linear pressure-settlement relationship of deep
footings in sand. One of the key distinguishing elements of this study is that the footing experiments were performed on reconstituted fine silica sand that was recreated in an identical fashion to that employed for a parallel series of triaxial, CPTs and pressuremeter tests. The comparison between these tests and the footing experiments is therefore not hindered by differences between the performance of the in-situ sand and that inferred from tests on sand reconstituted in the laboratory. The aim of the study is to explore the ability of the soil constitutive model to extrapolate from element test data to more complex boundary value problems, involving very different stress paths, without uncertainties associated with the state or fabric influencing the results. Studies such as this are rare but are a fundamentally important way of assessing the predictive capability of constitutive models and the influence various components of the models have.

Tests were performed using a manufactured fine-medium sand supplied by Sibelco (2017). This sand (or minor variants of it) has been used extensively in geotechnical centrifuge testing at the University of Western Australia (Govoni et al., 2006; Lee et al., 2013; O’Loughlin & Lehane, 2003; Teh et al., 2008; Xu & Lehane, 2008). However, despite its widespread use as a standard geotechnical laboratory sand, very few studies have examined the fundamental mechanical behaviour of the material. Therefore, a further aim of this thesis is to address the shortage of high quality test data on this laboratory sand.

1.2 Thesis outline

This thesis is prepared as a series of papers that have either been published or submitted for publication. Therefore, some repetition exists throughout the thesis. To avoid more repetition, a general literature review has been omitted in the thesis as the background literature relevant for each topic is discussed in each paper. Each paper forms a chapter of this thesis as presented below.

  This paper presents the derivation of Hardening Soil-Small (HSS) constitutive model parameters for a reconstituted sand using results from triaxial tests. The
model is shown to be unable to capture the observed dependence of secant soil stiffness on stress level. Finite Element analyses of model footing tests conducted using the same sand in a laboratory pressure chamber are presented and it is shown that reasonable predictions of the footing response under load can be obtained provided that the selected HSS parameters lead to operational stiffness moduli consistent with the initial stress level in the vicinity of the footing.


  The pressuremeter is a well-known geotechnical test, used to measure soil strength and stiffness. In this paper, a miniature pressuremeter device, developed at the University of Western Australia (UWA), was employed to measure the stress-strain behaviour of dense fine silica sand at a range of stress levels. Back-analysis was performed using the Finite Element method and a widely accessible and popular constitutive model, referred to as the HSS or Hardening soil model (Schanz et al., 1999) with a small strain overlay (Benz, 2007). The HSS model was found to provide a reasonable match to the measured stress-strain response using parameters derived from triaxial compression tests for dense sand.


  The paper presents the results and interpretation of triaxial compression tests for a uniformly graded fine to medium siliceous sand. These results are used together with data obtained from boundary value problems to provide an objective assessment of the predictive capabilities of the HSS model. The triaxial compression tests, which involved a range of relative densities and stress levels, are used to derive model parameters providing the optimal fit to the triaxial experiments. Measurements obtained in pressuremeter and footing experiments
performed in a laboratory testing chamber with the same reconstituted sand are then compared with finite element analyses of these experiments using the selected model parameters. These comparisons combined with a review of simulations for the triaxial tests are used to draw conclusions related to the application of the HSS model in sand.


The paper examines methods for predicting the settlement of deep footings in reconstituted sand. Results from vertical load tests on deep circular plates within a laboratory pressure chamber are interpreted using data from triaxial tests, CPTs and pressuremeter tests obtained for the same reconstituted sand. A simple non-linear CPT-based relationship is shown to match the response observed in the plate tests and be consistent with finite element analyses as well as other comparable physical tests. The relationships between foundation stiffness, and the sand’s small strain stiffness and its response to pressuremeter loading are also explored. Comparisons with full scale tests in the field reveal a strong effect of ageing on foundation stiffness, which appears to be better captured by small strain stiffness than CPT end resistance. Measurements confirm that vertical loading of a deep plate is analogous to the expansion of a spherical cavity.


In this study, a simple shear hardening soil constitutive model is calibrated using triaxial compression data on reconstituted laboratory sand. The model is then applied to simulate the response of miniature pressuremeter tests in the same material, reconstituted in the same way. The ability of this simple constitutive model to extrapolate from triaxial stress paths to more complex boundary value problems, without differences in soil state or fabric, is assessed. Two forms of the
plastic potential in the deviatoric plane are considered, both giving identical responses in the triaxial stress space.

- **Chapter 7:** Presents some concluding remarks from the research project and puts forward recommendations for future studies.

- **Appendices** includes some information that are not presented in the main text of this thesis. A summary is shown in Table 1.1.

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<td>This appendix explained the procedure and results in details for basic soil properties such as particle size distribution, determination of specific gravity, minimum and maximum soil density.</td>
</tr>
<tr>
<td>B</td>
<td>Triaxial Tests</td>
<td>This part explained the results of all triaxial tests conducted during the period of this research.</td>
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<tr>
<td>C</td>
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<td>Experimental Results for Pressuremeter Tests</td>
<td>This appendix shows the result for individual pressuremeter test divided into categories. The category depend on the state of vertical stress on the sample during the test and the relative density of the soil sample.</td>
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Chapter 2  Derivation of hardening soil model properties from triaxial tests

Abstract

This paper presents the derivation of Hardening Soil-small (HSS) constitutive model parameters for a reconstituted sand using results from triaxial tests. The model was unable to simultaneously capture the small strain and large strain stiffness. Finite Element analyses of model footing tests conducted using the same sand in a laboratory pressure chamber are presented and it is shown that reasonable predictions of the footing response under load are obtained provided that the selected HSS parameters lead to operational stiffness moduli consistent with the initial stress level in the vicinity of the footing.

2.1 Introduction

The most popular model used to predict soil behaviour is the linear elastic perfectly plastic Mohr-Coulomb model. Soil is, however, well known to be a highly non-linear material with a linear range that only extends to a strain of about 0.001% to 0.05%. Consequently, the prediction of boundary value problems with the assumption of linearity leads to estimates of ground movements that are primarily a function of, empirically assessed, equivalent linear moduli of the soil. There is no rational basis for selection of this modulus and therefore Engineers now employ non-linear elasto-plastic models more frequently. The Cam-Clay constitutive model, incorporating an elastic stiffness modulus and plastic hardening stiffness modulus, has been a popular model choice for soft clays, for which deformations at smaller strains are less significant (Hashiguchi, 1993). The hyperbolic model, also called the Duncan and Chang model (1970) was one of the first non-linear elastic models used for sands, where pre-failure deformation is dominated by non-linear stiffness. Schanz et al. (1999) followed by Benz (2007) developed the Hardening Soil Small (HSS) model, that overcame some of the limitations of the Duncan and Chang model (1970).

This HSS model is assessed in this paper by using data from triaxial tests performed on a reconstituted sand to determine the HSS parameters and then using these parameters in a Finite Element analysis to predict the response of a deep footing founded in the same
sand. Insights into the predictive capability of the HSS model are obtained by comparing these predictions with the footing response measured in pressure chamber tests.

2.2 Hardening Soil Small (HSS) model

The Mohr-Coulomb failure surface is employed by the HSS model and is defined by the friction angle, dilation angle and cohesion, \( \phi_p' \), \( \psi_p' \) and \( c' \). The other parameters are listed in Table 2.1 and described in detail by Schanz et al. (1999). The model parameters presented in Table 2.1 derivation and calculation method is explained in Sections 2.4 and 2.5, respectively. The model improves upon the Duncan & Chang model by using the theory of plasticity instead of elasticity, and introducing soil dilatancy and a yield cap. The hyperbolic response is controlled by the secant Young’s modulus \( E_{50} \) at 50% of the mobilised strength from initial isotropic stress state in triaxial compression as expressed in Equation (2.1). A reference stiffness modulus \( E_{50}^{ref} \) is used to allow for the stress level dependent nature of soil stiffness. An unloading and reloading stiffness modulus, \( E_{ur} \) (see Equation (2.2)), is employed to allow for the stiffer response of overconsolidated soil. Khoiri and Ou (2013) suggested that \( E_{ur}^{ref} \) is typically three times \( E_{50}^{ref} \) and this ratio is incorporated in the assumptions in this paper. To define the cap yield surface, a stress dependent oedometer modulus is used, \( E_{oed} \), and expressed as a function of a reference oedometric stiffness; (see Equation (2.3)). In keeping with the recommendations of Schanz (1998) after a study of a variety of sands, the value of \( E_{oed}^{ref} \) is taken equal to \( E_{50}^{ref} \); this assumption is widely employed by users of the HSS model (Mathew & Lehane, 2013).

\[
E_{50} = E_{50}^{ref} \left( \frac{\sigma_3'}{p^{ref}} \right)^m \quad \text{when } c' = 0 \quad (2.1)
\]

\[
E_{ur} = E_{ur}^{ref} \left( \frac{\sigma_3'}{p^{ref}} \right)^m \quad \text{when } c' = 0 \quad (2.2)
\]

\[
E_{oed} = E_{oed}^{ref} \left( \frac{\sigma_1'}{p^{ref}} \right)^m \quad \text{when } c' = 0 \quad (2.3)
\]

The power \( m \) defines the stress dependency level of stiffness. Stresses are normalised by a reference stress, \( p^{ref} \), which in this paper is set equal to atmospheric pressure (100kPa).
The $E_{50}$ and $E_{ur}$ values are described as a function of the minor principal effective stress ($\sigma'_3$) whereas $E_{oed}$ is given as a function of the major principal effective stress ($\sigma'_1$). The behaviour at very small strains is modelled using two additional parameters, namely $G_0^{ref}$ and $\gamma_{0.7}$ (see Table 2.1). The very small strain (or elastic) shear modulus is described in the same fashion as the other moduli in Equation (2.4). The value $\gamma_{0.7}$ is the shear strain at which the shear modulus drops to 70% of the elastic small strain value ($G_0$).

$$G_0 = G_0^{ref} \left( \frac{\sigma'_3}{p^{ref}} \right)^m \quad \text{when } c' = 0 \quad (2.4)$$

### Table 2.1 Hardening Soil Small Parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Description</th>
<th>M1</th>
<th>M2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_r$</td>
<td>Relative Density</td>
<td>0.70</td>
<td>0.70</td>
</tr>
<tr>
<td>$e$</td>
<td>Void ratio</td>
<td>0.54</td>
<td>0.54</td>
</tr>
<tr>
<td>$e_{max}$</td>
<td>Void ratio limits</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>$e_{min}$</td>
<td>Void ratio limits</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>$m$</td>
<td>Power for stress-level dependency of stiffness</td>
<td>0.5</td>
<td>1</td>
</tr>
<tr>
<td>$E_{50}^{ref}$</td>
<td>Reference secant stiffness modulus at 50% of the failure load corresponding to the reference stress, $p^{ref}$</td>
<td>50MPa *</td>
<td>70MPa</td>
</tr>
<tr>
<td>$E_{oed}^{ref}$</td>
<td>Reference tangent stiffness for oedometer loading modulus corresponding to $p^{ref}$</td>
<td>50MPa</td>
<td>70MPa</td>
</tr>
<tr>
<td>$E_{ur}^{ref}$</td>
<td>Reference unload-reload stiffness modulus corresponding to $p^{ref}$</td>
<td>150MPa</td>
<td>210MPa</td>
</tr>
<tr>
<td>$c'$</td>
<td>Cohesion</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Friction angle</td>
<td>38.5°</td>
<td>38.5°</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Dilation angle</td>
<td>10°</td>
<td>10°</td>
</tr>
<tr>
<td>$K_0^{nc}$</td>
<td>$K_0$-value for normal consolidation</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$p^{ref}$</td>
<td>Reference pressure (atmospheric pressure)</td>
<td>100kPa</td>
<td>100kPa</td>
</tr>
<tr>
<td>$R_1$</td>
<td>Failure ratio of deviatoric stress at failure over deviatoric stress at Asymptote</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>$v_{ur}$</td>
<td>Poisson’s ratio for unloading-reloading</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>$\gamma_{0.7}$</td>
<td>Shear-strain at 70% of small-shear modulus, $G_0$</td>
<td>2.8×10^{-5} *</td>
<td>2.8×10^{-5} *</td>
</tr>
<tr>
<td>$G_0^{ref}$</td>
<td>Reference small-strain shear modulus</td>
<td>185MPa</td>
<td>262MPa *</td>
</tr>
</tbody>
</table>

* Values selected to match measured data at $\sigma'_3=50kPa
2.3 Triaxial testing

Three consolidated drained triaxial tests were conducted on reconstituted silica sand at the University of Western Australia (UWA). This sand has been used extensively in centrifuge testing at UWA over the past 20 years (e.g. Lehane et al., 2005; Xu & Lehane, 2008). The mean effective particle size of the sand \((D_{50})\) is 0.29 mm and its minimum and maximum void ratios are 0.45 and 0.75 respectively (see Appendix A).

O’Loughlin and Lehane (2010) completed model scale footing tests in the centrifuge using the same sand, but these were not accompanied by high quality triaxial tests. For the tests reported in this paper, bender elements located at the top and bottom of the samples enabled measurement of shear wave velocity, which was converted into small strain stiffness values using the relationship \(G_0 = \rho(V_s)^2\), where \(V_s\) is the shear-wave velocity and \(\rho\) is the measured sample bulk density. The samples were created by dry pluviation and vibrated gently to obtain a relative density \((D_r)\) of 70%. They were then consolidated anisotropically to the stress levels provided in Table 2.2, with a ratio of horizontal to vertical effective stress \((K)\) of 0.5, which was considered representative of the \(K_0\) value in a normally consolidated sand. The samples were sheared to failure under axial compression after completion of the bender element tests. Local strain instrumentation (in the form of submersible LVDTs) was mounted on samples to allow accurate resolution of the degradation of stiffness with axial strain during the shearing stage (see Appendix B).

### Table 2.2 Triaxial test results for \(D_r = 0.7\)

<table>
<thead>
<tr>
<th>Test</th>
<th>(\sigma'_3)</th>
<th>(\sigma'_1)</th>
<th>(q_f)</th>
<th>(p'_f)</th>
<th>(G_o)</th>
<th>(E_0)</th>
<th>(E_{50})</th>
<th>(\phi'_p)</th>
<th>(\psi'_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>MPa</td>
<td>MPa</td>
<td>MPa</td>
<td>°</td>
<td>°</td>
</tr>
<tr>
<td>1</td>
<td>40</td>
<td>80</td>
<td>148</td>
<td>92</td>
<td>124</td>
<td>272</td>
<td>24.9</td>
<td>39.4</td>
<td>11.3</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>200</td>
<td>313</td>
<td>205</td>
<td>172</td>
<td>378</td>
<td>70.7</td>
<td>37.5</td>
<td>9.4</td>
</tr>
<tr>
<td>3</td>
<td>300</td>
<td>600</td>
<td>988</td>
<td>627</td>
<td>339</td>
<td>746</td>
<td>203</td>
<td>38.6</td>
<td>10.3</td>
</tr>
</tbody>
</table>

2.4 Derivation of HSS parameters

The very small strain shear moduli \((G_0)\) recorded in bender element tests conducted while samples underwent anisotropic consolidation with \(K=0.5\) are presented in Figure 2.1 as a function of the sample radial effective stress (noting all samples had a relative density of 0.7). The best fit line to these data match the general form of Equation (2.4), where
Derivation of hardening soil model properties from triaxial tests

\[ G_0^{\text{ref}} = 185 \text{MPa} \ (\text{at } p^{\text{ref}} = 100 \text{kPa}) \] with \( m = 0.5 \) which is in line with expectation (e.g. Hardin & Black, 1969; Zhou & Chen, 2005). Figure 2.1 also shows the inaccuracies involved if an assumption of \( m = 1 \) is made (i.e. \( G_0 \) linearly varying stress level).

\[ \gamma_{0.7} = 4 \times 10^{-5} \left( \frac{\sigma'_3}{p^{\text{ref}}} \right)^{0.5} \] \hspace{1cm} (2.5)

It is important to note that the HSS model assumes that \( \gamma_{0.7} \) is a material constant and does not change with stress level.
Figure 2.2 Normalised secant shear modulus variation with shear strain

\[
\frac{G_{\text{sec}}}{G_0} = \frac{\gamma_{0.7}}{\sigma_3' / p_{\text{ref}}}^{0.5}
\]

Figure 2.3 Shear strain ($\gamma_{0.7}$) vs. minor principal effective stress (normalised by reference stress)
The $E_{50}$ values could not be estimated directly from the measured test data as the test samples were consolidated anisotropically (and simple parameter derivation for the HSS model requires a drained test on an isotropically consolidated sample). This limitation was overcome by employing a trial and error procedure using various $E_{50}$ values applied to the formulation to produce a best fit to the measured deviator stress-axial strain curves. An example of the fit achieved for Test 1 is provided in Figure 2.5.

It was found that, for the three tests examined here (see Table 2.2), values of $E_{50}$ varied approximately linearly with the initial stress level. This is illustrated in Figure 2.4 which plots the variation of $E_{50}$ with the samples’ initial radial effective stress ($\sigma'_{3i}$). The best fit value of ‘$m$’ in Equation (2.1) is evidently 1.0. However, this value differs from the best fit ‘$m$’ value of 0.5 for the very small strain stiffness ($G_0$) and it is therefore not possible to satisfy Equations (2.1) and (2.4) simultaneously with a single value of $m$.

![Figure 2.4 Variation of the $E_{50}$ soil stiffness modulus with confining stress ($\sigma'_{3}$)](image-url)
The triaxial data indicated that the very small strain stiffness \( G_0 \) varies with the effective stress level raised to the power of 0.5 (i.e. \( m=0.5 \)) while the larger strain \( E_{50} \) values increase approximately linearly with the effective stress level (i.e. \( m=1.0 \)). These different exponents cannot be incorporated into the HSS model as implemented in the Plaxis FE program and therefore, for examination of the footing loading experiment discussed in the following, two sets of best fit parameters are employed, namely parameters derived with the assumption of \( m=0.5 \) (case M1) and of \( m=1.0 \) (case M2). Although \( G_0^{\text{ref}} \) and \( E_{50}^{\text{ref}} \) values of 185MPa and 70MPa were derived for all samples tested (each at \( D_r=0.7 \)), it can be seen in Table 2.1 that a lower \( E_{50}^{\text{ref}} \) is specified for the M1 case and a higher \( G_0^{\text{ref}} \) is specified for the M2 case so that the calculated stiffness values at \( \sigma'_3=50\text{kPa} \) are approximately the same (see Figure 2.1 and Figure 2.4). This stress level was chosen as it represents the approximate initial horizontal stress in the footing test described in the next section. 

**Figure 2.5** Deviator stress vs axial strain calculation for Test 1 using the HSS model vs. measured data 

### 2.5 HSS model calculations

The triaxial data indicated that the very small strain stiffness \( G_0 \) varies with the effective stress level raised to the power of 0.5 (i.e. \( m=0.5 \)) while the larger strain \( E_{50} \) values increase approximately linearly with the effective stress level (i.e. \( m=1.0 \)). These different exponents cannot be incorporated into the HSS model as implemented in the Plaxis FE program and therefore, for examination of the footing loading experiment discussed in the following, two sets of best fit parameters are employed, namely parameters derived with the assumption of \( m=0.5 \) (case M1) and of \( m=1.0 \) (case M2). Although \( G_0^{\text{ref}} \) and \( E_{50}^{\text{ref}} \) values of 185MPa and 70MPa were derived for all samples tested (each at \( D_r=0.7 \)), it can be seen in Table 2.1 that a lower \( E_{50}^{\text{ref}} \) is specified for the M1 case and a higher \( G_0^{\text{ref}} \) is specified for the M2 case so that the calculated stiffness values at \( \sigma'_3=50\text{kPa} \) are approximately the same (see Figure 2.1 and Figure 2.4). This stress level was chosen as it represents the approximate initial horizontal stress in the footing test described in the next section.
The HSS predictions for Test 1 using the parameters given in Table 2.1 are presented in Figure 2.5. It should be noted that both sets of parameters (i.e. M1 and M2) gave the same predictions for this test as the stiffness parameters (e.g. $G_0$ and $E_{50}$) for both were the same at the consolidation stress employed in this test ($\sigma'_3=40\text{kPa}$). The comparison with measured $q$ vs $\varepsilon_a$ data on Figure 2.5 looks reasonable, although the level of dilation is lower than measured (see Figure 2.6). A better match to the level of dilation for this test would be achieved using a slightly higher angle of dilation ($\psi$) than the value of $10^\circ$ employed in the calculations which was the average measured in the three tests (see Table 2.2). Dilation ceased at an axial strain of about 15% as a dilation cut-off with a maximum void of 0.75 was specified.

Figure 2.6 Volumetric strain vs axial strain calculation for Test 1 using the HSS model vs. measured data
2.6 Application of HSS model

The ability of the HSS model to predict the load settlement response of a deep footing on sand in a testing chamber is now examined. The sand in the chamber was placed in an identical manner to that used for triaxial sample preparation (full experimental procedures for the footing tests are presented in Appendix C). The chamber sample was consolidated under one dimensional conditions (with $K_0\sim 0.5$) to a vertical effective stress of 100kPa i.e. equal to the reference stress adopted in the derivation of the HSS parameters with an initial horizontal effective stress of about 50kPa. The chamber diameter was about 15 times the footing diameter employed of 25mm, as shown in Figure 2.7. The sand relative density was the same as that used in the triaxial tests ($D_r=0.7$). The Plaxis FE code (Brinkgreve et al., 2014), was employed for the calculations as the HSS model has been implemented in Plaxis. The calculations adopted the parameters provided in Table 2.1.

2.7 Predicted and measured footing response

Figure 2.8 plots the calculated response of the 25mm diameter deep footing using Plaxis and the HSS M1 and M2 parameter sets (see Table 2.1). It is evident that both parameter sets give similar bearing stress ($\sigma$) versus settlement curves and these are on average a little stiffer than the measured response (note that Figure 2.8 shows the settlement, $s$, normalised by the plate diameter, $D$). The calculations give reasonable predictions in the area of general serviceability interest (i.e. $s/D < 2\%$). A lack of the fit between the calculations and measurements is not very surprising given that the difference between the M1 and M2 parameters and the employment of a $\gamma_{0.7}$ strain that is not stress level dependent. The lower settlements at a given bearing stress given by the M2 parameters (with $m=1$) may be anticipated following inspection of Figure 2.5 which shows that operational $E_{50}$ values become higher for this case as the load level increases about the in-situ stress levels.

It should be noted that the displacement induced by the initial stress applied to the footing due to the weight of the loading frame was not measured (see Appendix D). The applied stress at $s/D=0$ on Figure 2.8 of 240kPa represents this frame weight. Another feature of the experimental set-up worthy of mention is that the initial stress distribution beneath the test footing was constant and therefore the test conditions were more analogous to the end bearing of a pile than to a shallow footing. For the former case, calculations using the
HSS model are likely to be less sensitive to the various stress level dependencies of the HSS stiffness parameters.

Figure 2.7 FE model of the experimental footing test
2.8 Conclusions

The Hardening Soil Small (HSS) constitutive model is particularly popular among practitioners, partly because it is implemented in the Plaxis FE code. A comprehensive series of triaxial tests involving bender elements and local strain instrumentation demonstrated that the stress level dependence of very small and larger strain stiffness is different and that this difference cannot be accommodated in the current HSS model. Contrary to the experimental observations, the HSS model also assumes that rate of normalised stiffness degradation with shear strain is independent of the stress level. These limitations contribute to a slightly stiffer calculated response for a vertical load test on a deep footing on the same reconstituted sand.
Chapter 3  Stress-strain response of fine silica sand using a miniature pressuremeter

Abstract

The pressuremeter is a well-known geotechnical test, used to measure soil strength and stiffness. In this paper, a miniature pressuremeter device, developed at the University of Western Australia (UWA), was employed to measure the stress-strain behaviour of dense fine silica sand at a range of stress levels. The UWA miniature pressuremeter has a diameter to length ratio of unity, and its inflation after burial in a normally consolidated sand represents a well-defined boundary value problem. Back-analysis was performed using the Finite Element method and the well-known Hardening Soil-Small (HSS) model. The HSS model was found to provide a reasonable match to the measured stress-strain response using parameters derived from triaxial compression tests.

