COLLAPSE BEHAVIOUR OF A COMPACTED SILTY CLAY

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By

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ABSTRACT

A large number of engineering problems are associated with partly saturated soils. One of the most common problems is that of collapse of materials such as loess, or loosely compacted fills, which can undergo large settlements as they are wetted at relatively large stresses. Another major problem is that of shear failure of these materials. A sound understanding of the mechanical behaviour of this type of material is thus required so that the engineer can devise safe and cost-effective solutions to these problems. A number of constitutive models have been proposed to describe the behaviour of such soils, many of which however still need further experimental validation.

A suction-controlled triaxial apparatus that incorporates the tensiometer was developed at Imperial College and used for investigating the behaviour of a reconstituted silty clay. This apparatus has been developed further in this research so that both drying and wetting paths could be followed and so that the moisture change of the sample could be monitored during tests. This apparatus was used to investigate the collapse, shear, and water-retention behaviours of a loosely compacted mixture of 70% silt, 20% kaolin, and 10% London clay. In addition to the triaxial tests, other experiments carried out included the suction-controlled oedometer, unconfined drying/wetting tests and a fabric study using a petrological microscope, in order to gain a more complete understanding of the behaviour. The influence of fabrics induced by differing compaction properties was also examined in relation to the water retention and collapse behaviour.

The two main types of constitutive models describing the collapse behaviour of unsaturated soils were used to fit the experimental results: those using the conventional stress variables (net stress and suction) and those using modified stresses. The main assumption regarding the uniqueness of the Loading-Collapse surface was validated as identified by results of loading and wetting tests. The model employing the modified stress approach could not reproduce accurately the test results at suctions greater than about 3000kPa. The evidence for critical states for compressive shearing has also been examined using stress-dilatancy plots. The concept of Soil-Water-Retention Surface has been validated and extended to cover the influence of different components of strain.
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LIST OF SYMBOLS AND ABBREVIATIONS

\[ s = u_a - u_w \]  
matrix suction

\[ u_a \]  
pore air pressure

\[ u_w \]  
pore water pressure

\[ Rh \]  
relative humidity

\[ R \]  
The universal gas constant (8.314 J.mol\(^{-1}\).K\(^{-1}\))

\[ V_{mol} \]  
molecular volume of water vapour (0.01802 m\(^3\))

\[ T \]  
absolute temperature (°K)

\[ \psi = s + \pi \]  
total suction

\[ \pi \]  
 osmotic suction

\[ S_r \]  
degree of saturation

\[ w, w/c \]  
gravimetric water content

\[ \theta \]  
volumetric water content

\[ G_s \]  
specific gravity of the soil

\[ e \]  
void ratio

\[ \nu = e + 1 \]  
specific volume

\[ n \]  
porosity

SWRC  
Soil Water Retention Curve

\[ \alpha_d \]  
contact angle for the meniscus during drying

\[ \alpha_w \]  
contact angle for the meniscus during wetting

R  
radius of meniscus

PEG  
Polyethylene Glycol

LVDT  
linear variable differential transformer

\[ \sigma' = (\sigma - u_a) + \chi(u_a - u_w) \]  
Bishop’s effective stress

\[ \sigma \]  
total stress

\[ \chi \]  
Bishop’s parameter (function of degree of saturation)

\[ \tau \]  
shear strength

\[ c' \]  
effective cohesion intercept

\[ \phi' \]  
effective friction angle

\[ \phi^b \]  
friction angle with respect to changes in suction
K fitting parameter Vanapalli et al. (1996)
Θ normalized water content, $\theta_w/\theta_s$
$\theta_w$ volumetric water content
$\theta_s$ saturated volumetric water content
$\theta_r$ residual volumetric water content
$q, q = (\sigma_v - \sigma_h)$ deviatoric stress for the triaxial condition ($\sigma_v = \sigma_a, \sigma_h = \sigma_r$)
p mean total stress or net mean stress if $u_a = 0$
\[ p = \left(\sigma_v + 2\sigma_h\right)/3 \] for the triaxial condition
$M_a$ critical state stress ratio due to total stress (similar to $\phi'$)
$M_b$ critical state stress ratio due to suction (similar to $\phi^b$)
$M_s$ or $M$ critical state stress ratio for fully saturated condition
$S_{r1}, S_{r2}$ reference degrees of saturation
$k_a, k_b$ fitting parameters due to total stress and suction
$p'' = p - u_a$ net stress or Bishop stress as stated in text
$p_o^*$ hardening parameter (Alonso et al., 1990)
LC Loading Collapse
$p^c$ reference pressure (Alonso et al., 1990)
$\lambda(s)$ gradient of the Normal Compression Line
SI Suction Increase (yield surface)
SD Suction Decrease (yield surface)
$\Gamma_a(s)$ intercept of critical state line for volumetric plane at $p-u_a = 1$ kPa
$\phi(s)$ gradient of critical state line in $\nu$-$\ln (p-u_a)$ plane
$\Gamma_{ab} = 1 + \frac{\Gamma_r - 1}{S_r}$ critical state intercept for volumetric plane at $p-u_a = 1$ kPa and $u_a-u_w = 1$ kPa
$\lambda_a$ slope of critical state line in $\nu$-$\