3.1 Introduction

Fine ‘UWA sand’ is a manufactured, uniformly graded silica sand used in geotechnical centrifuge testing at UWA e.g. O’Loughlin and Lehane (2003), Xu and Lehane (2008) and Lee et al. (2013). Despite its widespread use, there have been few studies that examine its mechanical properties, with O’Loughlin and Lehane (2003) one of the few studies involving triaxial testing of the sand. More recently, Bagbag et al. (2016b) (Chapter 2) presented results from three anisotropically consolidated drained triaxial compression tests on UWA sand reconstituted to a relative density \( D_r \) of 70%.

The pressuremeter test is well adapted the soil strength measurement and compressibility characteristics of sands, gravels and rock in-situ (Gibson & Anderson, 1961). The results of the pressuremeter device can be used directly to predict foundations settlement without any other supplementary laboratory testing (Ménard, 1957).

This paper presents a series of laboratory scale pressuremeter tests on dense UWA sand at a range of confining stresses. The tests were conducted using the UWA miniature pressuremeter described by Johnston et al. (2013). This device is significantly different to any previously built miniature pressuremeter as it uses air as the pressurising fluid and the membrane displacement is measured using strain gauged ‘feeler-arm’ transducers,
rather than inferring displacements from measured volume changes in the pressuring fluid. This direct method of measuring displacement is more accurate than using changes in the fluid volume (Johnston et al., 2013).

The UWA miniature pressuremeter has a diameter to length ratio of unity, and its inflation after burial in a normally consolidated sand represents a well-defined boundary value problem. Therefore, tests were interpreted using available elastic and plasticity solutions based on spherical and cylindrical cavity expansion theory, respectively. The interpreted friction angles, dilation angles and unload reload stiffnesses are presented and compared with data from other tests involving the same sand (Lehane et al., 2005; Xu & Lehane, 2008; Lee et al., 2013; Bagbag et al., 2016b).

The well-defined nature of the boundary value problem provides an opportunity to interpret the pressuremeter tests using the Finite Element (FE) method and sophisticated soil constitutive models. In this paper, the test results are compared with pressuremeter responses calculated using the FE method and the Hardening Soil Small (HSS) model; Plaxis 2D (version 2015.2) was used for the computations (Brinkgreve et al., 2014). The parameters employed for the HSS model were derived in a separate study reported by Bagbag et al. (2016b), as presented in Chapter 2. The aim of this paper is to assess the validity for application to pressuremeter loading of the HSS parameters derived from the triaxial testing.

### 3.2 Laboratory set-up

#### 3.2.1 Classification data

‘UWA sand’ has a minimum and maximum void ratio of 0.45 and 0.75 respectively (see Appendix A). The material specific gravity is 2.67 and is classified as a uniform sub-rounded to sub-angular fine sand. The particle size distribution is shown in Figure 3.1.

#### 3.2.2 Preparing and setting-up pressuremeter tests

A 20 mm diameter laboratory scale pressuremeter developed at UWA by Johnston et al. (2013) was used (see Figure 3.2a). Unlike previously developed laboratory pressuremeters, the UWA device uses air as the pressurising fluid and the membrane
displacement (0.3mm thick latex) is measured using strain-gauged “feeler-arm” transducers. The pressuremeter was built into an aluminium rod and was located 180mm above the base of this rod. The rod was positioned at the centre of a 393mm internal diameter, 400mm high steel chamber (see Figure 3.2b, 3.2c). The top of aluminium rod was clamped to the top of the chamber to prevent it moving during sample preparation and testing (see Figure 3.2b). Sand was rained into the chamber once the pressuremeter was fixed in place (see Figure 3.3). The soil density was controlled using an automatic hopper with specific slot widths and heights. Dense sand was achieved by using a slot width of 1.5mm and the height measured from the sand surface to the opening slot was held constant at 1m. This produced sand with 70% relative density (void ratio, \( e=0.54 \)). The sample mass and volume was measured for each sample. The soil filled the chamber with an allowance for a 40mm thick top plate through which vertical stress was applied to the sand.

Figure 3.1 Partial Size Distribution (PSD)
Figure 3.2 (a) Pressuremeter membrane, (b) pressuremeter in position prior to pouring sand (c) schematic diagram of the experimental set-up

Figure 3.3 The automatic hopper to rain the sand into pressuremeter chamber test
A vertical load was applied to the top plate using a hydraulic jack, as shown in Figure 3.4. This load was applied for a minimum period of 48 hours prior to pressuremeter testing, to allow creep rates to reduce to negligible values (see Lim & Lehane, 2014). In this paper, tests are presented for vertical applied pressures of 50kPa, 60kPa and 100kPa.

The pressuremeter device uses a digitally controlled air compressor to expand the membrane. The feeding rate of the pressure was held constant at 50kPa/min.

![Typical Pressuremeter Test Setup](image)

Figure 3.4 A typical pressuremeter test setup using a hydraulic system to apply the surcharge load

### 3.3 Results and discussion

#### 3.3.1 Experimental results

The measured cavity pressure versus corresponding cavity strain for applied vertical stresses of 50kPa, 60kPa and 100kPa are presented in Figure 3.5a.
Friction ($\phi'$) and dilation ($\psi$) angles were estimated using the method of Hughes et al. (1977) as presented in Table 3.1. This method is based on the slope ($s$) of a plot of the logarithm of the cavity pressure versus the logarithm of cavity strain (see Figure 3.5b). An initial estimate of the constant volume friction angle ($\phi'_{cv}$) is required. Hughes et al. (1977) proposed Equation (3.1) for friction angle and Equation (3.2) for peak dilation angle. These equations are for cylindrical cavity expansion, which is representative of standard in-situ pressuremeter. The friction and dilation angles inferred using these equations for the three stress levels are presented in Table 3.1, assuming a $\phi'_{cv}$=34º as recommended for the ‘UWA sand’ by O’Loughlin and Lehane (2010).

$$\sin \phi' = \frac{s}{1+(s-1) \sin \phi'_{cv}} \quad (3.1)$$
$$\sin \psi = s+(s-1) \sin \phi'_{cv} \quad (3.2)$$

Table 3.1 Friction and dilation angles inferred using the Hughes et al. (1977) method for cylindrical expansion.

<table>
<thead>
<tr>
<th>$\sigma'_v$ (kPa)</th>
<th>$s$</th>
<th>$\phi'$ (degs)</th>
<th>$\psi$ (degs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.56</td>
<td>47.9</td>
<td>18.2</td>
</tr>
<tr>
<td>60</td>
<td>0.51</td>
<td>44.3</td>
<td>13.3</td>
</tr>
<tr>
<td>100</td>
<td>0.58</td>
<td>49.1</td>
<td>19.9</td>
</tr>
</tbody>
</table>

Mair and Wood (1987) mentioned that the Hughes et al. (1977) method should be considered approximate, as the friction and dilation angles derived are highly dependent on the estimated value of $s$. The method also makes the assumption of cylindrical cavity expansion, which is clearly not well suited to the UWA pressuremeter device (given the length/diameter ratio of unity). By comparison with the angles listed in Table 3.1, the triaxial tests reported in Chapter 2 gave an average peak friction angle ($\phi'$) value of 38.5º and peak dilation angle ($\psi$) of 10º at the same initial stress level and same relative density of 0.7. Xu and Lehane (2008) deduced peak friction and dilation angles of 42º and 12º for a somewhat denser UWA sand sample at an in-situ stress level of 120kPa.
Stress-strain response of fine silica sand using a miniature pressuremeter

\[ G_{ur} = \frac{\Delta p_c}{2k\Delta \varepsilon_c} \]

Figure 3.5 Measured variation of cavity pressure with cavity strain (a) linear scale and (b) logarithmic axes
Fahey (1991) proposed a method to measure the elastic shear modulus by performing small unload-reload loops during plastic monotonic loading in pressuremeter tests. These loops exhibit a quasi-linear behaviour and it is suggested that the loop gradient may be used to determine the unload-reload shear modulus from the elastic cavity expansion solutions proposed by Yu (2000). For an assumed elastic response of the soil, the unload-reload shear moduli \( G_{ur} \) can be determined as:

\[
G_{ur} = \frac{\Delta p_c}{2k\Delta \varepsilon_c}
\]  

(3.3)

where \( \Delta p_c \) is the difference between the cavity pressures at the start and end of the unload-reload loop and \( \Delta \varepsilon_c \) is the corresponding change in cavity strain; the constant \( k \) is 2 for spherical expansion and unity for cylindrical cavity expansion. As shown in Figure 3.5a, one test, with an applied vertical stress of 60kPa, is included an unload-reload loop. Assuming \( k = 2 \) (i.e. spherical cavity expansion), the unload-reload shear modulus \( (G_{ur}) \) was found to be 25MPa and 25.8MPa from the first and second loops, respectively.

It is evident from Figure 3.5 that the lift-off pressure is not easy to distinguish at the start of the tests. However, it is apparent from the final unloading stages that there is a cavity pressure at which the cavity strain reduces rapidly while the cavity pressure remains constant i.e. the unload curve is approximately horizontal. This pressure is likely to be related strongly to the in-situ horizontal stress. The cavity stresses at which this transition occurs can be seen in Figure 3.5b to be at 20kPa, 50kPa and 25kPa for tests with applied vertical stress of 50kPa, 60kPa and 100kPa respectively, i.e. on average the cavity stresses are approximately half of the applied vertical stress, which is consistent with the expectation that \( K_0 \) is approximately 0.5 for the normally consolidated stress history of the sand (see Figure 3.5).

### 3.3.2 Back analysis

The Plaxis 2D (version 2015.2) Finite Element program, developed by Brinkgreve \textit{et al.} (2014), was employed along with the small strain hardening soil model (HSS), introduced by Benz (2007), to simulate the pressuremeter tests. The HSS model soil parameters used are presented in Table 3.2. These were derived in Chapter 2 from consolidated drained triaxial tests on ‘UWA sand’ reconstituted to the same relative density of 0.7 used for the
Stress-strain response of fine silica sand using a miniature pressuremeter pressuremeter experiments. One of the difficulties encountered by Bagbag et al. (2016b) was that the secant stiffness at 50% of the peak deviator stress \(E_{50}\) was found to vary with the stress level raised to the power \(m\) of 1.0, whereas the very small strain shear stiffness \(G_0\) was better represented using ‘\(m\)’ an exponent of 0.5. As the HSS model only allows input of a single exponent, the \(m\) value was taken equal to 0.5 for the purposes of this paper and the reference stiffness values inputted were those estimated at the average of the three initial lateral stresses in the tests (25kPa, 30kPa and 50kPa). Chapter 2 also point out that the shear strain at 70% of the maximum shear modulus \(\gamma_{0.7}\) is stress dependent. As the model does not allow for this stress dependency, the value of \(\gamma_{0.7}\) assumed was also taken to be that corresponding to the average of the three initial lateral stresses in the tests.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>HSS</th>
</tr>
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<tbody>
<tr>
<td>(m)</td>
<td>Power for stress-level dependency of stiffness</td>
<td>0.5</td>
</tr>
<tr>
<td>(E_{50}^{\text{ref}})</td>
<td>Reference secant stiffness modulus at 50% of the failure load corresponding to the reference stress, (p_{\text{ref}})</td>
<td>35MPa</td>
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<tr>
<td>(E_{\text{oed}}^{\text{ref}})</td>
<td>Reference tangent stiffness for oedometer loading modulus corresponding to (p_{\text{ref}})</td>
<td>35MPa</td>
</tr>
<tr>
<td>(E_{\text{ur}}^{\text{ref}})</td>
<td>Reference unload-reload stiffness modulus corresponding to (p_{\text{ref}})</td>
<td>105MPa</td>
</tr>
<tr>
<td>(c')</td>
<td>Cohesion</td>
<td>0</td>
</tr>
<tr>
<td>(\phi)</td>
<td>Friction angle</td>
<td>38.5°</td>
</tr>
<tr>
<td>(\psi)</td>
<td>Dilation angle</td>
<td>10°</td>
</tr>
<tr>
<td>(K_0^{\text{nc}})</td>
<td>(K_0)-value for normal consolidation</td>
<td>0.5</td>
</tr>
<tr>
<td>(p_{\text{ref}})</td>
<td>Reference pressure at which quoted stiffness values apply (taken as atmospheric pressure)</td>
<td>100kPa</td>
</tr>
<tr>
<td>(R_t)</td>
<td>Failure ratio of deviatoric stress at failure over deviatoric stress at Asymptote</td>
<td>0.9</td>
</tr>
<tr>
<td>(v_{\text{ur}})</td>
<td>Poisson’s ratio for unloading-reloading</td>
<td>0.2</td>
</tr>
<tr>
<td>(\gamma_{0.7})</td>
<td>Shear-strain at 70% of small-shear modulus, (G_0)</td>
<td>(2 \times 10^{-5})</td>
</tr>
<tr>
<td>(G_0^{\text{ref}})</td>
<td>Reference small-strain shear modulus</td>
<td>185MPa</td>
</tr>
</tbody>
</table>

An axisymmetric model using 687 triangular 15-noded elements was used. The mesh was refined around the membrane as shown in Figure 3.6. The chamber was modelled assuming a smooth inner surface. The boundary to the right (see Figure 3.6) was allowed to move vertically, but was fixed radially. The boundary at the base was free radially and fixed vertically. The model includes three major steps: (i) the surcharge pressure was applied in the initial step to establish the initial stresses in the soil with \(K_0\) specified as
0.5, (ii) the pressure was applied to the vertical faces of the pressuremeter to simulate its expansion and (iii) unload-reload loops were applied, as in the physical experiments.
The Plaxis 2D HSS model simulation is presented in Figure 3.7. The model evidently provides a very good simulation of the measured pressuremeter response at small and medium cavity strains. It may therefore be inferred that the parameters determined from triaxial tests provide a reliable simulation of the pressuremeter tests, especially over the first 2%, which is usually most critical in geotechnical design.

Figure 3.7 Hardening Soil-Small model for pressuremeter tests

Figure 3.8 explores the sensitivity to friction and dilation angles of the HSS model predictions. All parameters in two sets of analyses (HSS-TX and Hughes et al. (1977) method) were the same except for the friction and dilation angles. The friction and dilation angles for HSS-TX model are presented in Table 3.2, whereas the angles for Hughes et al. (1977) method are those presented in Table 3.1 at $\sigma'_v=50$kPa. It is evident that the triaxial angles provide a better fit to the measurements, which is likely to be due to differences between the near spherical expansion mode in the experiments whereas the plane strain cylindrical expansion mode assumed in the Hughes et al. (1977) formulation. It should be noticed that the HSS model used the actual boundary conditions and it is
consistent with the triaxial data rather than assuming plane strain cylindrical cavity expansion, as adopted by Hughes et al. (1977) method.

Figure 3.8 Comparison of measured pressuremeter response at $\sigma' = 50$ kPa with HSS predictions using triaxial (TX) friction angles and friction angles calculated using Hughes et al. (1977)

3.4 Conclusion

This paper presents results from three laboratory-scale pressuremeter tests performed in reconstituted dense sand. It is shown that, despite its limitations, the Hardening Soil Small model provides a good match to the measured response, when parameters derived from triaxial compression tests are used in the simulations. Further studies are required to understand the differences between the operational friction angles observed in the experiments (where the mode of deformations was approximately spherical) and those backanalysed using existing solutions for pressuremeter tests.
Chapter 4  Predictions of footing and pressuremeter response in sand using a hardening soil model

Abstract

The paper presents the results and interpretation of triaxial compression tests for a uniformly graded fine to medium siliceous sand. These results are used together with data obtained from boundary value problems to provide an objective assessment of the predictive capabilities of a widely accessible and popular constitutive model, referred to as the HSS or Hardening soil model (Schanz et al., 1999) with a small strain overlay (Benz, 2007). The triaxial compression tests, which involved a range of relative densities and stress levels, are used to derive model parameters providing the optimal fit to the triaxial experiments. Measurements obtained in pressuremeter and footing experiments performed in a laboratory testing chamber with the same reconstituted sand are then compared with finite element analyses of these experiments using the selected model parameters. These comparisons combined with a review of simulations for the triaxial tests are used to draw conclusions related to the application of the HSS model in sand.

4.1 Introduction

The finite element (FE) method is an established technique for predicting ground deformations and serviceability limit state design. Simpson et al. (1979), Jardine et al. (1986), Burland (1990), and others, have clearly illustrated that the calculation of realistic deformation patterns in FE analyses requires modelling of the elasto-plastic, non-linear nature of soil. Potts (2003), however, describes how the geotechnical profession still has significant difficulties with the application of this approach which, withstanding differences in FE codes and algorithms, arise primarily because of limitations of constitutive soil models and difficulties in measuring and selecting appropriate parameters. There are numerous examples in the literature showing how a given soil model with a particular set of parameters can provide predictions that are close to field observations (e.g. see Al-Defae et al., 2013; Lehane et al., 2008). Such examples rarely present predictions in advance of the field observations being available (i.e. ‘Class A’ predictions), leaving doubts about the general applicability of the model and the associated parameter selection technique. There are far fewer published ‘Class A’
examples and these are usually accompanied with better performing ‘Class C’ predictions that involve some manipulation of model parameters to match the field observations.

The focus of this chapter is therefore to perform an objective assessment of the predictive capabilities of a popular constitutive model, referred to as the HSS or Hardening soil model (Schanz et al., 1999) with a small strain overlay (Benz, 2007). This model, which is most often applied to sands and stiff clays, is arguably the most popular non-linear elasto-plastic model used by geotechnical practitioners, primarily because of its implementation in the widely used PLAXIS FE code (Brinkgreve et al., 2017). An experimental programme was designed to test the ability of the HSS model to extrapolate from triaxial stress space, used by the HSS model for its parameter derivation, to the more complex boundary value problems in pressure chamber tests under conditions where uncertainties relating to differences between the state and fabric of the sand in chamber experiments and those in triaxial samples did not exist.

A summary of relevant features of the HSS model is presented. The HSS model parameters are derived from ten consolidated drained triaxial tests (with bender elements) on reconstituted samples of the University of Western Australia fine silica sand (UWA-FSS), performed at a range of densities and stress levels. Although this material has been used for many years in centrifuge experiments at the University of Western Australia (UWA), relatively few element tests are reported in the literature. Therefore, an important aim of the chapter is to firstly present an interpretation of the properties of this sand, which is independent of the model used to simulate its response. The parameter derivation for the HSS model using the triaxial tests is then described and attention is drawn to particular issues related to shortcomings of this constitutive model and associated parameter selection.

The laboratory pressure chamber experiments involved pressuremeter loading and vertical loading of footings in normally consolidated ‘UWA sand’. The testing arrangement and procedures employed in these experiments are described and measured data are compared with FE simulations of the experiments performed with the HSS model and the derived parameters. These comparisons, combined with difficulties encountered with parameter selection, are used to draw conclusions relating to strengths and weaknesses of the HSS model when applied to typical boundary volume problems in normally consolidated, reconstituted sand.
4.2 Hardening soil model

The HSS model is an extension of the original Hardening Soil model (HS model), developed by Schanz et al. (1999). The extension uses a ‘Small Strain Overlay’ (Benz, 2007) to allow better representation of the soil stiffness at low strains. Important features of the model are detailed below and a full description of the implementation of HSS in the Plaxis FE program is provided by (Brinkgreve et al., 2017).

4.2.1 Small strain parameters

The small strain component is defined by the specification of a ‘reference’ small-strain shear modulus ($G^\text{ref}_0$) and the parameter $\gamma_{0.7}$, which is the shear strain required to reduce the shear modulus from $G_0$ to 70% of $G_0$. For a cohesionless material, the operational small strain stiffness, ($G_0$), is described as a function of the minor principal effective stress, ($\sigma_3'$), and is given by Equation (4.1).

$$G_0 = G^\text{ref}_0 \left( \frac{\sigma_3'}{p^\text{ref}} \right)^m$$  \hspace{1cm} (4.1)

where $p^\text{ref}$ is the reference atmospheric pressure (by default 100kPa); and $m$ is a power law constant. The small strain stiffness decays with increasing strain level using the following relationship proposed by Santos and Correia (2001):

$$\frac{G_{\text{sec}}}{G_0} = \frac{1}{1 + \frac{3}{7} \frac{\gamma}{\gamma_{0.7}}}$$  \hspace{1cm} (4.2)

Outside of the small strain region, a hyperbolic relationship is used to describe a response in which the shear stiffness reduces as the material is sheared toward failure. This relationship is defined for isotropically consolidated drained triaxial shearing as expressed in Equation (4.3).

$$\varepsilon_a = \frac{2 - R_f}{2E_50} \frac{q}{1 - q/q_a}$$  \hspace{1cm} (4.3)
where $q$ is the deviatoric stress, $q_a$ is the asymptotic value of shear stress (defined by the strength of the material), $\varepsilon_a$ is the axial strain, $R_f$ is the failure ratio (with a default value of 0.9) and $E_{50}$ is a parameter that defines the slope of a line connecting the origin ($\varepsilon_a=0$ and $q=0$) to a point on the curve at $q = R_f q_a/2$. For cohesionless material, the operational value of $E_{50}$, as it appears in Equation (4.3), varies with the minor principal effective stress, $\sigma_3'$, as shown in Equation (4.4).

$$E_{50} = E_{50}^\text{ref} \left( \frac{\sigma_3'}{p^\text{ref}} \right)^m$$ \hspace{1cm} (4.4)

where $E_{50}^\text{ref}$ is the reference input parameter ($p^\text{ref}$ and $m$ are described above).

For primary one-dimensional compression, the HSS model assumes a non-linear relationship between major principal stress ($\sigma_1'$) and strain ($\varepsilon_a$). The tangent to this non-linear relationship ($E_{oed}$) is given as shown in Equation (4.5).

$$E_{oed} = E_{oed}^\text{ref} \left( \frac{\sigma_1'/K_{0\text{NC}}}{p^\text{ref}} \right)^m$$ \hspace{1cm} (4.5)

where $p^\text{ref}$ is a reference pressure and $K_{0\text{NC}}$ is the ratio of vertical to horizontal effective stress in one-dimensional compression. Elastic unload-reload behaviour is defined by a Young’s modulus $E_{ur}$ and Poisson’s ratio, $\nu_{ur}$. The elastic stiffness varies with stress level in the same way as $E_{50}$ that is:

$$E_{ur} = E_{ur}^\text{ref} \left( \frac{\sigma_3'}{p^\text{ref}} \right)^m$$ \hspace{1cm} (4.6)

Equations (4.1), (4.4), (4.5) and (4.6) indicate the same stress level dependence of the four quite different aspects of soil response. It should also be noted that the acceptable range of values for $E_{oed}^\text{ref}/E_{50}^\text{ref}$ and $E_{ur}^\text{ref}/E_{50}^\text{ref}$ ratios that can be accommodated by the HSS model is relatively small; respective values of unity and three are typically adopted, based on research such as that of Khoiri and Ou (2013).
4.3 Yielding and strength parameters

The HSS model employs the Mohr-Coulomb failure criterion, but does not model post-peak softening. The mobilised friction angle at which constant volume shearing takes place \( (\phi'_{cv}) \) is calculated from the assigned peak friction angle \( (\phi') \) and the dilation angle \( (\psi) \) using the Equation (4.7).

\[
\sin\phi'_{cv} = \frac{\sin\phi' - \sin\psi}{1 - \sin\phi'\sin\psi}
\]  

(4.7)

The sand contracts when the mobilised friction angle \( \phi'_m \) is below \( \phi'_{cv} \) and dilates when \( \phi'_m \) exceeds \( \phi'_{cv} \) up to the specified maximum void ratio \( (e_{max}) \) is attained. The ratio of the plastic volumetric strain rate to the plastic shear strain rate is assumed equal to \( \sin \psi_m \), where \( \psi_m \) is the mobilised dilation angle obtained from Equation (4.7) at higher mobilised strengths by setting \( \phi' = \phi'_m \) and \( \psi = \psi_m \); the plastic volumetric strain rate is taken as zero when \( \sin \phi'_m < 0.75 \sin \phi' \). A cap yield surface enables plastic volumetric strain hardening under compression. The specified value of \( E_{oed}^{ref} \) controls the magnitude of these plastic strains and the size of the elastic region bounded by the cap and shear hardening yield surfaces depend on the nominated \( K_0 \) value for normal consolidation \( (K_0^{NC}) \).

4.4 Triaxial tests on UWA sand

Ten anisotropically consolidated drained compression tests were performed on samples of ‘UWA sand’. This siliceous sand has a mean effective particle size \( (D_{50}) \) of 0.29mm, a uniformity coefficient of 2.1 and maximum and minimum void ratios \( (e_{max} \text{ and } e_{min}) \) of 0.75 and 0.45 respectively (see Appendix A). The (normally consolidated) triaxial samples were created by air pluviation and then vibrated to achieve the desired relative densities. Full saturation of the samples was achieved under a backpressure of 500kPa with the triaxial B-value in excess of 0.95 for all cases. Bender elements were located at both ends of the samples to allow measurement of shear wave velocity and hence inference of the small strain shear stiffness \( (G_0) \). Displacement transducers mounted on pedestals were pinned to the membranes to measure axial strains \( (\varepsilon_a) \) locally on the samples. Radial strains were not measured directly (due to equipment limitations) and were therefore inferred from the axial strain measurements and the volumetric strains that were recorded using a GDS advanced pressure/volume controller. Details of all ten tests
are summarised in Table 4.1. The test series comprised a range of different sand relative densities ($D_r$) and consolidation stresses. The ratio of the radial stress ($\sigma'_r$) to axial stress ($\sigma'_a$) imposed for consolidation was fixed at 0.5; this ratio was selected for consistency with the footing and pressuremeter experiments in normally consolidated sand, described later.

The variations of the deviator stress ($q$) and volumetric strain ($\varepsilon_v$) with axial strain ($\varepsilon_a$) recorded in all of the triaxial tests are plotted on Figure 4.1. The trends observed are consistent with typical characteristics exhibited by clean sands tested under triaxial conditions. It is seen that (i) peak deviator stresses increase with the relative density and stress level, (ii) samples dilate under shear with greater dilation exhibited at lower stress levels and in denser sands and (iii) deviator stresses develop peak values at axial strains of about 5% and reduce post-peak to reach constant ultimate values at an axial strain of 20% when volumetric strain changes effectively cease.

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Consolidation stresses (kPa)</th>
<th>Relative density ($D_r$)</th>
<th>Void ratio ($e$)</th>
<th>$G_0$ (measured) (MPa)</th>
<th>Best fit (MPa)</th>
<th>$G_0 = G_0_C$</th>
<th>$E_{50}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20 40</td>
<td>36</td>
<td>0.64</td>
<td>66</td>
<td>75 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>20 40</td>
<td>69</td>
<td>0.55</td>
<td>84</td>
<td>100 11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>40 80</td>
<td>42</td>
<td>0.62</td>
<td>90</td>
<td>105 9</td>
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</tr>
<tr>
<td>4</td>
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<td>120</td>
<td>160 25</td>
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<td>143</td>
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<td>300 600</td>
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<td>66</td>
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<tr>
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<td>96</td>
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<td>406</td>
<td>590 140</td>
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</tr>
</tbody>
</table>
Predictions of footing and pressuremeter response in sand using a hardening soil model

Figure 4.1 Deviator stress-axial strain-volumetric strain relationships measured in triaxial tests (a) at $\sigma_3 = 300\text{kPa}$ and different $D_r$ values, (b) for $D_r = 35-55\%$ at different stress levels, (c) for $D_r = 65-80\%$ at different stress levels
4.4.1 Very small-strain (elastic) stiffness

The very small-strain shear moduli ($G_0$) were calculated from shear wave velocities derived from the Bender element data. As is typical of sands, $G_0$ varied in approximate proportion to the square root of the stress level, e.g. Zhou and Chen (2005) obtained similar findings with silica sand. The density dependence of $G_0$ was captured using the void ratio function, $F(e)$, proposed by Hardin and Richart (1963), where $e$ is the initial void ratio.

$$F(e) = \frac{(2.17 - e)^2}{1 + e} \quad (4.8)$$

$G_0$ is presented in the normalised form in Figure 4.2, which also plots the following best-fit equation to the data using the format of Equation (4.1), where $p_a$ is taken as the reference stress (atmospheric pressure =100kPa).

$$\frac{G_0}{p_a} = 1000 F(e) \left(\frac{\sigma_3'}{p_a}\right)^{0.55} \quad (4.9)$$

Equation (4.9) only provides an approximate representation of the actual stress level dependence of the $G_0$ values. This is because these were measured using a vertically propagating shear wave and, under these conditions, $G_0$ depends more on the axial effective stress and is more correctly expressed as a function of all of the three principal stresses in a triaxial test (e.g. Kohata et al., 1997; Stokoe et al., 1995). The Equation (4.9) gives $G_0$ values that are in good agreement with the mean trend established by Lehane and Cosgrove (2000) for a wide variety of reconstituted sands tested under triaxial conditions.
4.4.2 Stiffness non-linearity

The variations of secant Young’s moduli ($E_{\text{sec}}$) with axial strain ($\varepsilon_a$) measured in all triaxial tests are presented in Figure 4.3. The value of the very small strain Young’s modulus ($E_0$), which was derived from $G_0$ using a typical small strain Poisson’s ratio for sand of 0.1 (Lehane & Cosgrove, 2000), is plotted on these figures at an axial strain of 0.001%. It is evident that the elastic region extends to an axial strain of, at most, about 0.002%. There is a dramatic reduction of $E_{\text{sec}}$ with increasing strain and $E_{\text{sec}}$ is typically 20 to 50 times less than $E_0$ at an axial strain of 1%. Figure 4.3a shows that effects of density (for triaxial tests at the same confining stress) become less significant as straining progresses whereas Figure 4.3b and 4.3c show the pronounced effect of confining stress at two separate densities. More detailed examination of these data show that the stress level dependence of $E_{\text{sec}}$ becomes greater at larger strains. For example, at an axial strain of 0.1% and fixed density, $E_{\text{sec}}$ typically varies with the initial confining stress ($\sigma'_3$) raised.
to the power of \(0.75 \pm 0.05\), which is a larger exponent than the value of 0.55 seen at very small strains; see Equation (4.9).

The value of \(\gamma_{0.7}\) in Equation (4.2) can be derived from the data plotted on Figure 4.3 by transforming the \(E_{sec} \ vs. \ \varepsilon_a\) data to \(G_{sec} \ vs. \ \gamma\) data. Rather than inferring radial strains from volumetric strain measurements (which can lead to misrepresentative values on the samples’ middle third), this transformation was made assuming standard elastic relationships with an intermediate strain Poisson’s ratio of 0.2 (Lehane & Cosgrove, 2000). Values of \(\gamma_{0.7}\) determined in this way are plotted on Figure 4.4, and seen to be broadly independent of the relative density but increase with the confining stress (\(\sigma'\)). For example, \(\gamma_{0.7}\) doubles from a value of about 0.004% at \(\sigma'_3 \approx 25\)kPa to 0.01% at \(\sigma'_3 \approx 300\)kPa. A similar stress level dependence for the strain at which the secant stiffness is 50% of the very small strain value has been shown by Lehane and Cosgrove (2000).

The best-fit line shown on Figure 4.4 is expressed in Equation (4.10) (with reference stress \(p_a=100\)kPa, as before).

\[
\gamma_{0.7} = 6.4 \times 10^{-5} \left(\frac{\sigma'_3}{p_a}\right)^{0.4}
\] (4.10)

It is noteworthy that, contrary to the trend indicated on Figure 4.4 \(\gamma_{0.7}\) is a constant in the HSS model and does not vary with stress level.
Figure 4.3 Secant Young’s modulus; (a) at $\sigma'_3=300\text{kPa}$ with different $D_r$ values, (b) for $D_r=35-55\%$ with different stress levels, (c) for $D_r=65-80\%$ with different stress levels; $E_{sec}$ values plotted at $\varepsilon_a=0.001\%$ are inferred from the bender element data.
4.4.3 Strength and dilation parameters

Given the large number of reported centrifuge tests using UWA sand over the last few decades, it is of considerable interest to compare the measured strength and dilation parameters with commonly used correlations. Bolton (1986) proposed the empirical relationships given in Equations (4.11) and (4.12) for the peak friction angle ($\phi'_p$) of a sand tested under triaxial compression conditions. Schanz and Vermeer (1996) extended Bolton’s work to derive the expressions given in Equations (4.13) and (4.14) for the peak dilation angle in terms of the maximum rate of change of the volumetric strain to the axial strain, $(d\epsilon_v/d\epsilon_a)_p$ and the $I_e$ index, as defined in Equation (4.12). It is assumed that the elastic components of the increments of the axial and volumetric strains are negligibly small when applying Equation (4.13).
Predictions of footing and pressuremeter response in sand using a hardening soil model

\[ \phi'_p = \phi'_{cv} + 3I_r \]  

\[ I_r = D_r \left[ 5.4 - \ln \left( \frac{p'_f}{p_a} \right) \right] - 1 \]  

\[ \psi_p = \sin^{-1} \left[ -\frac{\left( \frac{d \varepsilon_Y}{d \varepsilon_a} \right)_p}{2 - \left( \frac{d \varepsilon_Y}{d \varepsilon_a} \right)_p} \right] \]  

\[ \psi_p = \sin^{-1} \left[ \frac{I_r}{I_r + 6.7} \right] \]  

where \( p'_f \) is the mean effective stress at failure with a minimum value of 150kPa and \( p_a \) is the atmospheric pressure with a value of 100kPa.

It is seen in Figure 4.5a that Equation (4.11), which incorporates the observed dependence of peak friction on density and stress level, leads to predicted peak friction angles (\( \phi'_p \)), which are in good agreement with the measured angles in all tests. A constant volume friction angle (\( \phi'_{cv} \)) of 33° was employed in Equation (4.11), which is also the mean \( \phi'_{cv} \) value inferred directly from the test data. Estimates of \( \psi_p \) from the triaxial tests were obtained using Equation (4.13) and the axial and volumetric strain measurements. These are compared on Figure 4.5b with the empirical values of \( \psi_p \) determined using Equation (4.14), where they are seen to be in reasonable agreement with the triaxial strain data. The data show that the magnitude of \( (d \varepsilon_Y/d \varepsilon_a)_p \) is about one tenth of \( (\phi'_p - \phi'_{cv}) \), which is identical to the relationship found by Vaid and Sasitharan (1992) for Erksak sand.
Figure 4.5 Measured angles: (a) peak friction angles compared with Equation (4.11); (b) peak dilation angles compared with Equation (4.14)
4.5 Derivation of HSS model parameters for UWA sand

The HSS $E_{50}$ parameter (see Equation (4.3)) is defined specifically for drained triaxial compression tests on isotropically consolidated samples (Schanz et al., 1999) and consequently it is not a straightforward task to establish $E_{50}$ from the anisotropic tests discussed above. It was also noted from simulations of triaxial tests (using the FE software’s in-built soil element test) that the input small strain stiffness, $G_0$, matched the calculated small strain response for isotropically consolidated tests but under-estimated the initial stiffness in anisotropically consolidated tests; the under-estimation is a consequence of the generation of compression plasticity when loading in triaxial compression from an anisotropic stress state (noting a triaxial test is not a pure deviatoric test). Given this deficiency, HSS stiffness parameters for the triaxial tests were derived by FE simulation of each test and adjusted to provide a best fit to the measured $q$-$\varepsilon_a$ and $\varepsilon_v$-$\varepsilon_a$ data. The FE simulations were two-dimensional axisymmetric analyses with 15-node triangular elements and frictionless horizontal and vertical boundaries. The values of $\gamma_{0.7}$ and the peak friction and dilation angles were those measured in each test while the reference pressure ($p^{\text{ref}}$) was set equal to the consolidation stress $\sigma_3$. $E_{50}$ ($=E_{\text{oed}}$) and $G_0$ parameters were varied for each test to achieve a best fit using a minimisation algorithm (Doherty et al., 2012); results for individual tests presented in Appendix B.

The parameters that provided the best match between measured and calculated responses are listed in Table 4.1 and comparison plots are shown for all triaxial tests in Figure 4.6. It is seen that the calculated pre-peak variations of deviator stress with axial strain are generally in good agreement in all cases. The post peak softening of deviator stress is not modelled with the HSS model, but evidently this is only a significant issue above an axial strain ($\varepsilon_a$) of 6 to 8%. It can be seen that the transition from contraction to dilation occurs at higher axial strains for the HSS model. This transition is controlled by the internal calculation of $\phi'$ using Equation (4.7), which, for the ‘UWA sand’ is evidently too large when $\phi'$ is fitted to match the peak shear stress and $\psi$ is fitted to match the peak dilation. Volumetric strains continue to increase in all cases up until the dilation cut-off was attained; this was specified as the maximum void ratio ($e_{\text{max}}$) of 0.75 and often was not attained until an axial strain of 15 to 30% (as shown clearly in Appendix B).
Figure 4.6 Comparison of measured and calculated $q-\varepsilon_a-\varepsilon_v$ responses in typical triaxial tests (a) at $\sigma'_3 = 300$ kPa and different $D_r$ values, (b) for $D_r = 35-55\%$ at different stress levels, (c) for $D_r = 65-80\%$ at different stress levels.
The best-fit of the small-strain shear modulus used in the calculations is given the symbol $G_{0,C}$ in this thesis and differs from the actual $G_0$ moduli measured in the bender element tests. The $G_{0,C}$ values determined are presented in Figure 4.7 in a similar format to that used in Figure 4.2. The best-fit of the calibrated reference small-strain shear modulus is expressed in Equation (4.15).

$$G_{0,C} = G_{0,C}^{ref} \left(\frac{\sigma'}{p_a}\right)^{0.65}; \quad G_{0,C}^{ref} (\text{MPa}) = 140 \; F(e)$$

A comparison of Equation (4.15) with Equation (4.9) indicates that $G_{0,C}/G_0$ ratios increase from about 1.2 at $\sigma' = 20\text{kPa}$ to 1.5 at $\sigma' = 200\text{kPa}$. This is a relatively significant effect which clearly needs to be allowed for in finite element analyses of boundary value problems involving normally consolidated material in which the initial stress state is anisotropic.
A closer examination of the calculated response at small and intermediate strains is presented on Figure 4.8, which plots the measured and calculated secant Young’s modulus variations with axial strain for a loose and dense sand consolidated to axial and radial effective stresses of 80kPa and 40kPa respectively. It is evident that the hyperbolic format adopted by the HSS model (Equation (4.3)) provides an approximate match to the observed responses in both samples in the plotted strain range of 0.01% to 1%; this is the critical range for the serviceability limit state (SLS) in most geotechnical engineering problems.

Figure 4.8 Calculated and measured stiffness degradation for dense and loose samples (Tests 3 and 4)
The best-fit $E_{50}$ values derived from the triaxial tests (Table 4.1) display a similar dependence on the void ratio function (Equation (4.8)) to $G_{0,C}$ values (Equation (4.15)) but a stronger dependence on stress level. This trend is illustrated on Figure 4.7 which plots the best fit $E_{50}$ and $G_{0,C}$ values normalised by the void ratio function against the normalised confining stress ($\sigma'_3/p_a$). The comparable values of normalised stiffness at any given stress level support the general use of the void ratio function. However, while the power law relationship for $G_{0,C}$ provides a good representation of its dependence on $\sigma'_3$, it is evident that $E_{50}/F(e)$ varies in an approximately linear fashion with $\sigma'_3$ with a best fit equation as follows:

$$E_{50} = E_{50}^{\text{ref}} \left( \frac{\sigma'_3}{p_a^{\text{ref}}} \right)^{m=1}; \quad E_{50}^{\text{ref}} (\text{MPa}) = 22F(e)$$ (4.16)

Different stress level exponents ($m$) for $G_0$ and $E_{50}$ cannot be accommodated by the HSS model as shown in Equations (4.1) and (4.4). One way of partly overcoming this limitation is to select the small strain $m$ value of 0.65 for fine silica sand used at UWA (Equation (4.15)) and then ensure that the $E_{50}$ values evaluated using Equation (4.16) with $m = 0.65$ are reasonably well represented by Equation (4.4) over the $\sigma'_3$ range of interest; this procedure is employed in the subsequent analyses presented in this paper. In addition, and in keeping with Khoiri and Ou's (2013), the oedometric stiffness ($E_{oed}$) is assumed equal to $E_{50}$ and the unload-reload stiffness ($E_u$) is taken as $3E_{50}$.

The solution of boundary value problems using the HSS model requires specification of the $\phi'_p$ and $\psi_p$ values operating at the appropriate stress level and density. It has been shown that Equations (4.11) to (4.14) provide a reasonable means of determining these angles for ‘UWA sand’. However, the value of the mean effective stress at failure ($p'_f$) in Equation (4.12) is difficult to determine in general and it is proposed here to relate the friction angles with the initial horizontal effective stress ($\sigma'_3$), in the same way that the HSS model relates the stiffness terms to $\sigma'_3$. The resulting best-fit expression to the measured $\phi'_p$ values for ‘UWA sand’ is as follows (noting $33^\circ$ is the constant volume friction angle, $\phi'_{cv}$) and is shown on Figure 4.9a to provide a good match to the data (and arguably a better match than Equation (4.11)).

$$\phi' = \phi'_{cv} + 2 \left\{ D_r \left[ 5.4 - \ln(\sigma'_3/p_a) \right] - 1 \right\}$$ (4.17)
The HSS model employs Equation (4.7) to determine mobilised dilation angles $\psi_m$, as described previously, with samples continuing to dilate until the maximum void ratio ($e_{\text{max}}$) is reached. Bolton (1986) relates the difference between the peak and constant volume friction angle to the peak dilation angle in the plane strain. The same approach is employed here to provide a convenient means of estimating the dilation angle used for input to the HSS model. The best-fit dilation angle ($\psi$) used to match the data presented in Appendix B.2 for individual testing, and it is summarised in Table B.2. Figure 4.9b compares the measured ($\phi_p$-$\phi_{cv}$) values (with $\phi_{cv}$ fixed at 33°) and the best-fit dilation angle ($\psi$) used for the data matching shown on Figure 4.6. The correspondence between both parameters is seen on Figure 4.9b to be well represented by the following simple relationship (Equation (4.18)) which, together with Equation (4.17), can be used to determine the dilation angle for the HSS model.

$$\psi^o = 2 (\phi' - \phi'_{cv})$$  \hspace{1cm} (4.18)
Figure 4.9  (a) Comparison of measured peak friction angles with Equation (4.17), (b) best-fit dilation angle for HSS model compared with measured ($\phi'_p - \phi'_cv$) values
4.5.1 HSS parameter summary

A summary of the HSS parameters of UWA fine silica sand derived using the preceding equations is provided in Table 4.2. In the next section of this chapter, these parameters are employed in finite element analysis of two boundary value problems involving normally consolidated UWA sand to test the ability of the model to extrapolate beyond the simple triaxial stress path for which the parameters have been derived. A stress exponent, \( m \), of 0.65 is employed throughout and therefore parameters that depend on stress level such as \( E_{50} \) (and hence \( E_{50}^{ref}, E_{oed}^{ref}, E_{ur}^{ref} \)) \( \gamma_{0.7} \) and \( \phi'_p \) need to be assessed using the relevant equations provided for the \( \sigma'_3 \) value relevant to the problem being analysed; values of 50kPa and 100kPa are selected here as these were the initial effective stress imposed in the experiments described below.

Table 4.2 Hardening soil small (HSS) parameters for normally consolidated UWA-FSS

<table>
<thead>
<tr>
<th>Parameter</th>
<th>( D_r=40% )</th>
<th>( D_r=70% )</th>
<th>( D_r=45% )</th>
<th>( D_r=75% )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( e_{\text{max}} )</td>
<td>0.75</td>
<td>( e = 0.63 )</td>
<td>( e = 0.54 )</td>
<td>( e = 0.61 )</td>
</tr>
<tr>
<td>( e_{\text{min}} )</td>
<td>0.45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( F(e) )</td>
<td>Equation (4.8)</td>
<td>1.47</td>
<td>1.72</td>
<td>1.50</td>
</tr>
<tr>
<td>( m )</td>
<td>0.65</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \sigma'_3 )</td>
<td>For analyses presented on Figures 4.11 and 4.12</td>
<td>50kPa</td>
<td>50kPa</td>
<td>100kPa</td>
</tr>
<tr>
<td>( E_{50}^{ref} )</td>
<td>Determined using Equation (4.15) to give ( E_{50} ) values consistent with Equation (4.16) at representative ( \sigma'_3 ) value</td>
<td>25MPa</td>
<td>30MPa</td>
<td>33MPa</td>
</tr>
<tr>
<td>( E_{oed}^{ref} )</td>
<td>Assumed equal to ( E_{50}^{ref} ) (Schanz, 1998)</td>
<td>25MPa</td>
<td>30MPa</td>
<td>33MPa</td>
</tr>
<tr>
<td>( E_{ur}^{ref} )</td>
<td>Assumed equal to ( 3E_{50}^{ref} ) (Khoiri &amp; Ou, 2013)</td>
<td>75MPa</td>
<td>90MPa</td>
<td>99MPa</td>
</tr>
<tr>
<td>( c' )</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi'_o )</td>
<td>Equation (4.17)</td>
<td>35.7</td>
<td>39.0</td>
<td>36</td>
</tr>
<tr>
<td>( \psi )</td>
<td>Equation (4.18)</td>
<td>6.7</td>
<td>12.9</td>
<td>6</td>
</tr>
<tr>
<td>( K_{0}^{NC} )</td>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( p_{ref}^{NC} )</td>
<td>( p_a )</td>
<td>100kPa</td>
<td>100kPa</td>
<td>100kPa</td>
</tr>
<tr>
<td>( R_f )</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>( v_{ur} )</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>( \gamma_{0.7} )</td>
<td>Equation (4.10)</td>
<td>4.85\times10^{-5}</td>
<td>4.85\times10^{-5}</td>
<td>6.4\times10^{-5}</td>
</tr>
<tr>
<td>( G_{0,C}^{ref} )</td>
<td>Equation (4.15)</td>
<td>205MPa</td>
<td>241MPa</td>
<td>210MPa</td>
</tr>
</tbody>
</table>
4.6 Using the HSS model to solve boundary value problems

4.6.1 Pressuremeter tests

The first boundary value problem addressed involves inflation of a 20mm diameter (model) pressuremeter in ‘UWA sand’ located within a 393mm diameter, 400mm high steel pressure chamber. Full details relating to the pressuremeter, which is shown on Figure 4.10a, are described by Johnston et al. (2013) while Lim and Lehane (2014) describe the pressure chamber and associated operational details. It is important to note that the aspect ratio of this particular pressuremeter is unity and therefore cavity expansion approximates spherical rather than cylindrical conditions.

After locating the pressuremeter at the centre of the chamber (see Figure 4.10b and 4.10c), the sand was air pluviated into the chamber and then vibrated to achieve a relative density ($D_r$) of 70% ($e=0.54$) in the first test and a $D_r$ value of 40% ($e=0.63$) in the second test; this procedure is identical to that adopted in preparation of the triaxial samples. A vertical consolidation stress ($\sigma_{cv}$) of 100kPa was applied to the sand under $K_0$ conditions; the in-situ horizontal stress for this condition is assumed to be approximately 50kPa. A teflon coating on the inside of the rigid wall of the chamber minimised friction between the sand and the wall. The vertical stress was applied to the samples for a period of at least 1 week prior to testing, at which stage no measurable creep movements were detected by the displacement transducers.

Figure 4.10 (a) Pressuremeter with the inflated membrane, (b) pressuremeter in position prior to pouring sand, (c) schematic of the experimental set-up
The (axisymmetric) finite element mesh and associated boundary conditions employed to simulate the expansion of the pressuremeter in the sand are shown on Figure 4.11a. The parameters employed in the analysis were derived from Table 4.2 for a void ratio \( e \) of 0.54 and \( \sigma''_3 \) of 50kPa. The calculated cavity pressure \( (p) \)-cavity strain \( (\varepsilon_c) \) curves are compared with the measured curves for the two sand densities on Figure 4.12. It is evident that the match between calculations and measurements for loading and unloading is remarkably good for the denser sand but the response for the looser sand at larger cavity strains is over-predicted.

Figure 4.11 Axisymmetric finite element meshes for (a) pressuremeter tests and (b) footing tests
Predictions of footing and pressuremeter response in sand using a hardening soil model

Figure 4.12 Comparison between measured and calculated pressure-cavity strain relationship measured in pressuremeter tests (vertical stress = 100kPa)

4.6.2 Footing tests

Vertical load tests on 25mm and 75mm diameter ($D$) circular footings founded on UWA fine silica sand were performed in the same testing chamber employed for the pressuremeter tests. The steel footings had a thickness of 10mm and were located at a depth of 200mm in the 400mm high testing chamber. The sand sample preparation procedure employed was the same as that used for the pressuremeter tests but pluviation was halted to allow placement of the footing. A 12.7mm diameter rigid steel rod was screwed to the centre of the footing to facilitate application of the vertical load to the footing from above the chamber. Calibrated strain gauges acting as load cells located at the lower end of this rod allowed direct measurement of the load applied to the footing and hence eliminated potential errors due to friction between the rod and adjacent sand. As for the pressuremeter experiments, tests on footings with $D=75$mm were performed on sand prepared to relative densities of 40 to 45% and 70% with $\sigma'_v=100$kPa. A second
series of tests at similar relative densities were performed using 25mm diameter footings and a higher $\sigma'\nu$ value of 200kPa, with an assumed initial lateral effective stress of 100kPa.

The (axisymmetric) finite element mesh and associated boundary conditions employed to simulate the footing tests are shown on Figure 4.11b. The HSS parameters employed for each relative density were the same as those employed for the corresponding pressuremeter tests at $\sigma'\nu\approx50$kPa. For the reasons described previously, a different set of parameters, as shown in Table 4.2, was derived for the tests with $\sigma'\nu=200$kPa, $\sigma'\nu\approx100$kPa. The measured and calculated variations of applied pressure ($q_{app}$) with normalised settlement ($s/D$) of the footings are presented on Figure 4.13. It is noted that no settlement measurements were obtained during placing of the loading hanger used to facilitate application of stress to the footings. This initial applied stress was 27kPa for $D=75$mm and 240kPa for $D=25$mm and the comparisons of measurements and calculations on Figure 4.13 begin at this initial stress level.

As seen for the pressuremeter tests, the calculated footing responses in the denser sand are in reasonable agreement with the measurements for the cases with $\sigma'\nu=100$kPa (Figure 4.13a). However, as is evident on Figure 4.13b, settlements are over-predicted at both densities for the tests with $\sigma'\nu=200$kPa, and this is particularly significant for the looser sand. Apart from the $D_e=45\%$ and $\sigma'\nu=200$kPa case, the HSS calculations provide an estimate of the bearing stress to within 20% at $s/D$ ratios between 0.5% and 1%, which is typical range of normalised settlement for full scale footing under service loads.
Figure 4.13 Comparison between measured and calculated vertical stress-normalised settlement relationship measured in vertical loads tests on: (a) 75mm diameter footings ($\sigma'_v=100\text{kPa}$), (b) 25mm diameter footings ($\sigma'_v=200\text{kPa}$)
4.7 Concluding remarks

The comparisons shown on Figure 4.12 and Figure 4.13 provide an indication of the margin of error that may be expected using the HSS model in the solution of two typical boundary value problems involving reconstituted sand. Difficulties encountered in the derivation of appropriate parameters for the HSS model as well as some limitations of this model have been identified as follows:

- The input small strain (elastic) stiffness to the HSS model needs to be modified from the expected in-situ value to compensate for the model’s underestimation of small strain stiffness where the initial stress state is anisotropic. A full simulation of an anisotropically consolidated triaxial test needs to be undertaken to ensure appropriate HSS parameters are inferred.

- The HSS model only allows one exponent, \( m \), to describe the stress level dependence of the very small strain (elastic) stiffness \( G_0 \) and the secant stiffness at 50% mobilised strength \( E_{50} \). Experimental observations show that these exponents differ. Moreover, the HSS model assumes that stiffness varies only with the minor principal effective stress whereas previous research has indicated that the major principal effective stress is more influential.

- The observed stress level dependence of a parameter defining the rate of shear stiffness degradation with strain \( (\gamma_{0.7}) \) is not accounted for by the HSS model.

- The HSS predictions for the axial strain corresponding to the onset of dilation in anisotropically consolidated triaxial tests is greater than that observed in triaxial tests.

Every constitutive model has limitations, often because of the importance of having a small number of easily measurable parameters. The HSS model is designed with this aim and, despite the inadequacies outlined above, has been seen to capture many important characteristics observed in triaxial compression. Extension of the model to the prediction of pressuremeter and footing experiments was also seen to have reasonable success, but its relatively poor performance for the looser sand samples serves as a general reminder of the imperfect nature of constitutive models, even for reconstituted sand.
Chapter 5  Settlement of deep footings on reconstituted sand

Abstract

The paper examines methods for predicting the settlement of deep footings in reconstituted sand. Results from vertical load tests on deep circular plates within a laboratory pressure chamber are interpreted using data from triaxial tests, cone penetration tests (CPTs) and pressuremeter tests obtained for the same reconstituted sand. A simple non-linear CPT-based relationship is shown to match the response observed in the plate tests and be consistent with finite element analyses as well as other comparable physical tests. The relationships between foundation stiffness, and the sand’s small strain stiffness and its response to pressuremeter loading are also explored. Comparisons with full scale tests in the field reveal a strong effect of ageing on foundation stiffness, which appears to be better captured by small strain stiffness than CPT end resistance. Measurements confirm that vertical loading of a deep plate is analogous to the expansion of a spherical cavity.

5.1 Introduction

There is a multitude of published papers dealing with settlement predictions methods for shallow foundations on sand (e.g. Lutenegger & DeGroot, 1995; Poulos et al., 2001). The focus of this paper is to examine settlement prediction methods for deep footings or piers (which are equally applicable to the end-bearing component of bored piles) for which there is comparatively little information. As for shallow foundations, almost all of the approaches used in practice employ a single equivalent linear elastic modulus ($E_{eq}$) and rely on correlations with in-situ test data in the selection of an appropriate value. These correlations have been derived by back-figuring $E_{eq}$ values from case histories assuming linear elasticity. The following relationship for settlement ($s$) proposed by Randolph and Wroth (1978), and others, is commonly adopted:

$$s = \frac{\pi \ q \ D \ (1 - \nu^2)}{4 \ \frac{E_{eq}}{\eta}}$$

(5.1)
where \( q \) is the applied bearing stress, \( D \) is the footing (equivalent) diameter, \( \nu \) is Poisson’s ratio (which for drained conditions can be assumed as a constant value of 0.2), \( E_{eq} \) is the equivalent linear modulus for a surface footing and \( \eta \) is a depth correction factor. Doherty and Deeks (2006) show that \( \eta \approx 0.43 \) for a footing in an elastic full space (where potential for the soil to flow around the plate exists) while Randolph and Wroth (1978) recommend a \( \eta \) value close to unity for a pile base area, contesting that the soil along the pile shaft is already being deformed by the action of shear stresses. Burland (1970) and Chaudhry (1994) suggest values of 0.8 and 0.6 respectively for deep footings.

The range of \( \eta \) values used in practice is not surprising given the larger uncertainty concerning the value of \( E_{eq} \) employed in Equation (5.1). Guidelines offered by various methods to aid the selection of this ‘operational’ modulus cannot be generalized due to variable dependencies on stress level, strain level, density, stress history, anisotropy, ageing and soil layering. In relation to stress level, it is noted that the vertical stress gradient normalised by the stress at the base of the foundation is significantly less than that of a shallow foundation.

Equation (5.1) employs a linear modulus and hence predicts a linear load–settlement relationship. However, experimental and numerical research such as that presented by Lehane et al. (2008), Gavin et al. (2009), and Mayne (2014) has shown that, for shallow foundations on sand, \( q \) varies approximately with the square root of the normalised settlement \((s/D)\) and this relationship is relatively unique for a given initial sand state. The load tests on deep plates reported by Lee and Salgado (1999) also display a non-linear load settlement relationship but simple means for estimating the relationship are not available.

This paper uses a series of laboratory-scale plate load tests conducted at typical in-situ stress levels to explore a means of determining the full non-linear pressure \((q)\)–settlement \((s)\) relationship of deep footings in sand. One of the unique elements of this study is that the plate experiments were performed on reconstituted fine silica sand that was recreated in an identical fashion to that employed for a parallel series of triaxial, Cone Penetration Tests (CPTs) and pressuremeter tests. The comparison between these tests and the footing experiments is therefore not hindered by differences between the performance of the in-situ sand and that inferred from tests on sand reconstituted in the laboratory. This allows an assessment of non-linear predictive methods employing data from CPTs, small strain
(elastic) stiffnesses ($E_0$), triaxial compression tests and pressuremeter test data. The experimental procedures employed for the plate experiments and the associated soils tests are first described before examining the merits of various approaches.

5.2 Sand properties

All experiments were performed using a fine to medium sub-angular silica sand with a mean effective particle size ($D_{50}$) of 0.29mm, a uniformity coefficient of 2.1 and minimum and maximum void ratios ($e_{\text{min}}$ and $e_{\text{max}}$) of 0.45 and 0.75. The very small strain (elastic) Young’s modulus ($E_0$) of the sand was measured by Bagbag et al. (2017) (see Chapter 4) using bender element tests on samples reconstituted at various densities, who give the following best-fit expression for $E_0$:

$$\frac{E_0}{p_a} = 1500 \ F(e) \left(\frac{\sigma'_v}{p_a}\right)^{0.55} \quad (5.2)$$

Where, $p_a$ is atmospheric pressure (assumed equal to $=100\text{kPa}$), $\sigma'_v$ is the vertical effective stress and the void ratio function, $F(e)$, is defined for angular sand using the following expression (as expressed by Bovolenta, 2011; Fahey & Carter, 1993; Hardin, 1978; Hardin & Richart, 1963; Lehane & Cosgrove, 2000; Lehane & Fahey, 2002; Richart et al., 1970; Surarak, 2010; and many others):

$$F(e) = \frac{(2.17 - e)^2}{1 + e} \quad (5.3)$$

Bagbag et al. (2017) (in Chapter 4) also showed that the secant stiffness at 50% mobilised strength ($E_{50}$) in an isotropically consolidated sample sheared in a triaxial test showed a higher dependence on $\sigma'_v$ than $E_0$ and deduced the following equation:

$$E_{50} = 110 \ F(e) \ \sigma'_v \quad (5.4)$$

Triaxial tests showed that the sand was well characterised by a constant volume friction angle ($\phi'_c$) of 33º. Peak friction angles ($\phi'_p$) for the ten triaxial tests conducted Bagbag et al. (2017) are shown in Figure 5.1 where they are seen to compare well with the following expression proposed by Bolton (1986):
\[ \phi'_p = \phi'_c + 3[D_r(5.4 - \ln p'_f) - 1] \]  

(5.5)

where \( D_r \) is the sand relative density and \( p'_f \) is the mean effective stress at failure (in kPa).

Figure 5.1 Measured peak friction angles compared with angles given by Equation (5.5)

**5.3 Experimental procedures**

**5.3.1 Pressure chamber and sample preparation**

All experiments were performed in the same 393mm diameter, 400mm high pressurised cylindrical steel chamber employed by Lim and Lehane (2014). This system comprises a low friction, semi-rigid lateral wall with constant vertical stress and is classified as a BC3 chamber test (Parkin & Lunne, 1982). The inner wall of the steel chamber was coated in teflon to minimise friction and ensure uniform stresses within the sample. All tests involved dry sand, which was rained from a constant height of 1m; see Figure 5.2a. The sand were created by air pluviation technique where the drop heights and slot widths of the Sand-Rainer were varied to create different sand relative densities using air pluviation
Settlement of deep footings on reconstituted sand technique. For example, dense sand was achieved by using a slot width of 1.5mm and constant drop-height of 1m. The relative densities quoted below were derived from measured samples weights and volumes. Vertical stresses were applied hydraulically to the sand via a 40mm thick top plate; see Figure 5.2 and 5.5.

Figure 5.2 Pressuremeter test details (a) sand pluviation, (b) pressuremeter cell, (c) photo of set-up (d) sketch of set-up
5.3.2 Pressuremeter testing

Nine pressuremeter tests, designated P1 to P9 with details provided in Table 5.1, were conducted in the sand using the University of Western Australia (UWA) laboratory-scale pressuremeter described by Johnston et al. (2013) and shown on Figure 5.2b. This device has a diameter of 20mm and uses air as the pressurising fluid to expand a 0.3mm thick latex membrane. The radial displacement of the membrane is measured using strain gauged ‘feeler-arms’ rather than inferring displacements from measured volume changes, as is often the case for laboratory-scale pressuremeters (Beckerich et al., 1998; Gaudin et al., 2005; Schnaid & Houlsby, 1992; Zhao, 2008). The air pressure is supplied by an external air compressor via a 3mm diameter nylon tube that also acts as an electrical conduit for the feeler-arm transducers. The membrane stiffness was measured prior to each test by inflation of the membrane in air. The pressuremeter was located in the position indicated on Figure 5.2d (also see Appendix E, Figure E.8) at 180mm above the base of the chamber. A 20mm diameter aluminium rod extends above the pressuremeter through the central hole in the top plate and this rod was clamped to the chamber to keep the pressuremeter in position during sand pluviation and testing.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$\sigma'_v$</th>
<th>$D_r$</th>
<th>$G_0$</th>
<th>$P_{\text{lim},s}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>38</td>
<td>38</td>
<td>58</td>
<td>0.80</td>
</tr>
<tr>
<td>P2</td>
<td>38</td>
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<tr>
<td>P5</td>
<td>50</td>
<td>58</td>
<td>75</td>
<td>1.30</td>
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<td>71</td>
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<td>P7</td>
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<tr>
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</tr>
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<td>P9</td>
<td>100</td>
<td>69</td>
<td>117</td>
<td>1.90</td>
</tr>
</tbody>
</table>
Typical examples of measured cavity pressure ($p_c$) versus cavity strain ($\varepsilon_c$) curves are provided in Figure 5.3. Accurate identification of the lift-off stress was not possible as cavity strains could only be resolved to an accuracy of approximately 0.01% (due primarily to the small diameter of the device). Given that the pressuremeter was buried in normally consolidated sand, each expansion curve was adjusted to a lateral stress corresponding to an assumed $K_0$ value of 0.5 at a cavity strain of zero. Figure 5.3 highlights the strong influence of initial state (i.e. density and stress level) on the expansion curves.

![Figure 5.3 Typical pressuremeter test data showing effects of relative density and stress level](image)

The ratio of the pressuremeter membrane length to diameter is unity (as the device was designed for use in the geotechnical centrifuge with the associated high stress gradients). Appendix F.2 demonstrates numerically that this aspect ratio leads to conditions closely resembling spherical cavity expansion. These finite element calculations were performed...
using the hardening soil model (Schanz et al., 1999) and predicted that, over the full non-linear range up to \( \varepsilon_c = 10\% \), the pressure required to induce a given cavity strain in spherical cavity expansion was approximately twice that required for cylindrical expansion. Based on this finding, the pressures recorded by the device at any given cavity strain were reduced by a factor of 2 to derive cylindrical cavity expansion curves.

Spherical cavity expansion limit pressures \((P_{\text{lim,s}})\) were derived from the expansion curves using the conventional logarithmic extrapolation, as indicated in Figure 5.4a. Cylindrical cavity expansion limit pressures \((P_{\text{lim,c}})\) were then assumed to be half of the spherical values. The ratio of the CPT end resistances \((q_c)\), discussed below, to the \(P_{\text{lim,c}}\) values measured in sand at the same stress level and relative density \((D_r)\) are plotted on Figure 5.4b, where they are compared with the \(P_{\text{lim,c}}\) values obtained from the following expression proposed by Schnaid and Houlsby (1990); this equation was derived as a best-fit to measurements obtained using a cone pressuremeter in pressure chamber tests in coarse Leighton Buzzard sand (where \(\sigma_h\) is the in-situ horizontal stress).

\[
D_r = 0.09 \left( \frac{q_c - \sigma_h}{P_{\text{lim,c}} - \sigma_h} \right) - 0.3 \quad (5.6)
\]

It is seen that the recorded data are in reasonable agreement with Equation (5.6), which predicts \(P_{\text{lim,c}}\) increasing as \(D_r\) and \(\sigma_h\) increase. The slightly higher \(P_{\text{lim,c}}\) values inferred from Equation (5.6) may be due to the displacement induced by the cone pressuremeter employed by Schnaid and Houlsby (1990). The agreement increases confidence in the measured CPT and pressuremeter data.

Pressuremeter shear stiffness values \((G_p)\) can be derived as the half of the slope of the pressure vs (cylindrical) cavity strain expansion curve. It was found that \(G_p\) values at \(\varepsilon_c = 1\%\) were only 0.1 ± 0.05 times the very small strain elemental shear stiffness \((G_0)\) derived from Equation (5.2), using \(G_0 = E_0/2(1 + v)\). This small \(G_p (\varepsilon_c = 1\%)/G_0\) ratio is consistent with data reported for sands by Vucetic and Dobry (1991), and others, noting that the average shear strain in the sand mass surrounding a pressuremeter is approximately 25% of the cavity strain (Jardine, 1992).
Figure 5.4 (a) Derivation of $P_{\text{lim},s}$ in typical pressuremeter test, (b) Experimentally derived $P_{\text{lim},c}$ compared with Equation (5.6)
5.3.3 Plate tests

Thirty vertical load tests were conducted using 10mm thick steel footings/plates of 25mm, 50mm and 75mm diameter. The plates were founded on sand placed at the relative densities and vertical chamber stresses indicated in Table 5.2. As for the pressuremeter tests, sand was placed initially to a height of 180mm above the base of the chamber (see Figure 5.5 and Figure C.4). Each of the tested plates and the associated 17.3mm central loading rod (see Figure 5.5) was then placed at the sand surface and sand placement was resumed to fill the chamber; see Figure 5.5a. The rod was screwed into the centre of each footing and included calibrated strain gauges located at either end to allow measurement of the load applied at the top of the rod and that transmitted directly to the footing (see Figure 5.5b). Vertical load was applied via the loading rod using the hanger arrangement shown on Figure 5.5c and 5.5d. All footing tests were preceded by a period of at least 2 days after the chamber pressure was applied to minimise effects of creep. Laser displacement transducers were used to ensure that there was no measurable ongoing settlement of the top plate at the time of testing.

Loads were applied to each footing in 10 to 15 increments, with a 5-minute creep period allowed after application of each increment; measured creep rates at the end of each period were extremely small at footing settlement to diameter ratios \( s/D \) less than 1%. Footings were generally loaded to a settlement to footing diameter \( s/D \) ratio of between 0.05 and 0.15 over a period of 2 hours. Footing settlements (recorded by laser displacement transducers) and strain gauge data were logged at 10-second intervals.

5.3.4 Cone Penetration Testing (CPT)

An actuator, which is normally employed in UWA centrifuge experiments, was used to perform CPTs. These involved insertion, via the small circular opening in the upper plate, of a 7mm diameter cone penetrometer at a speed of 3mm/min. The CPT end resistances \( q_c \) stabilised to a near constant value after a penetration of about 70mm and the \( q_c \) values listed in Table 5.2 are the average values recorded over the penetration depth of 70mm to 300mm. Examples of the type of CPT profiles recorded in the chamber are available in Lim and Lehane (2014).
Table 5.2 Summary of Footing Tests

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<tr>
<th>Test No.</th>
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<td>(%)</td>
<td>(MPa)</td>
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Figure 5.5 Plate test details (a) raining sand with plate in position, (b) photo showing plate and load cells on rod attached to the footing, (c) photo of set-up for plate test, (d) sketch showing set-up for footing tests
5.4 Plate load test results

The variations with the plate settlement diameter ratio \((s/D)\) of the applied vertical stress, \(q\) (as derived from the lowermost load cell on the loading rod; see Figure 5.5b) are presented for a number of typical cases on Figure 5.6. Figure 5.6a presents results for 50mm diameter footings founded on sand with three different relative densities \((D_r)\) with a constant chamber consolidation pressure \((\sigma'v)\) of 38kPa. Figure 5.6b also plots data for 50mm diameter footings but for a single sand \(D_r\) value of 46\% and four different \(\sigma'v\) values of 38kPa, 50kPa, 100kPa and 200kPa. Figure 5.6c examines the effect of footing diameter with \(D=25mm, 50mm\) and \(75mm\) in sand with \(D_r=46\%\) and with a vertical stress of 200kPa.

The comparisons on Figure 5.6a and 5.6b highlight the dependence of the footing load-settlement relationship on the relative density and stress level i.e. the sand state. This dependence provides the basis for empirical correlations with in-situ test data such as the CPT \(q_c\) value, which varies directly with the sand state. Clearly, however, the measured relationships between applied stress and settlement are non-linear and use of a single operational modulus \((E_{eq}\) in Equation (5.1)) cannot be justified.

Although the load applied to the 75mm diameter footings is an order of magnitude larger than that applied to the footings with \(D=25mm\), it is evident on Figure 5.6c that \(q\) vs \(s/D\) relationships for all three footing diameters vary within 10\% of the mean trend and show no systematic dependence on \(D\) (although boundary effects for the larger plate may have contributed to a slightly stiffer response). Briaud and Gibbens (1997) and Lehane et al. (2008) also found that the \(q\) vs \(s/D\) characteristics were closely comparable for different footing widths in the same sand, while Lehane et al. (2008) show that this trend is predicted numerically for shallow footings using a hardening non-linear elasto-plastic constitutive model for the sand (Schanz et al., 1999).
(a) $D = 50\text{mm}$  
$\sigma'_v = 38\text{kPa}$  
$F8, \ D_t = 73\%$  
$F5, \ D_t = 63\%$  
$F2, \ D_t = 46\%$

(b) $D = 50\text{mm}$  
$D_t = 46\%$  
$F29, \ \sigma'_v = 200\text{kPa}$  
$F20, \ \sigma'_v = 100\text{kPa}$  
$F11, \ \sigma'_v = 50\text{kPa}$  
$F2, \ \sigma'_v = 38\text{kPa}$
5.5 Settlement prediction approaches

The ability of various in-situ and laboratory test based approaches for predicting the load-displacement responses measured in the footing tests are now examined.

5.5.1 Use of the CPT $q_c$

Database reviews of field tests performed by Mayne (2014), Gavin et al. (2009), and Lehane (2012) indicate the following general relationship for full scale shallow foundations on sands:

$$\frac{q}{q_c} \approx \lambda \frac{s}{\sqrt{D}}$$

(5.7)
where the constant $\lambda$ varies from about 0.65 for typical aged silica sands to about 0.4 for more compressible sands. The $q_e$ value used in Equation (5.7) is typically the average CPT end resistance in a zone extending over one equivalent foundation diameter ($D$) below the footing level.

Although no equivalent formulation similar to Equation (5.7) has been proposed for deep footings on sand, a comparable relationship can be derived by determining a best fit equation to the plate load tests in Ticino sand presented by Lee and Salgado (1999) and Ghionna et al. (1994). Lee and Salgado (1999) employed a non-linear elasto-plastic soil model with parameters representative of reconstituted Ticino sand in a finite element model of the plate tests and found good agreement between measurements and predictions, as illustrated on Figure 5.7. The CPT $q_e$ resistance was computed using the same soil model and the CONPOINT algorithm (Salgado, 1993) and was also in good agreement with measurements. Lee and Salgado (2000) extended their research using the same numerical model to derive pile base load-settlement relationships for a 600mm diameter pile, wished-in-place to depths of 5m, 10m and 20m in dry sands with relative densities ($D_r$) between 30% and 90%. Regression analyses of all these relationships indicated that they were well represented by the following simple equation for $s/D \leq 0.1$:

$$\frac{q}{q_e} = e^{-D_r \left(\frac{s}{D}\right)^{0.6}}$$

(5.8)

An indication of the degree of fit obtained using Equation (5.8) is provided by the example on Figure 5.7, which shows that the equation approximates the actual response in the Ticino sand plate load tests as well as the finite element computations.

Equation (5.8) is compared with the tests on the 75mm diameter footings in the UWA chamber on Figure 5.8 and evidently provides a reasonable representation with estimated $q$ values for this plate diameter generally being within 20% of the measured values for all $s/D$ values of interest; similar agreement was found for the 25mm and 50mm diameter footings. It can be concluded that Equation (5.8) provides a simple, but effective, means of determining the full non-linear load-displacement response of a deep plate in reconstituted sand.
Figure 5.7 Comparing of plate response measured by Lee and Salgado (1999) with finite element predictions and Equation (5.8)

Figure 5.8 Comparison of measured bearing stresses in plate tests with those predicted using Equation (5.8) at different s/D values for footing with D=75mm
The similarity between the responses of the embedded plates in UWA sand with the tests in Ticino sand, which were conducted at the base of a steel casing within the testing chamber, indicates that the sand immediately above the UWA plates has little influence on the measured and predicted responses over the $s/D$ range considered. This similarity suggests that the appropriate depth correction factor ($\eta$) to employ in Equation (5.1) is closer to unity than to the full elastic space solution of 0.43. A $\eta$ value of 0.8, as proposed by Mayne and Poulos (1999), is therefore employed in the following interpretation.

Equations (5.1) and (5.7) can be combined to derive an expression for $E_{eq}/q_c$ ratios for shallow footings in the field.

$$
\frac{E_{eq}}{q_c} = \frac{\eta \pi (1 - v^2)}{4} \lambda \left(\frac{S}{D}\right)^{-0.5}
$$

(5.9)

For a $v=0.2$ and an average $\lambda$ value of 0.5, $E_{eq}/q_c$ ratios for shallow footings (for which $\eta=1$) in the field are about 12 and 3.8 at $s/D=0.1\%$ and $1.0\%$, respectively. Combining Equations (5.1) and (5.8) a similar expression for deep foundation in laboratory sand can be derived

$$
\frac{E_{eq}}{q_c} = \frac{\eta \pi (1 - v^2)}{4} e^{-D_r} \left(\frac{S}{D}\right)^{-0.4}
$$

(5.10)

For $v=0.2$ and $D_r = 0.6\pm 0.2$, $E_{eq}/q_c$ ratios are around 5.3$\pm$1.0 at $s/D=0.1\%$ and 2.1$\pm$0.4 at $s/D=1.0\%$, for the anticipated depth correction factor ($\eta$) value of 0.8.

Equations (5.9) and (5.10) therefore indicate that $E_{eq}$ values in field are about double those in reconstituted sand with a given $q_c$ value, reflecting a significant effect of ageing on stiffness, as noted by Mitchell (2008), and others. Baldi et al. (1989), for example, show that the $E_{eq}/q_c$ ratio of a freshly deposited sand in the laboratory is also about half of that of a natural normally consolidated sand. It is evident that the CPT $q_c$ value does not capture the effects of various ageing processes on foundation stiffness, which explains the wide variety of $E_{eq}/q_c$ recommendations found in the literature. It is also noted that differences in stiffness between the plate tests and shallow footing tests may also be expected due to contrasting initial stress gradients and variations of $q_c$ with normalised depth below the foundations.
5.5.2 Use of elastic (small strain) stiffness, $E_0$

The growing use of the seismic cone has prompted significant interest in settlement prediction methods that employ the small strain (elastic) stiffness, $E_0$. Equivalent linear stiffness values ($E_{eq}$), as defined in Equation (5.1), were derived for the plate tests by assuming a depth correction factor ($\eta$) of 0.8. These $E_{eq}$ values are normalised by $E_0$, as derived using Equations (5.2) and (5.3), and plotted against the corresponding $s/D$ values for the 75mm diameter footings on Figure 5.9a. Given that $s/D$ values for most deep footings at working loads range from 0.2% to 1%, Figure 5.9a indicates that a first order estimate of settlement using Equation (5.1) can be obtained by assuming an operational modulus ($E_{eq}$) of 0.15$E_0$ and employing Mayne and Poulos (1999) recommended $\eta$ value of 0.8. Bounds to the $E_{eq}/E_0$ data for the 75mm diameter plate tests are shown on Figure 5.9a and were constructed using the following modified hyperbolic equation.

$$
\frac{E_{eq}(\eta = 0.8)}{E_0} = \frac{1}{1 + \left(\frac{s/D}{(4 \pm 2)10^{-4}}\right)^{0.6}} \quad ; \quad \frac{s}{D} > 0.0005 
$$

(5.11)

It is seen that this equation provides a reasonable estimate of $E_{eq}$ for $s/D$ values over the displacement range of interest (with $s/D > 0.05\%$). This mean trend given by Equation (5.11) is compared on Figure 5.9b with $E_{eq}/E_0$ variations with $s/D$ presented by Gavin et al. (2009) for shallow foundation tests on Texas, Blessington and Shenton Park sands. It is evident the $E_{eq}/E_0$ values at any given settlement ratio in the aged (Eocene) alluvial Texas sand and the very heavily overconsolidated glacial Blessinton sand are significantly higher than equivalent ratios found in the plate load tests. There is clearly no general relationship that can be established to relate $E_{eq}/E_0$ with $s/D$, although it would appear from the reasonable agreement of the plate test data on Figure 5.9b with $E_{eq}/E_0$ ratios determined from tests on the relatively young dense sand at Shenton Park (Lehane et al., 2008) that $E_{eq}$ is likely to correlate more consistently with $E_0$ than with $q_c$ in such sands.
Figure 5.9 (a) Interpreted $E_{eq}/E_0$ ratios in plate load tests, (b) Comparison of $E_{eq}/E_0$ ratios
5.5.3 Equivalent modulus derived from triaxial data

Some practitioners adopt the Youngs modulus at 50% mobilized strength ($E_{50}$) in a triaxial test as an estimate of the equivalent linear stiffness ($E_{eq}$) in foundation settlement predictions. Comparing the $E_{eq}$ values back-figured for the 75mm diameter plate on Figure 5.9a with $E_{50}$ as determined using Equation (5.4) indicates that this is a reasonable approximation for $s/D$ ratios of the order of 2.5%. Settlement ratios at the base of bored piles under working conditions are typically of this magnitude and therefore Equation (5.1) with $E_{eq}=E_{50}$ and $\eta=0.8$ can be used to provide an estimate of bored pile base stiffness for settlement calculations.

5.5.4 Constitutive model with parameters from triaxial data

Bagbag et al. (2017) (Chapter 4) present a detailed description of the triaxial tests performed on the reconstituted sand employed in the experiments and then formulated parameters for the Hardening Soil model with small strain overlay, referred to as the HSS model (Schanz et al., 1999). This model employs triaxial data alone in the derivation of the constitutive parameters.

Finite element predictions employing the HSS model for the four cases presented by Bagbag et al. (2017), in Chapter 4 and Appendix D, are plotted on Figure 5.10, where they are compared with the corresponding footing tests. These predictions involved two footing diameters, two different initial stress conditions and two different relative densities. The comparisons show good agreement for the denser sand ($D_r=76\%$) but poorer agreement for the looser sand ($D_r=46\%$), where stiffness is over-predicted by between 50 and 100%. The predictions obtained using Equation (5.8), derived from the research of Lee and Salgado (1999, 2000), are also shown on Figure 5.10 and seen to be generally in better agreement with measured data than the predictions with the HSS model.
Equation (5.8)

\[ \sigma_{vo} = 200 \text{kPa} \]
\[ D_i = 76\% \]
\[ D = 25 \text{mm} \]

(a)

Equation (5.8)

\[ \sigma_{vo} = 200 \text{kPa} \]
\[ D_i = 46\% \]
\[ D = 25 \text{mm} \]

(b)
Figure 5.10 Comparisons of measured plate responses with predictions obtained using Equation (5.8) and with finite element predictions employing the HSS model.
5.5.5 Simplified approach using pressuremeter data

A relationship between the pressuremeter data and the plate load tests was investigated by nominally assuming that the settlement ratio \((s/D)\) of a deep plate was equivalent to half of the cavity strain \((\varepsilon_c)\) induced by a spherical cavity expansion. A best fit was obtained if the net cavity pressure \((p_0 - \sigma_h)\) was simply factored by 1.6. Three examples illustrating the accuracy of this approach for different sand relative densities and stress levels are shown on Figure 5.11. It may therefore be inferred that, as postulated by Randolph et al. (1994), and others, loading of a deep foundation is analogous to the expansion of a spherical cavity. It is noted, however, that at ultimate conditions, the expression of Randolph et al. (1994) indicates a higher footing stress \((q_{ult})\) to \(P_{lim,s}\) ratio of about 2.6 ±0.4 for typical sand friction angles.
Settlement of deep footings on reconstituted sand

Figure 5.11 Predicting footing settlement using pressuremeter data for 75mm diameter plates
5.6 Conclusions

Deep footing tests and associated mechanical characterization tests (triaxial tests and in-situ CPT and pressuremeter tests) conducted in reconstituted sand at an identical stress state revealed the following:

1. A simple non-linear relationship involving the CPT \( q_c \) value and relative density (i.e. Equation (5.8)) captures the responses observed in the deep footing tests and is consistent with other deep plate load tests performed in Ticino sand.

2. Although the CPT \( q_c \) value provides a useful means of normalising data on reconstituted sands, the vertical stiffness of footings on natural (aged) sands is about double that indicated by reconstituted sand with the same CPT \( q_c \) value. Enhancement in footing stiffness due to ageing effects is evidently not fully captured by increases in \( q_c \) values with ageing.

3. The settlement of deep plates on reconstituted sand can also be estimated approximately using the small strain (elastic) stiffness \( E_0 \) using the approximate expression given in Equation (5.11). This trend is comparable to that reported for younger sands tested in the field, indicating that \( E_0 \) is better correlated with operational foundation stiffness than the CPT \( q_c \) value for these sand types.

4. The responses of deep plates on reconstituted dense sand were predicted with reasonable accuracy using a particular non-linear elasto-plastic soil model with parameters derived from corresponding triaxial tests. However, the same constitutive model over-predicted the plate response in loose-medium dense sand.

5. A comparison of measured bearing stress-settlement ratios for deep plates with spherical cavity expansion curves provides general support for the hypothesis made by a number of workers that loading of a deep foundation is analogous to the expansion of a spherical cavity.
Chapter 6  Evaluating the ability of the Extended Mohr-
Coulomb model to extrapolate from triaxial
stress paths to general stress space

Abstract

In this study a simple shear hardening soil constitutive model is calibrated using triaxial compression data on reconstituted laboratory sand. The model is then applied to simulate the response of miniature pressuremeter tests in the same material, reconstituted in the same way. The ability of this simple constitutive model to extrapolate from triaxial stress paths to more complex boundary value problems, without differences in soil state or fabric, is assessed. Two forms of the plastic potential in the deviatoric plane are considered, both giving identical response in triaxial stress space.

6.1 Introduction

The aim of this paper is to determine if the relatively simple Extended Mohr-Coulomb (EMC) soil model (Doherty & Muir Wood, 2013; Muir Wood, 2004), with 6 parameters can be calibrated using triaxial test data and then applied to simulate a more complex boundary value problem involving a miniature pressuremeter in the same material reconstituted in the same way. One of the EMC model advantages over the HSS model is that the process of fitting the EMC model parameters to the triaxial test data is very straightforward. Moreover, the number of unknown parameters in EMC model make it easier than HSS to apply the optimisation solution for the purpose of matching the triaxial data. The study therefore explores the ability of this constitutive model to extrapolate from element test data to a more complex boundary value problem, involving very different stress paths, without uncertainties associated with the state or fabric influencing the results. Studies such as this are rare in literature, but are a fundamentally important way of assessing the predictive capability of constitutive models and the influence various components of the models have. In this study, particular focus is given to the form of the plastic potential in the deviatoric plane.

Tests were performed using a manufactured fine-medium sand supplied by Sibelco (2017) with the index properties shown in Table 6.1 (also see Appendix A). This sand (or
Chapter 6

minor variants of it) has been used extensively in geotechnical centrifuge testing at the University of Western Australia (Govoni et al., 2006; Lee et al., 2013; O’Loughlin & Lehane, 2003; Teh et al., 2008; Xu & Lehane, 2008). However, despite its widespread use as a standard geotechnical laboratory sand, very few studies have examined the fundamental mechanical behaviour of the material. Bagbag et al. (2017) (see Chapter 4) recently addressed the shortage of high quality element test data on this material by presenting the results of 10 anisotropically consolidated drained triaxial compression tests over a range of initial stresses and densities. This paper uses these results directly in the assessment of the EMC model.

In the following section of the paper, the triaxial testing procedure is briefly described before a summary of the EMC model is presented. The influence of stress and density on the peak friction angle and elastic secant stiffness at low strains is presented. The EMC model is then calibrated against each of the tests using single variable numerical optimisation over the stress-strain and volumetric strain-axial strain data. Finally, the calibrated EMC model is used to simulate the boundary value problem involving a miniature pressuremeter. Two forms of the plastic potential in the deviatoric plane are investigated.

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6.2 Triaxial testing

Details of the axial ($\sigma'_a$) and radial ($\sigma'_r$) consolidation stresses and the relative density ($D_r$) of each triaxial test is given in Table 6.2.

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</tbody>
</table>

The triaxial samples were created by air pluviation and then vibrated to achieve the desired relative densities. Samples were saturated under a backpressure of 500kPa and then consolidated anisotropically prior to drained shearing in compression. Displacement transducers were pinned to the membranes to measure axial strains ($\varepsilon_a$) locally over the middle third of each sample. Volumetric strains were recorded by measuring the flow of pore water into or out of the sample using a GDS advanced pressure volume controller (see Appendix B).

6.3 The Extended Mohr Coulomb (EMC) soil model

The EMC model (Doherty & Muir Wood, 2013; Muir Wood, 2004) uses a Mohr-Coulomb yield surface given by

$$F(\sigma, a, \phi_y) = (p' - a) \sin \phi_y + J K(\theta, \phi_y) = 0$$

(6.1)

where

$$J = q / \sqrt{3}$$

(6.2)
where \( \theta \) is Lode’s angle, measuring \( \pi/6 \) for triaxial compression and \(-\pi/6\) for triaxial extension. For triaxial stress states,

\[
p' = \frac{(\sigma'_a + 2\sigma'_r)}{3} \quad (6.4)
\]

\[
q = \sigma'_a - \sigma'_r \quad (6.5)
\]

(noting compression stresses are assumed to be negative). The mobilised friction angle \( \phi_y \) defines the current size of the yield surface, together with a constant attraction component \( a \) (which can be related to an effective cohesion \( c' = atan\phi_y \)). A hardening rule is introduced that allows the yield locus to expand to some limiting failure size, given by \( \phi_p \), which is the peak friction angle for the material (see Figure 6.1).

Figure 6.1 The EMC model with yield locus separating elastic and elasto-plastic regions and failure locus separating elasto-plastic and inaccessible regions (after Doherty & Muir Wood, 2013)

The model follows Taylor's (1948) proposed link between dilatancy and mobilised friction to define the ratio of incremental plastic shear strain and incremental plastic volumetric strain:
Evaluating the ability of the EMC model to extrapolate from triaxial stress paths...

\[
\frac{\delta \varepsilon^p_p}{\delta \varepsilon^p_j} = - \frac{J}{(p' - a)} - \frac{\sin \phi_{cv}}{G(\theta, \phi_{cv})}
\]  

(6.6)

where \(G(\theta, \phi_{cv})\) is a function that defines the shape of the plastic potential in the deviatoric plane and \(\delta \varepsilon^p_p\) and \(\delta \varepsilon^p_j\) are the incremental plastic and volumetric shear strains, respectively. A positive value of \(\delta \varepsilon^p_p\) indicates dilation whereas a negative value indicates contraction. For triaxial conditions

\[
\varepsilon_v = \varepsilon_a + 2\varepsilon_r 
\]

(6.7)

\[
\varepsilon_j = \frac{2}{\sqrt{3}} (\varepsilon_a - \varepsilon_r)^2 
\]

(6.8)

where \(\varepsilon_a\) is the axial strain and \(\varepsilon_r\) is radial strain. The first term on the right hand side of Equation (6.6) is the ratio of shear to mean stress (and therefore a measure of the mobilised friction). The second term represents a critical stress ratio at which constant volume shearing can occur (i.e. Equation (6.6) is zero). The magnitude of this stress ratio is determined by an input parameter \(\phi_{cv}\). It can be deduced that the flow rule in Equation (6.6) corresponds to the original Cam clay plastic potential function, which are plotted along with a set of yield loci in Figure 6.2.

![Figure 6.2](image)

Figure 6.2 The EMC model with the direction of the plastic strain increments shown normal to the plastic potential (dashed line) (after Doherty & Muir Wood, 2013)
The EMC model describes a mechanical response in which the stiffness of the material falls steadily as it is sheared towards failure. This was achieved by introducing a simple relationship between plastic distortional strain and mobilised friction, in which the plastic distortional strain increases exponentially (towards infinity) as $\phi_r$ approaches $\phi_p$:

$$\varepsilon^p_J = \frac{\beta \phi_y}{\phi_p - \phi_y} - \frac{\beta \phi_y^0}{\phi_p - \phi_y^0} \quad \text{(for } \phi_y > \phi_y^0)$$

(6.9)

where $\beta$ is a model input parameter. $\phi_y^0$ is the maximum previously mobilised friction angle defined when establishing the initial conditions and is used in Equation (6.9) to ensure that the plastic shear strain is zero when the initial yield surface (defined using $\phi_y^0$) is first intersected (i.e. $\phi_r = \phi_y^0$). It should be noted that when $\phi_r < \phi_y^0$ the model remains elastic and Equation (6.9) is not invoked, so the plastic shear strain cannot be negative. It is then convenient to introduce the constant

$$\alpha = \frac{\phi_y^0}{\phi_p - \phi_y^0}$$

(6.10)

as it can then be shown that $\varepsilon^p_J = \beta$ when

$$\frac{\phi_y}{\phi_p} = \frac{1 + \alpha}{2 + \alpha}$$

(6.11)

The complete list of model parameters is given in Table 6.3. In addition to these parameters, the initial stress state and the maximum previously mobilised friction angle $\phi_y^0$ must be defined as part of the definition of the initial conditions.

The response of the EMC model for an isotropically consolidated drained triaxial compression test is illustrated in Figure 6.3 using the following fixed parameters: $G = 4\text{MPa}$, $\mu = 0.25$, $\phi_p = 30^\circ$, $a = 0$, $\beta = 0.005$, and three different values of $\phi_{cv}$ ($20^\circ$, $30^\circ$ and $40^\circ$). As shown in Figure 6.3a, the shear stress: strain response is independent of $\phi_{cv}$. The volumetric response, however, depends on the difference between $\phi_{cv}$ and $\phi_p$ (Figure 6.3b), as noted by Doherty and Muir Wood (2013). $\phi_{cv}$ should not be thought of as the critical state friction angle, as this would imply the material always tends to an ultimate state at which constant volume shearing occurs at this friction angle. Rather, $\phi_{cv}$
should be regarded as the mobilised friction angle at which constant volume shearing takes place. This may be an ultimate state in the case \( \phi_{cv} = \phi_p \), a temporary state in the case \( \phi_{cv} < \phi_p \), or a state never reached in the case \( \phi_{cv} > \phi_p \).

This formulation makes fitting parameters to triaxial data particularly convenient. Once elastic properties are determined and the peak friction angle evaluated by direct inspection of the data (as described in the following sections of the paper), the hardening parameter (\( \beta \)) can be varied to achieve the best possible match to the stress strain response. Independently, \( \phi_{cv} \) can be varied to match the volumetric response.

A stress depend elastic stiffness model could also be easily introduced into the EMC model. However, for applications in this paper, this is considered unnecessary as both the triaxial and pressuremeter tests have an initial uniform stress state.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>( G )</td>
<td>Elastic shear modulus [kPa]</td>
</tr>
<tr>
<td>( \mu )</td>
<td>Poisson’s ratio [-]</td>
</tr>
<tr>
<td>( \phi_p )</td>
<td>Peak friction angle [degrees]</td>
</tr>
<tr>
<td>( \phi_{cv} )</td>
<td>Constant volume friction angle [degrees]</td>
</tr>
<tr>
<td>( a )</td>
<td>Attraction [kPa]</td>
</tr>
<tr>
<td>( \beta )</td>
<td>Hardening parameter [-]</td>
</tr>
</tbody>
</table>
Figure 6.3 EMC model response in triaxial compression
6.4 Elastic properties

The EMC model is capable of modelling the gradual reduction in stiffness with increasing shear stress and strain via plastic strain hardening. It is therefore considered reasonable to adopt the elastic properties associated with a moderately small strain range (and not the very small strain range). Secant Young’s modulus values ($E_{sec}$) from eight triaxial tests on UWA sand prepared at various relative densities ($D_r$) are plotted in Figure 6.4, where

$$E_{sec} = \frac{q}{\varepsilon_a} \quad (6.12)$$

From the test data in Figure 6.4, $E_{sec}$ values at axial strains ($\varepsilon_a$) of 0.01, 0.001 and 0.00075 were extracted and are plotted in Figure 6.5 against the radial effective stress (cell pressure), which was held constant throughout the shearing stage of the tests. It can be seen that, for a given relative density and strain level, the secant Young’s modulus varies in a near linear fashion with the radial effective stress (also the minor principal effective stress).
Figure 6.4 Measured secant Young’s modulus

Figure 6.5 Secant Young’s modulus values for various stress and strain levels
6.5 Peak friction angle

Peak friction angles from the eight triaxial tests were determined by evaluating the maximum $q/p'$ ratio mobilised in the shearing stage of the test. These values are plotted against the radial effective stress in Figure 6.6. The data show the expected trend of decreasing peak friction angle with increasing confining pressure and an increasing peak friction angle with increasing density.

![Figure 6.6 Peak friction angle](image)

Figure 6.6 Peak friction angle
6.6 Deriving the stress-strain and volumetric EMC parameters

With the elastic stiffness evaluated at relatively small strains using local strain gauges and the peak friction angle determined as described in the previous section, numerical optimisation can now be used to determine parameters for the EMC model to match the measured stress-strain and volumetric response. This process is now described.

6.6.1 Matching triaxial response

Optimisation problems usually seek to minimise an objective function, which measures the difference between numerically generated model data and measured test data. The difference between a given set of model and test data is expressed as a single scalar value, $I$. A large value of $I$ indicates a large difference between the model and test data, whereas a value of zero indicates a perfect match (Doherty et al., 2012). There are a number of rational methods for comparing model and test data and computing the value of the objective function $I$. In this paper, the method described in detail by Mattsson et al. (2001) has been used where the objective function is formed by summing the minimum distance of each test data point from a straight-line fit between the two nearest model data points ($d_{\text{min}}$), as shown in Figure 6.7.

![Figure 6.7](image-url)
Evaluating the ability of the EMC model to extrapolate from triaxial stress paths…

The eventual value of the objective function $I$ can be expressed as

$$I = \frac{w}{n} \sum_{j=1}^{n} d_{\text{min}}^{j}$$

(6.13)

where $n$ is the number of experimental data points and $w$ is a weighting factor which assigns the relative importance of the particular data set (or even data point).

In this paper, a weighting factor of unity has been assumed. Test data points have only been included if the distance from the particular test point to model data point $i+1$ is greater than the distance to model data points $i$ and $i-1$. This ensures that $I$ does not increase if the model data extend further than the test data, or vice versa.

### 6.6.2 Stress-strain response

The EMC parameters that control the stress-strain response in triaxial compression are the Young’s modulus ($E$), Poisson’s ratio ($\mu$), peak friction angle ($\phi_p$) and the hardening parameter ($\beta$). The EMC model is capable of modelling the gradual reduction in stiffness with increasing shear strain via plastic strain hardening, and for this exercise, the elastic properties associated at a moderately small strain range are selected. In matching the EMC model to the triaxial compression data, secant Young’s modulus values ($E_{\text{sec}}$) for an axial strain of 0.00075 in Figure 6.5 were used. A typical elastic Poisson’s ratio of 0.2 was adopted for all cases and peak friction angles from Figure 6.6 were used. The attraction parameter was fixed to zero as the sand has no cementation. The parameter $\phi_{cv}$ has no impact on the shear stress – axial strain response in drained triaxial compression, and therefore given an arbitrary value of 30°. The only remaining unknown parameter, $\beta$, was adjusted to fit to the data using numerical optimisation. Figure 6.8 presents a comparison of the measured stress-strain response with the optimised EMC response for all tests. The three dominant input parameters used to achieve the match are shown above each subplot. In general there is a very good match to the data, although the softening at axial strains of around 6% and above cannot be replicated by the EMC model.
Figure 6.8 Comparison of measured and computed stress-strain response
6.6.3 Volumetric response

With parameters determined to match the stress-strain response, the remaining unknown EMC parameter, $\phi_{cv}$, can be determined by matching the model response to the measured shear strain versus volumetric strain response. Numerical optimisation was again used to find the optimum value of $\phi_{cv}$. Figure 6.9 presents comparisons of the model response using optimised $\phi_{cv}$ values against measured test data for all eight tests. Data is plotted against the shear strain invariant, $\varepsilon_j$. It can be seen that there is a very good match to the volumetric response at lower shear strains. At higher strains (and higher mobilised shear stresses) the EMC model tends to over predict the volumetric expansion. The tests data exhibit a clear slowing in dilation, possibly towards some critical state, whereas the EMC continues to dilate at the same rate indefinitely. As the EMC model does not capture the post peak softening it may over predict strength and stiffness at higher strains. The model could in future be adapted to include dilation cut-off.
Figure 6.9 Comparison of test data and EMC volumetric response
6.7 Parameter trends for the EMC model

The non-linear stress-shear strain response in the EMC model is primarily controlled by the \( \beta \) parameter. The \( \beta \) values determined using numerical optimisation for each of the triaxial tests are plotted against the minor principal effective stress in Figure 6.10. As noted above, \( \beta \) is a measure of the plastic shear strain accumulated when 50% of the peak friction angle is mobilized for isotropically consolidated triaxial tests, or when Equation (6.11) is satisfied for anisotropically consolidated tests. Despite the \( \beta \) values at 40kPa of radial stress, there is a clear trend for \( \beta \) to reduce with increasing density and also a tendency for \( \beta \) to reduce with increasing stress level, although not all data points fit this trend precisely.

![Figure 6.10 Shear hardening parameter](image)

Figure 6.10 Shear hardening parameter
Chapter 6

The mobilised friction angle at which constant volume shearing occurs (\( \phi_{cv} \)), based on matching data in Figure 6.9, is plotted in Figure 6.11. The values of \( \phi_{cv} \) varies between 27.2 and 29.8 degrees. Lower stress levels and higher densities lead to slightly higher \( \phi_{cv} \) values. The volumetric response in the EMC model is governed by the difference between \( \phi_p \) and \( \phi_{cv} \). That is, the dilatancy is controlled by \( \psi = \phi_p - \phi_{cv} \). The fitted values for each test are plotted against stress in Figure 6.12. As expected, this value increases for higher densities and lower stress levels.

Figure 6.11 The variation of \( \phi_{cv} \) with stress and density
Laboratory pressuremeter

The model calibration process described above has identified density and stress level dependent parameters for the EMC model. In this section of the paper, the ability of the EMC model to extrapolate from triaxial stress paths to more complex boundary value problems is examined by conducting a back analysis of a 20mm diameter and 20mm long laboratory scale pressuremeter embedded in the same sand using the same reconstitution procedure.

The pressuremeter was located at the centre of a 393mm diameter, 400mm high steel pressure chamber (see Figure 6.13). A full description of the miniature pressuremeter device is given by Johnston et al. (2013). Lim and Lehane (2014) describe the pressure chamber and associated operational details.
The testing procedure involves fixing the pressuremeter in the centre of the chamber on an aluminium rod. Sand is then air pluviated into the chamber and vibrated to achieve a particular target relative density.

A 40mm thick stress plate is placed on top of the sand and vertical consolidation stress $\sigma_v'$ is applied via the plate to the sand. The walls of the chamber are lined with a Teflon coating to minimise friction between the sand and the wall and achieve $K_0$ consolidation conditions. Based on the observed lift-off pressures, $K_0$ is estimated at approximately 0.5 for all tests. The vertical stress was applied to the samples for a period of at least one week prior to pressuremeter testing to ensure that creep deformations within the sand have diminished to negligible rates.

Two tests were back analysed; one with a vertical consolidation stress of 50kPa and a relative density of 60% and another with a vertical consolidation stress of 100kPa and a relative density of 70%. The density and stress level dependent Young’s modulus (at an axial strain of 0.00075), peak friction angle and $\phi_{cv}$ parameter were interpolated from values used in the triaxial tests. The sets of parameters used for two back analyses cases are given in Table 6.4. A small attraction value ($a$) of 1kPa was used to enhance numerical stability.

Figure 6.13 Pressuremeter tests (a) the device; (b) air pluviation of sand; (c) loading set-up and actual testing
Table 6.4 EMC parameters used to back analyse pressuremeter tests

<table>
<thead>
<tr>
<th>EMC parameters</th>
<th>Values adopted pressuremeter test with $\sigma'_v = 50\text{kPa}$ and $D_r = 0.6$</th>
<th>Values adopted pressuremeter test with $\sigma'_v = 100\text{kPa}$ and $D_r = 0.7$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ [MPa]</td>
<td>23</td>
<td>48</td>
</tr>
<tr>
<td>$\mu$ [-]</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>$\phi_p$ [degree]</td>
<td>41.0</td>
<td>40.0</td>
</tr>
<tr>
<td>$\phi_cv$ [degree]</td>
<td>29.5</td>
<td>28.0</td>
</tr>
<tr>
<td>$a$ [kPa]</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.0018</td>
<td>0.0013</td>
</tr>
</tbody>
</table>

The EMC model was implemented in a FORTRAN subroutine (UMAT) in the Abaqus finite element software package (SIMULIA, 2014). An explicit forward Euler stress integration scheme was used with adaptive sub-stepping and error control, as detailed in Sloan (1987) and Abbo (1997).

An axisymmetric finite element model, presented in Figure 6.14, was created to represent the laboratory scale pressuremeter. The boundary conditions are as indicated on the figure and an initial $K_0$ value of 0.5 was adopted for the pluviated sand. The initial mobilised friction angle was taken to be 1% greater than the actual mobilised friction angle at the initial stress state, resulting in very small elastic range. The computed cavity strain was calculated by dividing the radial displacement at the node on the axis of symmetry by the initial radius of the pressuremeter (10mm).
As the EMC model was calibrated using triaxial data, assumptions must be made regarding the form of the yield surface and plastic potential in the deviatoric plane. The Mohr-Coulomb failure surface is well accepted and widely used for geo-materials, and was therefore adopted in this analysis. To improve stability and numerical convergence, a C2 continuous approximation of the Mohr-Coulomb surface, as described by Abbo et al. (2011), was implemented (i.e. a form that has a continuous first and second derivative).

There is less certainty regarding the shape of the plastic potential in the deviatoric plane. A number of researchers have investigated this, but these studies have focused mainly on predicting collapse loads (Grammatikopoulou et al., 2007; Lagioia & Panteghini, 2014; Potts & Gens, 1984). In this study, an investigation is conducted into the shape of the plastic potential in the deviatoric plane and its effect on deformation. The two options consider are illustrated in Figure 6.15. The first option adopted the Mohr-Coulomb hexagonal shape in the deviatoric plane, which was achieved by adopting the function $G(\theta, \phi_v)$ in Equation (6.6) as follows:

Figure 6.14 Pressuremeter mesh and relevant displacement contour
Evaluating the ability of the EMC model to extrapolate from triaxial stress paths…

\[ G_1(\theta, \phi_{cv}) = \left( \cos \theta - \frac{\sin \theta \sin \phi_{cv}}{\sqrt{3}} \right) \]  

(6.14)

The second plastic potential examined involves fixing the value of Lode’s angle (\( \theta \)) in Equation (6.14) to the triaxial compression value of \( \pi/6 \) to give the following expression,

\[ G_2(\theta, \phi_{cv}) = \left( \frac{\sqrt{3}}{2} - \frac{\sin \phi_{cv}}{2\sqrt{3}} \right) \]  

(6.15)

This results in a circular plastic potential in the deviatoric plane, as shown in Figure 6.15.

![Figure 6.15 Shape of the plastic potential in the deviatoric plane](image)

Using parameters for the test at \( \sigma'_v = 100kPa \) in Table 6.4, the impact that Equations (6.14) and (6.15) have on the mobilised stress dilatancy response is illustrated in Figure 6.16 for various stress paths, where the dilatancy \( d \) is the ratio of the incremental plastic volumetric and shear strains from Equation (6.6) (i.e. \( d = \delta\varepsilon_p^v/\delta\varepsilon^s \)). Based on the
convention used, negative values of $d$ represent plastic contraction, while positive values represent plastic dilation. It can be seen that plastic potentials $G_1$ and $G_2$ give an identical response for a triaxial compression stress paths ($\theta = \pi/6$), showing a contractile response for $\phi_y < 28^\circ$, constant volume shearing for $\phi_y = 28^\circ$ degrees and dilation for $\phi_y > 28^\circ$. For both triaxial extension ($\theta = -\pi/6$) and simple shear ($\theta = 0$), $G_1$ also predicts shearing at constant volume for $\phi_y = 28^\circ$, and contracts below and dilates above this value, by a lower magnitude than in triaxial compression. For plastic potential $G_2$, very similar levels of contraction occur at low mobilised friction angles for the three stress paths. However, the $G_2$ response remains more contractile and reaches constant volume shearing at mobilised friction angles of around $38^\circ$ and $40.5^\circ$ for simple shear and triaxial extension stress paths, respectively.

![Figure 6.16 Dilatancy ($d$) plotted as function of mobilised friction angle ($\phi_y$) for plastic potentials ($G_1$ and $G_2$) with different Lode’s angles for $\phi_{cv} = 28$ degrees](image)
Evaluating the ability of the EMC model to extrapolate from triaxial stress paths...

Figure 6.17 compares pressuremeter data with the calibrated EMC model for the pressuremeter test conducted with a vertical pressure of 50kPa. Simulations were conducted using the parameters in Table 6.4 for the Mohr-Coulomb ($G_1$) and circular forms ($G_2$) of the plastic potential in the deviatoric plane. It can be seen that the $G_1$ form of the EMC model provides a reasonable match to the data at low strain levels, whereas the $G_2$ form tends to underpredict the stiffness. However, at strains of around 2% and greater, the $G_1$ form of the plastic potential overpredicts the stiffness, whereas the $G_2$ form provides a good match to the data.

Figure 6.17 Comparison of pressuremeter model with test 50kPa vertical load, $D_r=60\%$
Figure 6.18 compares the EMC model response with the pressuremeter test with a vertical consolidation stress of 100kPa and a relative density of 70%. Both the $G_1$ and $G_2$ forms of the plastic potential provide a good match to the data up to around 0.5% cavity strain. Beyond this, the $G_1$ form over predicts the stiffness, whereas the $G_2$ form continue to provide a very good match to the data.

Figure 6.18 Comparison of pressuremeter model with test 100kPa vertical load, $D_r=70\%$
The fact that the Mohr-Coulomb form of the plastic potential over predicted the stiffness at higher strain levels is likely because the EMC model is unable to capture post peak softening, which was observed in the triaxial compression tests at shear strains ($\varepsilon_j$) of above around 6-8\% (see Figure 6.9). Figure 6.19 shows contours of $\varepsilon_j$ for the pressuremeter test with a surcharge load of 50kPa and a cavity pressure of 600kPa. It can be seen that peak shear strain values near the edge of the pressuremeter approach 20\%, and a significant area exceeds 6-8\% shear strain. In these regions, the EMC model would be expected to overestimate the dilatancy and this may have contributed to the over stiff response at higher cavity strains.

Figure 6.19 Shear strain contours ($\varepsilon_j$) for the simulation of the pressuremeter test with 50kPa surcharge with a cavity pressure of 600kPa
6.9 Conclusions

In this paper, a procedure for calibrating the EMC model using data from eight anisotropically consolidated drained triaxial compression tests was illustrated. Tests were conducted at two different densities and four different consolidation stresses. The model was found to provide a good match to the triaxial data, but was unable to match the post peak softening or the reduction in dilatancy rates observed at shear strains of above around 6-8%.

Using parameters derived from the triaxial calibration process, the model was then applied to simulate the response of a miniature pressuremeter in the same material using the same reconstitution procedure, thereby examining the ability of the EMC model to extrapolate from simple triaxial stress paths to a more complex boundary value problem. The EMC model was implemented in general stress space using two different plastic potentials in the deviatoric plane. Assumptions regarding the form of the plastic potential in the deviatoric plane were shown to influence EMC model’s response due to the highly confined nature of the boundary conditions for the pressuremeter.

For both pressuremeter simulations, it was found that the $G_1$ (Mohr-Coulomb hexagonal) form of the plastic potential gave a reasonable match to the data at low strain levels, but tended to over predict the stiffness at higher strain levels. The $G_2$ (circular) form of the plastic potential, which gives an identical stress-dilation response in triaxial compression but a less dilatant response in other stress paths, gave a much better match to the data at higher strain levels. It is possible that the over stiff response computed using the model with the $G_1$ (Mohr-Coulomb hexagonal) is a result of the models inability to capture post peak soften, as shear strains were shown significantly 6-8%. 
Chapter 7 Conclusions

In this thesis, data from an extensive programme of testing on reconstituted UWA fine silica sand are presented. The testing programme included deep footing/plate load tests and miniature pressuremeter tests in a pressure chamber, as well as Cone Penetration Tests in the same chamber to provide in-situ characterisation of the sand. A series of anisotropically consolidated drained triaxial compression tests on the UWA sand reconstituted to range of densities and consolidated to a range of stress levels was also conducted to assist interpretation of the deep plate tests and the miniature in-situ tests.

The experimental programme was designed to test the ability of the small strain hardening soil model (HSS), a popular constitutive model, to extrapolate from triaxial stress space to predict the response in more complex boundary value problems in the pressure chamber, where uncertainties relating to differences between the state and fabric of the sand in chamber experiments and those in triaxial samples did not exist. The much simpler Extended Mohr-Coulomb (EMC) soil model was also assessed. Studies such as this are relatively rare in literature, but are a fundamentally important way of examining the predictive capability of soil constitutive models as well as validating procedures for calibrating model parameters. Empirical relationships between CPT resistances ($q_c$), pressuremeter response and deep plate load-displacement responses were also explored.

7.1 Performance of HSS and EMC models

In Chapters 2, 3 and 4 a methodology was developed for calibrating the Hardening Soil Small (HSS) using anisotropically consolidated drained triaxial compression tests with a range of initial stresses and densities on reconstituted UWA fine silica sand. The ability of the model to predict pressuremeter and plate load response in the same material was explored.

One particular challenge in calibrating the HSS model using anisotropically consolidated triaxial data is that a number of HSS model parameters are defined with specific reference to isotropically consolidated triaxial data. It was therefore not possible to identify parameters directly from the measured anisotropically consolidated data. To overcome this, model parameters had to be adjusted in a systematic way and the model response compared with the triaxial test data. An algorithm was used compare the quality of the
match between model and test data (see Figure 7.1). This removed a significant element of subjectively in assessing the quality of the fit.

Figure 7.1 Comparison of the measured and calculated pressure-cavity strain relationship measured in pressuremeter tests (vertical stress =100kPa)

Several other limitations that result from the way the HSS small model is formulated were encountered. These were identified as follows:

- The input small strain (elastic) stiffness to the HSS model needs to be modified from the expected in-situ value to compensate for the model’s underestimation of small strain stiffness where the initial stress state is anisotropic. A full simulation of an anisotropically consolidated triaxial test needs to be undertaken to ensure appropriate HSS parameters are inferred.
• The HSS model uses a single exponent, $m$, to describe the stress level dependence of the very small strain (elastic) stiffness ($G_0$) and the secant stiffness at 50% mobilised strength ($E_{50}$). Experimental observations show that these exponents differ. Moreover, the HSS model assumes that stiffness varies only with the minor principal effective stress whereas previous research has indicated that the major principal effective stress is more influential.

• The observed stress level dependence of a parameter defining the rate of shear stiffness degradation with strain ($\gamma_{0.7}$) is not accounted for by the HSS model.

• It was difficult to achieve a high quality match between the HSS response the axial strain- volumetric strain data. This is because the model tended to over predict the amount of contraction prior to the onset of dilation.

Some of these limitations could be addressed with the inclusion of additional parameters. Despite these limitations, the HSS model was able to provide reasonable back analysis of pressuremeter and footing tests of denser sand, but provided relatively poor matches to pressuremeter and footing/plate load tests for the looser sand samples (see Figure 7.1 and Figure 7.2).

The EMC model, described in Chapter 6, is a much simpler soil model with 6 input parameters. The process of fitting the model to the triaxial test data was more straightforward than it was for the HSS model, involving a simple two stage single variable optimisation procedure. Compared to the HSS model, the EMC model gave a similar high quality match to the stress-strain triaxial data. However, the EMC model gave a better match to the triaxial volumetric response than the HSS model up to around 6-8% axial strain, whereas the test data exhibit a clear slowing in dilation, possibly towards some critical state and the EMC model does not capture this. The HSS model generally over predicted the initial contraction. For example, Figure 7.3 shows the response of HSS and EMC match to triaxial #4, with $\sigma' = 40kPa$ and $D_r = 70\%$. Both models provide a good match to the stress strain data. The EMC model clearly provides a better match to the volumetric response at axial strains less than about 7\%, while the HSS model over predicts the contraction. However, the HSS prediction eventually matches the volumetric strain at an axial strain of around 10\%, while the EMC model over predicts the dilation.
Figure 7.2  Comparison of the measured and calculated vertical stress-normalised settlement relationship measured in vertical loads tests on: (a) 75mm diameter footings ($\sigma'_{v}=100$ kPa), (b) 25mm diameter footings ($\sigma'_{v}=200$ kPa)
Figure 7.3 Comparison of the HSS model and EMC model fit to triaxial test #4
($\sigma'_v = 80$ kPa and $D_r = 70\%$)
The EMC model was implemented in general stress space using two forms of the plastic potential. One form adopted a Mohr-Coulomb hexagonal shape, while the other adopted a circular shape. Both forms give an identical response in triaxial stress space. However, in general stress space, the circular form is more contractile/less dilatant and was found to provide a better match to the pressuremeter test data. For example, Figure 7.4 compares the EMC model’s response with both forms of the plastic potential with the pressuremeter test with a vertical consolidation stress of 100kPa and a relative density of 70%. The HSS response is also shown in Figure 7.4. It can be seen that the more contract circular form of the EMC provides a very similar response to the HSS model. Given the more contractile model produced better match to the measured data, it is likely the tendency of the HSS model to over predict the contraction for the triaxial test data had a positive influence on its ability to match the pressuremeter test data.

Figure 7.4 Comparison between HSS model and EMC model for pressuremeter test
($\sigma'_{v} = 100$kPa and $D_r = 70\%$)
7.2 Empirical approach

Empirical relationships between the response to vertical load of deep footings on reconstituted sand and in-situ and laboratory tests on the same sand were also explored. It was found that:

- A simple non-linear relationship involving the CPT $q_c$ value and relative density captures the responses observed in the deep plate tests in UWA and Ticino sand.

- Although the CPT $q_c$ value provides a useful means of normalising load-displacement data on reconstituted sands, the vertical stiffness of footings on natural (aged) sands is between 2 and 5 times greater than indicated by reconstituted sand with the same CPT $q_c$ value. Enhancement in footing stiffness due to ageing effects is evidently not captured by increases in $q_c$ values with ageing.

- A comparison of measured bearing stress-settlement ratios for deep plates with spherical cavity expansion curves provides general support for the hypothesis made by a number of workers that loading of a deep foundation is analogous to the expansion of a spherical cavity (see Figure 7.5).

![Figure 7.5 Predicting footing settlement using pressuremeter data for 75mm diameter plates ($\sigma_{\sigma} = 100kPa$ and $D_r \approx 60\%$)](image-url)
7.3 Recommendations and future studies

- A comparison of the testing programme with field scale experiments highlighted the very strong effects of ageing. These effects need to be examined as the behaviour of reconstituted sand in the laboratory is clearly quite different to that of aged sands. Further in-situ pressuremeter tests and plate tests would assist the development of prediction methods.

- It is recommended that similar testing is conducted on a different type of reconstituted sand to investigate the effects of particle size, angularity and crushability as well as sand grading on the models and their predictions of the pressuremeter and footing stress-strain responses.

- The HSS model can be improved by introducing two exponents of stress level dependence—one for very small strain and another for a larger strain. A stress level dependence function for the reference strain, $\gamma_{0.7}$, is also required.
Appendix A  Soil properties basic tests

A.1 Introduction

This appendix presents laboratory tests on the University of Western Australia fine silica sand (UWA-FSS), commonly used for geotechnical centrifuge testing at UWA. The tests include partial size distribution, soil specific gravity, maximum and minimum density.

A.2 Index properties

The partial size distribution as shown in Figure 3.1. The mean effective particle size of the sand ($D_{50}$) is 0.29mm, which is higher than the 0.18mm report by O’Loughlin and Lehane (2003) or 0.2mm reported by Lehane et al. (2005) (see Table A.1). The soil is classified as a uniform soil with a uniformity coefficient (UC) of 2.1 (see Section A.3). Similar values of 2 and 2.1 were found by Lehane and White (2005), and Cheong (2002), respectively.

The Specific Gravity of the sand was found as $G_s = 2.676$g/cm$^3$, following the Australian Standard AS 1289.3.5.1 (1995) (see Section A.4).

The maximum and minimum void ratio of 0.745 and 0.455, respectively were found using the Australian Standard AS 1289.5.5.1 (1998). This is consistent with other researchers (such as Teh et al., 2008; Lee et al., 2013). However, other researchers present a smaller range of void ratio; for example, O’Loughlin and Lehane (2003) 0.76 and 0.49.

Toyoura sand (TOS) in Japan has very similar characteristics to UWA-FSS. TOS has a $G_s = 2.65$g/cm$^3$, a $D_{50}$ of 0.22mm and its minimum and maximum unit weight are 13kN/m$^3$ to 16kN/m$^3$ (Fioravante, 2002). In comparison, the minimum and maximum unit weight of UWA fine silica sand is between 15kN/m$^3$ to 18kN/m$^3$.

Fontainebleau sand in France has very similar characteristics to UWA-FSS with $G_s = 2.65$g/cm$^3$ and its minimum and maximum void ratio are 0.523 and 0.863, respectively (Thorel et al., 2007). This sand has very similar passing size distribution to UWA sand. The $D_{50}$ of Fontainebleau sand is 0.22mm, $D_{60} = 0.23$mm and $D_{10} = 0.14$mm, which gives the uniformity coefficient of 1.7.
HST95 silica sand has similar soil properties to UWA-FSS with $G_s = 2.63 \text{g/cm}^3$ and its minimum and maximum void ratio are 0.467 and 0.769, respectively; as presented by Al-Defae et al. (2013) and Lauder (2010) as shown in Table A.1.

### Table A.1 Soil Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>UWA-FSS</th>
<th>TOS</th>
<th>FBS</th>
<th>ICPMG</th>
<th>HST95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity, $G_s$</td>
<td></td>
<td>2.68</td>
<td>2.65</td>
<td>2.65</td>
<td>2.65</td>
<td>2.63</td>
</tr>
<tr>
<td>Particle size, $D_{60}$</td>
<td>μm</td>
<td>329</td>
<td>230</td>
<td>208</td>
<td>140</td>
<td></td>
</tr>
<tr>
<td>Average particle size, $D_{50}$</td>
<td>μm</td>
<td>287</td>
<td>220</td>
<td>220</td>
<td>190</td>
<td></td>
</tr>
<tr>
<td>Particle size, $D_{30}$</td>
<td>μm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>120</td>
</tr>
<tr>
<td>Particle size, $D_{20}$</td>
<td>μm</td>
<td>186</td>
<td></td>
<td></td>
<td></td>
<td>135</td>
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<tr>
<td>Particle size, $D_{10}$</td>
<td>μm</td>
<td>154</td>
<td></td>
<td></td>
<td></td>
<td>99</td>
</tr>
<tr>
<td>Uniformity Coefficient, UC</td>
<td></td>
<td>2.1</td>
<td>1.7</td>
<td>2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minimum unit weight, $\gamma_{\text{min}}$</td>
<td>kN/m$^3$</td>
<td>15</td>
<td>13</td>
<td>14</td>
<td>15</td>
<td>14.59</td>
</tr>
<tr>
<td>Maximum unit weight, $\gamma_{\text{max}}$</td>
<td>kN/m$^3$</td>
<td>18</td>
<td>16</td>
<td>17</td>
<td>18</td>
<td>17.58</td>
</tr>
<tr>
<td>Minimum void ratio, $e_{\text{min}}$</td>
<td></td>
<td>0.455</td>
<td>0.523</td>
<td>0.449</td>
<td>0.467</td>
<td></td>
</tr>
<tr>
<td>Maximum void ratio, $e_{\text{max}}$</td>
<td></td>
<td>0.745</td>
<td>0.863</td>
<td>0.747</td>
<td>0.769</td>
<td></td>
</tr>
<tr>
<td>Min dry density, $\rho_{\text{min}}$</td>
<td>kg/m$^3$</td>
<td>1,533.2</td>
<td></td>
<td></td>
<td></td>
<td>1,516.7</td>
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<tr>
<td>Max dry density, $\rho_{\text{max}}$</td>
<td>kg/m$^3$</td>
<td>1,838.8</td>
<td></td>
<td></td>
<td></td>
<td>1,829.6</td>
</tr>
</tbody>
</table>
A.3 Particle size distribution

The test was conducted on the 16\textsuperscript{th} of June 2015 following the test procedure described in Australian Standard AS 1141.11.1 (2009). The unwashed sample was oven dried 24 hours at 105\degree C prior to testing. Table A.2 and Figure 3.1 show the percentage of particles passing each sieving.

It was found that $D_{50} = 0.287\text{mm}$, $D_{10} = 0.154\text{mm}$ and $D_{60} = 0.329\text{mm}$, giving a uniformity coefficient (UC) of 2.1.

<table>
<thead>
<tr>
<th>Sieve aperture size (mm)</th>
<th>Percent of total passing (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.750</td>
<td>100</td>
</tr>
<tr>
<td>2.360</td>
<td>100</td>
</tr>
<tr>
<td>1.180</td>
<td>100</td>
</tr>
<tr>
<td>0.600</td>
<td>99.90</td>
</tr>
<tr>
<td>0.425</td>
<td>86.10</td>
</tr>
<tr>
<td>0.300</td>
<td>51.35</td>
</tr>
<tr>
<td>0.212</td>
<td>26.70</td>
</tr>
<tr>
<td>0.150</td>
<td>8.80</td>
</tr>
<tr>
<td>0.106</td>
<td>0.85</td>
</tr>
<tr>
<td>0.075</td>
<td>0.10</td>
</tr>
</tbody>
</table>

A.4 Specific gravity determination

The Specific Gravity test was conducted on fine silica sand on the 17\textsuperscript{th} of October 2014 following the test procedure described in Australian Standard AS 1289.3.5.1 (1995).

This section summarizes the test method conducted on UWA fine silica sand. First, the pycnometer was dried and the mass of the pycnometer and stopper determined ($m_1$) and repeated for three samples. The soil was oven dried at 110\degree C for 24 hours, allowed to cool, and poured into the pycnometer using the funnel. The mass of pycnometer with stopper and soil was recorded ($m_2$). Water was added to fill about two-thirds of the pycnometer and the soil particles soaked for about 24 hours. A partial vacuum was then applied to the three pycnometers to remove the dissolved air from the water (de-aired),
which took around 45 minutes. The pycnometer was filled with de-aired water to maintain the water level, the stopper inserted and the outside of the pycnometer dried. The mass of the pycnometer, stopper, soil and water were determined \( (m_3) \). The temperature of the contents was measured, allowing up to 1ºC difference. The pycnometer was cleaned and filled with water, which was then left to reach the equilibrium temperature within ± 2ºC. The stopper was installed, the outside of the pycnometer dried, and the mass of pycnometer, stopper and water determined.

Specific gravity was calculated using the following formula:

\[
G_s = \frac{m_2 - m_3}{(m_4 - m_1) - (m_3 - m_2)} \rho_w
\]

where the value of the water density \( \rho_w \) depends on the temperature (see Australian Standard AS 1289.3.5.1 (1995) for more details).

<table>
<thead>
<tr>
<th>Table A.3 Fine Silica Sand ( G_s ) Test Result</th>
</tr>
</thead>
<tbody>
<tr>
<td>( m_1 ) (g)</td>
</tr>
<tr>
<td>( m_2 ) (g)</td>
</tr>
<tr>
<td>( m_3 ) (g)</td>
</tr>
<tr>
<td>( m_4 ) (g)</td>
</tr>
<tr>
<td>Temperature (ºC)</td>
</tr>
<tr>
<td>( \rho_w ) (g/cm³)</td>
</tr>
<tr>
<td>( G_s ) (g/cm³)</td>
</tr>
</tbody>
</table>

Table A.3 shows the specific gravity results. The average of specific gravity \( (G_s) \) is 2.675715g/cm³.
A.5 Minimum density

Tests were conducted on fine silica sand at the UWA soil lab to define the minimum density. The test procedure followed Australian Standard AS 1289.5.5.1 (1998).

A.5.1 Test procedure

A summary of the test procedure is presented in this section. The soil was oven dried at 110ºC for 24 hours then the mass and volume of the mold measured. The soil was allowed to cool, then poured into the mold as loosely as possible; a funnel was used to allow soil to flow in a steady stream with maintaining of a free fall of not more than 20mm. In order to form a uniform thickness of soil layer, the pouring funnel was moved in a spiral motion from outside toward the centre of the mold. A straight edge was used toward completion of filling up the soil, to level off the sand with the prescribed height of the mold. Finally, the mass of mold and soil was determined and the soil mass calculated.

A.5.2 Minimum density result for fine silica sand

The test was conducted on a mold with average diameter $D=152.167$ mm and height $H=155.225$ mm; thus the mold volume was $V=2,822.868$ cm$^3$. The empty weight of the mold was $M_0 = 3.41$ kg.

Four tests were conducted on the 20$^{th}$ of October 2014 on fine silica sand. The test results are shown in Table A.4.

<table>
<thead>
<tr>
<th>Trial</th>
<th>$M_{\text{total}}$ (g)</th>
<th>$M_s$ (g)</th>
<th>$\rho$ (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7,756</td>
<td>4,346</td>
<td>1.540</td>
</tr>
<tr>
<td>2</td>
<td>7,752</td>
<td>4,342</td>
<td>1.538</td>
</tr>
<tr>
<td>3</td>
<td>7,738</td>
<td>4,328</td>
<td>1.533</td>
</tr>
<tr>
<td>4</td>
<td>7,744</td>
<td>4,334</td>
<td>1.535</td>
</tr>
</tbody>
</table>

The minimum density was $\rho_{\text{min}} = 1.533$ g/cm$^3$. Thus the maximum void ratio was

$$e_{\text{max}} = \frac{\rho_s}{\rho_{\text{min}}} - 1 = \frac{2.6757}{1.533} - 1 = 0.7454$$
A.6 Maximum density

Tests were conducted on fine silica sand at the UWA soil lab to define the maximum density. The test procedure followed Australian Standard AS 1289.5.5.1 (1998).

A.6.1 Test procedure

First, the soil was dried in an oven at 110°C for 24 hours. Second, the mass and volume of the mold was determined. The soil was set to cool, and a funnel used to pour the soil into the mold to obtain constant layers. A hammer was used after adding each layer to vibrate the soil in the mold to minimize the void between soil particles. After using the layering technique for sampling, about 10mm clearance was allowed from the top of the mold; then the surcharge base plate was installed in place. The mold was fixed on the vibrating table by tightening the bolts on the table and tightening the guide sleeves on top of the mold. The surcharge weight was guided to be vertical on top of the plate (surcharge weight was 23.90kg). The vibration table was operated for at least 12 minutes, ensuring the operating frequency was 50Hz. The dial indicator gauge was used to determine the height of the soil, and the volume of the soil sample was calculated. Finally, the mold and soil were weighed to determine the soil mass.

A.6.2 Maximum density result for fine silica sand

The test was conducted on the same mold used to determine the minimum density. Four tests were conducted on the 16th of October 2014 on fine silica sand. The test results are shown in Table A.5.

<table>
<thead>
<tr>
<th>Trial</th>
<th>$M_{\text{total}}$ (g)</th>
<th>$M_S$ (g)</th>
<th>$R_d$ (mm)</th>
<th>$H_s$ (mm)</th>
<th>$V_s$ (cm$^3$)</th>
<th>$\rho$ (g/cm$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8,308</td>
<td>4,898</td>
<td>6.998</td>
<td>149.773</td>
<td>2,723.7</td>
<td>1.798</td>
</tr>
<tr>
<td>2</td>
<td>8,244</td>
<td>4,834</td>
<td>1.943</td>
<td>144.718</td>
<td>2,631.8</td>
<td>1.837</td>
</tr>
<tr>
<td>3</td>
<td>8,240</td>
<td>4,830</td>
<td>2.870</td>
<td>145.645</td>
<td>2,648.7</td>
<td>1.824</td>
</tr>
<tr>
<td>4</td>
<td>8,170</td>
<td>4,760</td>
<td>-0.432</td>
<td>142.343</td>
<td>2,588.6</td>
<td>1.839</td>
</tr>
</tbody>
</table>

The maximum density was $\rho_{\text{max}} = 1.839$ g/cm$^3$ . Thus the minimum void ratio was

$$e_{\text{min}} = \frac{\rho_S}{\rho_{\text{max}}} - 1 = \frac{2.6757}{1.839} - 1 = 0.4549$$
Appendix B  Triaxial tests

B.1 Triaxial tests procedure

This section presents the procedure for conducting drained triaxial compression tests, with Bender element and local strain gauges, on UWA-FSS. Local strain gauges work in an oily environment; thus oil was used to fill in the triaxial cell. Prior to commencing the load cell, strain gauges (see Figure B.1) and Bender elements were calibrated using wave detector (see Figure B.2).

Figure B.1 Micromovement device to calibrate local strain gauge
B.1.1 Sample preparation procedure

The ten triaxial tests followed the procedure outline below.

- Place the O-stone and filter paper on top of the Bender element (see Figure B.3a).
- Set up the base (Bender element transmitter) on a holder.
- Apply grease all around contacted area between the Bender and the membrane. At the same time, glue the strain gauges inside the membrane and make sure all four are in the correct position. The distance between the top and the bottom of the local gauge’s assistance on the inside of the membrane was fixed carefully at 110mm.
- Install the membrane on the base (Bender element transmitter).
- Place two O-rings on the assigned position (see Figure B.3b).
- Place the mold on the Bender element base and firmly tighten the mold with the mold belt.
• Use the air-vacuum pump to apply suction air pressure from outside the mold to hold it on the membrane. Tie up the pressure line into the mold to allow the air-vacuum to suck the air around the membrane (see Figure B.3c).

• Measure the diameter and height of the mold with the membrane; take at least three readings in different directions. These measurements will be used to calculate the sample volume and the initial height of the sample.

• Pour sand into a vent and use the vent to fill in the mold using a layering technique. Use 10 spoons or one scoop of sand for each layer; hit the soil sample with a hammer and shake the mold when necessary to obtain the desired density (see Figure B.3d). In general, for tests with a relative density of 40% (loose sand), apply five hammer hits for each layer. For tests with relative densities of 70% (dense sand), apply 50 hammer hits. For maximum density of sand, the soil poured by the layering technique and vibrating table was used to achieve the desirable density.

• When the mold is full, place the top (Bender element receiver) on the sample to close it. Seal the top (receiver Bender) with two O-rings (see Figure B.3e). At this stage, make sure the transmitter and the receiver Benders are pointed in the same direction as marked when calibration was conducted. Marking the transmitter and receiver during calibration enhances the possibility of obtaining a clean wave during the test.

B.1.2 Test preparation procedure

The ten triaxial tests presented in this thesis used the following preparation procedure:

• Connect the Bender and the soil sample inside the triaxial device.

• Connect the drainage line from the top cap of the sample. Use this line at first to apply suction air pressure using the air vacuum. Apply at least 10kPa suction air pressure to prevent the soil sample from collapsing.

• Plug the Bender receiver wire line inside the triaxial cell chamber in the right position. Firmly tie the plug.
Appendix B

- Remove the mold, with an Allen key.

- Glue the four strain gauges’ assistance outside the membrane with super-fast glue; allow at least one minute for the glue to set and then attach the strain gauges (see Figure B.3f). Connect the strain gauges into the sample as demonstrated in Figure B.3f.

- Close and tie the triaxial chamber.

- Fill the chamber with oil from the fill and drainage line.

- Apply at least 10kPa cell pressure to hold the sample.

- Disconnect the air pressure line from the base, and connect the backpressure line to the base.

- Flush the sample by water from the top and leave the base open as this is a drained test procedure. During this process, monitor the water coming up from the baseline. Allow at least 40 minutes for the water to flush the sample.

- Saturate the sample with water.

- Raise the cell pressure to 510kPa and backpressure to 500kPa; allow a 10kPa difference from the sample to sustain and prevent the soil sample from collapsing. Allow at least one hour for this stage.

- Check the B-value by increasing the cell pressure to 100kPa. The difference between the cell pressure and backpressure should be more than 95%. If it is more than the requirement, proceed with the test; otherwise, increase the cell pressure and allow another hour, then check the B-value again. The triaxial tests conducted are drained tests; thus, the backpressure is obtained from both top and base lines.

- Now, start taking shear wave signals. Start consolidates the soil sample anisotropically. In this study, the ratio of axial stress $(\sigma'_a)$ and radial stress $(\sigma'_r)$ was fixed at $K_0 = 0.5$. Allow soil to consolidate for at least 4 hours.

- When consolidation is achieved, start shearing the sample by increasing the load pressure (deviator stress) from the top of the sample.
Figure B.3 Sample preparation
### Appendix B

#### B.2 Triaxial tests results

A summary of measurement from the ten tests is presented in Table B.1.

---

**Table B.1 Summary of Measured Parameters of Triaxial Tests**

<table>
<thead>
<tr>
<th>Test #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Date</td>
<td>14.08.2016</td>
<td>07.04.2016</td>
<td>13.08.2015</td>
<td>20.08.2015</td>
<td>18.08.2015</td>
<td>11.02.2015</td>
<td>26.08.2015</td>
<td>05.02.2015</td>
<td>17.02.2015</td>
<td>24.08.2015</td>
</tr>
<tr>
<td>Parameters</td>
<td>Unit</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
<td>kPa</td>
</tr>
<tr>
<td>$\sigma'_h$</td>
<td></td>
<td>20.2</td>
<td>20.6</td>
<td>39.3</td>
<td>40.9</td>
<td>102.2</td>
<td>100.6</td>
<td>300.2</td>
<td>301.9</td>
<td>295.8</td>
</tr>
<tr>
<td>$\sigma'_v$</td>
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<td>39.9</td>
<td>40.7</td>
<td>79.2</td>
<td>80.9</td>
<td>200.6</td>
<td>199.6</td>
<td>594.2</td>
<td>595.5</td>
<td>592.6</td>
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<tr>
<td>$D_r$ (initial)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>$D_r$ (at shearing)</td>
<td>%</td>
<td>40</td>
<td>70</td>
<td>40</td>
<td>70</td>
<td>6</td>
<td>40</td>
<td>70</td>
<td>95</td>
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<td>$G_0$</td>
<td>MPa</td>
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<tr>
<td>Void Ratio</td>
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<td></td>
<td></td>
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<tr>
<td>$\epsilon_{\text{initial}}$</td>
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<td>0.6396</td>
<td>0.5483</td>
<td>0.6260</td>
<td>0.5195</td>
<td>0.6077</td>
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<td>0.6432</td>
<td>0.6248</td>
<td>0.5639</td>
</tr>
<tr>
<td>$\epsilon_{\text{at shearing}}$</td>
<td></td>
<td>0.6417</td>
<td>0.5459</td>
<td>0.6225</td>
<td>0.5151</td>
<td>0.5965</td>
<td>0.5543</td>
<td>0.6269</td>
<td>0.6248</td>
<td>0.5542</td>
</tr>
<tr>
<td>Dilation cut off</td>
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<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td>851</td>
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A-12
Triaxial tests

In the following each test is discussed in detail and comparison of the HSS small model response is compared with the measured data using the parameters in Table B.2. The parameters were evaluated by implementing a minimisation algorithm proposed by Doherty et al. (2012).

Table B.2 Summary of HSS Parameters of Triaxial Tests

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<td>9.7</td>
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</table>

A-13
B.2.1 Test 1 results

The effect of shear on the sample can be seen in the photo in Figure B.4. The shear wave recorded at the end of the consolidation stage for three different frequencies is shown in Figure B.5. The shear wave calculation depends on the wave travel time. At the end of consolidation, three values were recorded and the average calculated to obtain the value of the small strain shear moduli. The local strain measurement was used until the axial strain plotted to 0.1%, then external strain measurement was used to generate the plot (see Figure B.6). The local strain measurement implies a Linear Variable Differential Transformer (LVDT) device mounted outside the triaxial chamber to measure the vertical movement of the load cell. The maximum deviator stress this sample could sustain was 66.5kPa, occurring at around 6.4% of axial strain. The peak friction angle was 37.5° which can be seen in the photo in Figure B.4. As demonstrated in Figure B.6 the dilation cut-off was not clearly detected for this test; however, it was approximated at 11% of axial strain. The axial strain for this test was recorded up to around 24% of axial strain. At around the dilation cut-off, a constant volume friction angle was detected at around 35°, which is about 91% of measured peak friction angle. The small strain shear modulus reduction parameter ($\gamma_{0.7}$) was found by creating a line in the small strain linear region. The 70% of maximum shear modulus measured using the Bender element test at the end of consolidation was used to identify the location of $\gamma_{0.7}$. 
Triaxial tests

Figure B.4 Test 1: sample after shearing

Figure B.5 Test 1: shear wave data at the end of consolidation
Figure B.6 Test 1: measured parameters
Figure B.7 Test 1: optimising operational stiffness

Figure B.8 Test 1: measured parameters and corresponding model
Appendix B

B.2.2 Test 2 results

Figure B.9 Test 2: sample after shearing

Figure B.10 Test 2: shear wave data at the end of consolidation
Figure B.11 Test 2: measured parameters
Figure B.12 Test 2: optimising operational stiffness

Figure B.13 Test 2: measured parameters and corresponding model
B.2.3 Test 3 results

Figure B.14 Test 3: sample after shearing

Figure B.15 Test 3: shear wave data at the end of consolidation
Figure B.16 Test 3: measured parameters
Triaxial tests

Figure B.17 Test 3: optimising operational stiffness

Figure B.18 Test 3: measured parameters and corresponding model
Appendix B

B.2.4 Test 4 results

Figure B.19 Test 4: sample after shearing

Figure B.20 Test 4: shear wave data at the end of consolidation
Figure B.21 Test 4: measured parameters
Figure B.22 Test 4: optimising operational stiffness

Figure B.23 Test 4: measured parameters and corresponding model
B.2.5 Test 5 results

Figure B.24 Test 5: sample after shearing

Figure B.25 Test 5: shear wave data at the end of consolidation
Figure B.26 Test 5: measured parameters
Triaxial tests

Figure B.27 Test 5: optimising operational stiffness

Figure B.28 Test 5: measured parameters and corresponding model
B.2.6 Test 6 results

Figure B.29 Test 6: sample after shearing

Figure B.30 Test 6: shear wave data at the end of consolidation
Figure B.31 Test 6: measured parameters
Appendix B

Figure B.32 Test 6: optimising operational stiffness

Figure B.33 Test 6: measured parameters and corresponding model
B.2.7 Test 7 results

Figure B.34 Test 7: sample after shearing

Figure B.35 Test 7: shear wave data at the end of consolidation
Figure B.36 Test 7: measured parameters
Triaxial tests

Figure B.37 Test 7: optimising operational stiffness

Figure B.38 Test 7: measured parameters and corresponding model
B.2.8 Test 8 results

Figure B.39 Test 8: sample after shearing

Figure B.40 Test 8: shear wave data at the end of consolidation
Triaxial tests

Figure B.41 Test 8: measured parameters
Appendix B

**Figure B.42 Test 8: optimising operational stiffness**

![Optimising operational stiffness](image1)

**Figure B.43 Test 8: measured parameters and corresponding model**

![Measured parameters and corresponding model](image2)
B.2.9 Test 9 results

Figure B.44 Test 9: sample after shearing

Figure B.45 Test 9: shear wave data at the end of consolidation
Figure B.46 Test 9: measured parameters
Triaxial tests

Figure B.47 Test 9: optimising operational stiffness

Figure B.48 Test 9: measured parameters and corresponding model
B.2.10 Test 10 results

Figure B.49 Test 10: sample after shearing

Figure B.50 Test 10: shear wave data at the end of consolidation
Figure B.51 Test 10: measured parameters
Figure B.52 Test 10: optimising operational stiffness

Figure B.53 Test 10: measured parameters and corresponding model
Appendix C  Experimental procedures for plate/footing tests

This Appendix describes the testing procedure for the buried plate/footing load tests.

C.1 Testing apparatus

The following describes the equipment used:

- **A 2cm thick steel pressure chamber** with an inner diameter of 390mm and a depth of 400mm (see Figure C.1).

- **Steel wires and clamp**: Steel wires used to guide the rod of plate/footing to ensure it located in the centre of the chamber (see Figure C.1a). As new load sensors had been introduced to the plate/footing rod, the technique used to hold the footing rod and PVC pipe was no longer appropriate in the new case. Instead, holders were introduced to hold the footing rod and to make sure the footing was placed in the centre of the chamber, as shown in Figure C.1b.

![Figure C.1](image_url)

(a) Install footing/plate with PVC sleeve technique; (b) install footing/plate with load sensors on the shaft

- **Internal load cells on footing rod**: Two strain gauges acting as load sensors were affixed to the rod of the plate/footing to measure the load at the top and the bottom of the consistency through the soil layers (see Figure C.2). The thickness of the footing plate was 10mm. The rod dimensions were a 300mm length, a 17.3mm
outer diameter and a 12.4mm inner diameter. The effective cross-sectional area was 114mm² and the Young’s modulus was 200GPa, according to Onesteel (2010). The buckling analysis of the member was designed following Australian Standards AS 4100 (1998). The assumption was made that sway members were free to rotate and fixed to the translation on one side, and the rotation was fixed and the translation was kept free on the other side. The yield strength of the material was $\sigma_y = 150\text{MPa}$ as stated in Handbook of Steel & Tube (2013). The factor of safety for this shaft was taken to be 50% more than the ultimate load that could be applied.

Figure C.2 (a) Welded plate to the shaft; (b) thread shaft with two strain gauges acting as load cells

- **Base load plate (chamber lid):** The chamber lid consisted of a 40kg steel plate with a diameter of 390mm and a thickness of 40mm. A slit of width 30mm from the centre to one edge was introduced to improve the sample preparation method (see Figure C.3).
Experimental procedures for plate/footing tests

- **Loading cylinder**: This was used to apply centralised load over the top of the sample (see Figure C.3).

- **Enerpac**: The Enerpac connected to a hydraulic jack, similar to the one used in pressuremeter tests to apply the vertical load on the sample (see Appendix E). Another Enerpac was used to apply load on the plate/footing (see Figure C.3a).

- **Hydraulic jack**: The jack was used to apply pressure on the sample via a reaction frame. The load applied by the hydraulic system was monitored by a load cell that was located between the hydraulic jack and reaction frame (see Figure C.3b). Another hydraulic jack was used to apply load on the plate/footing (see Figure C.3c).

- **External load cells**: Two load cells was used in the plate/footing test. One load cell used to monitor the load applied via the hydraulic jack and transferred to the reaction frame (see Figure C.3a). Another load cell was used to monitor the load applied on the footing externally (see Figure C.3c).

- **Internal load cells on footing rod**: Two load sensors was affix to the rod of the plate/footing (see Figure C.2b and Figure C.4).

- **LVDTs**: Two Linear Variable Differential Transformers (LVDTs) were used to measure the vertical displacement.

- **Laser sensor**: A laser sensor was used to measure the vertical displacement of the frame of the plate/footing.

- **Marble**: A steel marble was used to connect the footing frame to the rod of the plate/footing. It was used to remove any moment that may generated on the rod (see Figure C.3d).

- **Software**: DigiDAQ software was used to log and save the digital data recorded; as the one used in pressuremeter tests.
Figure C.3 (a) Photo of set-up for footing/plate test, (b) sketch showing set-up for footing tests, (c) zoom-in to show the hydraulic jack and external load cell used to apply load on the plate/footing; (d) zoom-in shows the steel marble.
Figure C.4 Sketch shows the location of footing during testing and local load cells.
C.2 Sample preparation method

Buried plate/footing sample was prepared using the Sand-Rainer technique (same procedure used to prepare the pressuremeter sample as will be discussed in Appendix E). First, the chamber was placed at 10cm above the floor. A Sand-Rainer was used with an automatic hopper to ‘rain’ the sand into the chamber. The sand pours through a slot to produce uniform layer of sand rain over a circular pressure chamber with a diameter of 393mm and height of 400mm as shown in Figure C.4. The distance between the location of the sand layer and the opening slot of the hopper was fixed to a metre and in order to keep this distance constant, it was raised by an increment of 40mm every time the Rainer had deposited 40mm of sand to ensure that the soil sample had a consistent relative density throughout the chamber. The travel speed or velocity rate of the Sand-Rainer was fixed for all tests at 104mm/second. The footing plate was buried at 180mm from the bottom of the pressure chamber (the same distance as the location of the pressuremeter device).

After the sample prepared, the chamber lifted from the ground level to the testing station via a crane. The vertical load then applied. Two different methods were used to apply load on the sample. The first method applied a dead load to reach the desired stress level (see Figure C.5). However, this method had a limited capacity of the initial vertical stress on the sample to a low level. Thus, a second method was introduced in which the hydraulic mechanism was used to apply the load (see Figure C.3). By using this concept, the stress level applied to the soil increased from 37kPa using a dead weight to 200kPa. To ensure the consistency of the surcharge, the vertical load applied by was left for at least two days. Then the footing-loading frame was attached on top of the footing rod. The load can be applied to the footing/plate by adding some weights (Figure C.5) or using the hydraulic system (Figure C.3). The load increment for the hydraulic system was maintained at around 0.2kN with each increment taking effect at intervals of more than 5 minutes. The maximum load was increased from 1.7kN using weights technique to 5.5kN when the hydraulic system applied.
Figure C.5 Applying a vertical load and the testing footing settlement in response to incremental weights
Appendix D  Experimental results for plate/footing tests

This Appendix presents the test results for the buried footing/plate load tests conducted in the pressure chamber using UWA fine silica sand. The test equipment and procedure are described in Appendix C. Table D.1 lists 48 tests conducted in chronological order. The table lists the test conditions, including, the footing diameter \( D \), the vertical consolidation pressure \( \sigma'_v \), relative density \( D_r \), void ratio \( e \), and soil unit weight \( \gamma \). The tests are classified into 33 categories depending on footing diameter, relative density and stress level (see Table D.1). The prefixes in the test numbers indicate whether the test was conducted by applying dead load, tests are prefixed by FT (Footing Test) or the test was conducted by applying hydraulic system, tests are prefixed by FLST (Footing with Load Sensors Test). The data is presented in the following figures. In each figure, a line of best fit was generated using Equation (D.1), where \( c \) and \( m \) are constants, \( s/D \) is in % and \( q \) is in kPa.

\[
q = c(s/D)^m \quad \text{(D.1)}
\]

It should be noted that the initial settlements that occur due to the weight of the loading frame were not measured. Therefore, these settlements were estimated using Equation (D.2) following Mayne (2014) and Mayne and Poulos (1999) methods.

\[
\frac{s}{D} = \left( \frac{\pi}{2} \frac{q}{q_c} (1 - v^2) \right)^2 \quad \text{(D.2)}
\]

Where \( q \) is the stress on footing/plate applied by the weight of the loading frame. The cone tip resistance \( q_c \) for the purpose of predicting initial settlement was obtained following Jamiolkowski et al. (2001) method as expressed in Equation (D.3). This method was found to be the best estimation method as suggested by Lim (2013).

\[
q_c = \sqrt{\sigma'_v} \times e^{(D_r+1.292)/0.268} \quad \text{(D.3)}
\]
### Table D.1 General summary of footing/plate load tests

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<th>$D_t$</th>
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### Appendix C

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#### D.1 Footing tests for loose sand ($\sigma'_v=38\text{kPa}$)

![Figure D.1 Footing test (category F1) for loose sand ($\sigma_v=38\text{kPa}$ and $D=75\text{mm}$)](image.png)

Figure D.1 Footing test (category F1) for loose sand ($\sigma_v=38\text{kPa}$ and $D=75\text{mm}$)
Experimental procedures for plate/footing tests

Figure D.2 Footing test (category F2) for loose sand ($\sigma_v=38\text{kPa}$ and $D=50\text{mm}$)

Figure D.3 Footing test (category F3) for loose sand ($\sigma_v=38\text{kPa}$ and $D=25\text{mm}$)
D.2 Footing tests for medium dense sand ($\sigma_v=38\text{kPa}$)

Figure D.4 Footing test (category F4) for medium dense sand ($\sigma_v=38\text{kPa}$ and $D=75\text{mm}$)

Figure D.5 Footing test (category F5) for medium dense sand ($\sigma_v=38\text{kPa}$ and $D=50\text{mm}$)
Experimental procedures for plate/footing tests

D.3 Footing tests for dense sand ($\sigma'_{v}=38$ kPa)

Figure D.6 Footing test (category F6) for medium dense sand ($\sigma_{v}=38$ kPa and $D=25$ mm)

Figure D.7 Footing test (category F7) for dense sand ($\sigma_{v}=38$ kPa and $D=75$ mm)
Figure D.8 Footing test (category F8) for dense sand (σv=38kPa and D=50mm)

Figure D.9 Footing test (category F9) for dense sand (σv=38kPa and D=50mm)
D.4 Footing tests for loose sand ($\sigma'_v=50\text{kPa}$)

Figure D.10 Footing test (category F10) for loose sand ($\sigma_v=50\text{kPa}$ and $D=75\text{mm}$)

Figure D.11 Footing test (category F11) for loose sand ($\sigma_v=50\text{kPa}$ and $D=50\text{mm}$)
Figure D.12 Footing test (category F12) for loose sand ($\sigma_v=50\text{kPa}$ and $D=25\text{mm}$)

**D.5 Footing tests for medium dense sand ($\sigma'_v=50\text{kPa}$)**

Figure D.13 Footing test (category F13) for medium dense sand ($\sigma_v=50\text{kPa}$ and $D=75\text{mm}$)
Experimental procedures for plate/footing tests

Figure D.14 Footing test (category F14) for medium dense sand ($\sigma_v=50\text{kPa}$ and $D=50\text{mm}$)

Figure D.15 Footing test (category F15) for medium dense sand ($\sigma_v=50\text{kPa}$ and $D=25\text{mm}$)
Appendix C

D.6 Footing tests for dense sand ($\sigma_v$=50kPa)

Figure D.16 Footing test (category F16) for dense sand ($\sigma_v$=50kPa and $D$=75mm)

Figure D.17 Footing test (category F17) for dense sand ($\sigma_v$=50kPa and $D$=50mm)
A - 63

Figure D.18 Footing test (category F18) for dense sand ($\sigma_v=50kPa$ and $D=25mm$)

D.7 Footing tests for loose sand ($\sigma_v'=100kPa$)

Figure D.19 Footing test (category F19) for loose sand ($\sigma_v=100kPa$ and $D=75mm$)
Figure D.20 Footing test (category F20) for loose sand ($\sigma_v=100\text{kPa}$ and $D=50\text{mm}$)

Figure D.21 Footing test (category F21) for loose sand ($\sigma_v=100\text{kPa}$ and $D=25\text{mm}$)
D.8 Footing tests for medium dense sand ($\sigma'_v=100\text{kPa}$)

![Graph showing footing test results](image1)

Figure D.22 Footing test (category F22) for medium dense sand ($\sigma_v=100\text{kPa}$ and $D=75\text{mm}$)

![Graph showing footing test results](image2)

Figure D.23 Footing test (category F23) for medium dense sand ($\sigma_v=100\text{kPa}$ and $D=50\text{mm}$)
Figure D.24 Footing test (category F24) for medium dense sand ($\sigma_v=100\text{kPa}$ and $D=25\text{mm}$)

**D.9 Footing tests for dense sand ($\sigma'_v=100\text{kPa}$)**

Figure D.25 Footing test (category F25) for dense sand ($\sigma_v=100\text{kPa}$ and $D=75\text{mm}$)
Experimental procedures for plate/footing tests

Figure D.26 Footing test (category F26) for dense sand ($\sigma_v=100$ kPa and $D=50$ mm)

Figure D.27 Footing test (category F27) for dense sand ($\sigma_v=100$ kPa and $D=25$ mm)
D.10 Footing tests for different density subject to same stress level ($\sigma'_v=200\text{kPa}$)

Figure D.28 Footing test (category F28) for loose sand ($\sigma_v=200\text{kPa}$ and $D=75\text{mm}$)

Figure D.29 Footing test (category F29) for loose sand ($\sigma_v=200\text{kPa}$ and $D=50\text{mm}$)
Experimental procedures for plate/footing tests

Figure D.30 Footing test (category F30) for loose sand ($\sigma_v=200$kPa and $D=25$mm)

Figure D.31 Footing test (category F31) for medium dense sand ($\sigma_v=200$kPa and $D=50$mm)
Figure D.32 Footing test (category F32) for medium dense sand ($\sigma_v=200\text{kPa}$ and $D=25\text{mm}$)

Figure D.33 Footing test (category F33) for dense sand ($\sigma_v=200\text{kPa}$ and $D=25\text{mm}$)
Appendix E  Experimental procedures for pressuremeter tests

This Appendix describes the test procedure used to conduct miniature pressuremeter testing in the pressure chamber.

E.1 Testing apparatus

The following describes the equipment used:

- A **2cm thick steel pressure chamber** with an inner diameter of 390mm and a depth of 400mm (see Figure E.1 and Figure E.2).

- **Clamp and steel wires**: In order to minimise the soil disturbance around the pressuremeter cell, thin steel wires were used during the sample preparation (see Figure E.1).

- **Base load plate (chamber lid)**: The chamber lid consisted of a 40kg steel plate with a diameter of 390mm and a thickness of 40mm. A slit of width 30mm from the centre to one edge was introduced to improve the sample preparation method (see Figure E.2).

- **Loading cylinder and upper plate**: These items were used to apply centralised over the top of the pressuremeter rod and allow access to cables. The hollow loading cylinder had an outer diameter of 145mm, a height of 280mm, and mass 5kg. The upper load plate had a diameter of 240mm and a mass of 15kg. The loading cylinder had a vertical section cut out in order to allow a stabilising clamp to pass through and remain holding the pressuremeter rod at all time during the test, thereby controlling the vertical and horizontal movement (see Figure E.2).

- **Cylindrical stand**: A cylindrical stand inside the chamber was used to control the vertical location of the device and minimise any disturbance of the sample. The cylindrical stand bolted to the base of the chamber was designed to make sure the device was always located at the centre of the chamber in order to improve the repeatability of tests.
Figure E.1 Pressuremeter and holder

Figure E.2 Typical pressuremeter test setup
• **Pressure controller and data monitoring:** A cylindrical vessel was used to control the air pressure with the assistance of a digital pressure controller (see Figure E.2 and Figure E.3). The air compressor (shown in Figure E.4) was used to supply the cavity pressure to the pressuremeter chamber.

![Digital Pressure Controller](image1.png)

Figure E.3 Digital Pressure Controller

![Air compressor](image2.png)

Figure E.4 Air compressor
Appendix E

- **Software**: DigiDAQ software was used to log and save the digital data recorded (see Figure E.5). The DigiDAQ interfaced with the direct pressure controller (DPC) software developed by UWA.

  ![Figure E.5 Digital Instrument Amplifier (DigDAQ)](image)

- **Laser sensors**: used to measure the vertical Displacement was measured using two laser sensors.

- **Vertical load application**: Vertical load was applied to the soil sample using a Enerpac hydraulic jack (see Figure E.2 and Figure E.6). The hydraulic jack applied pressure on the sample via a reaction frame. The load applied by the hydraulic system was monitored by a load cell that was located between the hydraulic jack and reaction frame (see Figure E.2).

  ![Figure E.6 Hydraulic compressor (Enerpac)](image)
E.2 Device calibration

The pressuremeter was calibrated using a series of calibration rings with internal diameters; ranging in size from 20.7mm to 22.0mm, as shown in Figure E.7. The calibration rings were placed one at a time on the body of the pressuremeter device and locked by an aluminium clamp that had been designed to centralise the calibration rings over the pressuremeter feeler-arms. Air pressure was used to inflate the device inside the ring with a constant pressure of 50kPa at a rate of 50kPa/min. During this procedure, the voltage output recorded the maximum inflation of the device. The relationship between the voltage output and the feeler-arm transducers displacement is linear.

![Image of calibration rings, clamp and holder](image)

Figure E.7 Calibration rings, clamp and holder

E.3 Sample preparation method

The sample preparation method used in the current work followed the preparation technique presented by Gaudin et al. (2005). The reconstituted UWA-FSS samples were prepared using the Sand-Rainer. This technique depends on depositing sand through air using an automatic hopper. Sand pours through a slot to produce uniform layer of sand rain over a circular chamber. The chamber has a diameter of 393mm and height of 400mm as shown in Figure E.8. According to Gaudin et al. (2005), looser samples were prepared using faster pouring rates and wider slots.
A 0.3mm thick elastic latex membrane surrounds the pressuremeter cell, which is sealed by the force of the screwed-in aluminium plates that sit over the rubber O-rings at either end (see Figure E.9). The device was installed inside the chamber, as shown in Figure E.10. The device was then fixed using some wires to ensure the pressuremeter was placed in the centre of the chamber and it was then kept inside an aluminium holder for the duration of the test to ensure the device did not move (see Figure E.1). Before the soil was poured into the chamber, the chamber was placed at 10cm above the floor. An automatic hopper was then used to ‘rain’ the sand into the chamber, appropriately therefore called the ‘Sand-Rainer’ (Figure E.11). The slot of the Sand-Rainer was one
Experimental procedures for pressuremeter tests

metre above the base of the chamber. The distance between the location of the sand layer and the opening slot of the hopper was fixed to a metre and in order to keep this distance constant, it was raised by an increment of 40mm every time the Rainer had deposited 40mm of sand to ensure that the soil sample had a consistent relative density throughout the chamber. The travel speed or velocity rate of the Sand-Rainer was fixed for all tests at 104mm/second. When the soil sample had filled the chamber, a steel plate was used to transfer pressure from the hydraulic system to the soil, as shown in Figure E.2. The weight and height of the sample were recorded for each test in order to identify the soil density for each sample. As shown in Figure E.2, the load was transferred to the soil sample through the hydraulic system, which included a hydraulic jack as well as a load cell to monitor the applied pressure.

The load transferred from the hydraulic system via the jack passed through a steel neck to the load plate which then beared down on the soil. On the other side, the load generated by the hydraulic system in the jack was transferred via a strong steel frame that was connected to the ground (see Figure E.2). The vertical pressure on the soil sample was produced using a hydraulic compressor, as shown in Figure E.6. At least two vertical displacement sensors were used in each test, which could have been laser sensors or Linear Variable Differential Transformers (LVDTs). All electrical sensors used in the test were connected to a computer through an electric signal-transforming device known as a Digital Instrument Amplifier called ‘DigDAQ’ (see Figure E.5). The vertical stress was held on the soil sample for seven days during the first few tests before the actual pressuremeter test was conducted in order to allow the sample to compact and creep under the applied load; this compaction period was then reduced to no less than 48 hours in subsequent tests.

The pressuremeter device drew the pressure from the air compressor as shown in Figure E.4. The air coming from the air compressor went through a digital pressure controller (see Figure E.3) and then onto a vessel to assist and control the pressure, as well as to provide clean pressure for the device (see Figure E.2). The air compressor feeding air pressure to the device caused the cavity expansion of the surrounding soil. The feeding rate of the pressure was 50kPa/min for most of the tests.
Figure E.9 Pressuremeter membrane

Figure E.10 Pressuremeter inside the chamber

Figure E.11 Preparing the sample using the Sand-Rainer
Appendix F  Experimental results for pressuremeter tests

This Appendix present the tests results for the buried pressuremeter tests conducted in the pressure chamber using UWA fine silica sand. The test equipment and procedure are described in Appendix E. Table F.1 listed 16 tests conducted in chronological order. The table list the test conditions, including, the vertical consolidation pressure ($\sigma'$), pressure rate, relative density ($D_r$), void ratio ($e$), and soil unit weight ($\gamma$). The tests there are classified into 9 categories depending on relative density and stress level (see Table F.1). The data is presented in the following figures. In each figure, a line of bets fit was generated using Equation (F.1), where $c$ and $m$ are constants.

$$p_c = c(e_c)^m$$  \hspace{1cm} (F.1)

<table>
<thead>
<tr>
<th>Test Cat.</th>
<th>Test No.</th>
<th>Test Date</th>
<th>$\sigma'$</th>
<th>Consolidation period</th>
<th>Pressure rate</th>
<th>$D_r$</th>
<th>$e$</th>
<th>$\gamma$</th>
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<td></td>
<td></td>
<td></td>
<td>(kPa)</td>
<td>(days)</td>
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<td>(%)</td>
<td>(kN/m$^3$)</td>
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F.1 Pressuremeter tests categories

Figure F.1 Pressuremeter test (category P1) for loose sand ($D_r \approx 40\%$, $\sigma_v \approx 38$ kPa)

Figure F.2 Pressuremeter test (category P2) for medium dense sand ($D_r \approx 60\%$, $\sigma_v \approx 38$ kPa)
Experimental results for plate/footing tests

Figure F.3 Pressuremeter test (category P3) for dense sand ($D_r \approx 70\%$, $\sigma_v \approx 38\text{kPa}$)

Figure F.4 Pressuremeter test (category P4) for loose sand under ($D_r \approx 40\%$, $\sigma_v \approx 50\text{kPa}$)
Figure F.5 Pressuremeter test (category P5) for medium dense sand ($D_r \approx 60\%, \sigma_v \approx 50\text{kPa}$)

Figure F.6 Pressuremeter test (category P6) for dense sand ($D_r \approx 70\%, \sigma_v \approx 50\text{kPa}$)
Experimental results for plate/footing tests

Figure F.7 Pressuremeter test (category P7) for loose sand ($D_r \approx 40\%, \sigma_v \approx 100\text{kPa}$)

Figure F.8 Pressuremeter test (category P8) for medium dense sand ($D_r \approx 60\%, \sigma_v \approx 100\text{kPa}$)
Figure F.9 Pressuremeter test (category P9) for dense sand ($D_r \approx 70\%$, $\sigma_v \approx 100\text{kPa}$)
F.2 Numerical investigation of pressuremeter chamber tests

Numerical investigations were conducted to verify the behaviour of pressuremeter chamber test. The Plaxis 2D (version 2015.2) Finite Element program, developed by Brinkgreve et al. (2017), was employed along with two models: Mohr-Coulomb (MC) model and Hardening Soil-Small (HSS) model. Three Finite Element (FE) models were created, to represent cylindrical cavity expansion, spherical cavity expansion and a third model to replicate the geometry of the actual UWA miniature pressuremeter device.

The cylindrical model, illustrated in Figure F.10, was created using 1750 axisymmetric triangular 15-noded elements. The mesh was refined around the membrane as shown in Figure F.10. The dimension of the model is illustrated in Figure F.10. The boundary to the right was allowed to move vertically, but was fixed radially (see Figure F.10). The boundary at the base was free radially and fixed vertically. The model includes three major steps: the surcharge pressure was applied using dummy layer in the initial step to establish the initial stresses in the soil with $K_0$ specified as 0.5; then the pressure was applied perpendicular to the face of the pressuremeter wall to simulate its expansion.

![Figure F.10 Cylindrical cavity pressure Plaxis model](image)

The spherical cavity expansion model was created using 980 axisymmetric triangular 15-noded elements was used. The mesh was refined around the membrane as shown in Figure F.11. The dimension of the model is illustrated in Figure F.11. The boundary to the two sides was allowed to move vertically, but was fixed radially. The boundary at the base was free radially and fixed vertically. The model includes three major steps: surcharge pressure was applied in the initial step, using a dummy layer as the surcharge following Suryasentana and Lehane's (2014) suggestion, to establish the initial stresses in the soil with $K_0$ specified as 0.5; then, the pressure was applied to the vertical faces of the pressuremeter wall to simulate its expansion.
Figure F.11 Spherical cavity pressure Plaxis model

The actual geometry of the UWA miniature pressuremeter was represented using the axisymmetric model, shown in Figure F.12, which comprise 687 triangular 15-noded elements. The mesh was refined around the membrane, as shown in Figure F.12. The dimension of the model is illustrated in Figure F.12. The boundary to the two sides was allowed to move vertically, but was fixed radially. The boundary at the base was free radially and fixed vertically. The model includes three major steps: the surcharge pressure was applied in the initial step to establish the initial stresses in the soil with $K_0$ specified
Experimental results for plate/footing tests

as 0.5; then, the pressure was applied to the vertical faces of the pressuremeter wall to simulate its expansion.

Figure F.12 Cavity pressure Plaxis model for UWA pressuremeter chamber test

Two constitutive models were used in this investigation: the MC model and the HSS model. Figure F.13 and Figure F.14 show that the pressure-expansion curve for the UWA miniature pressuremeter is most similar to spherical cavity expansion.
Figure F.13 Plaxis model result for cylindrical, spherical and UWA pressuremeter chamber test using Mohr-Coulomb (MC) model

Figure F.14 Plaxis model result for cylindrical, spherical and UWA pressuremeter chamber test using Hardening Soil-small (HSS) model
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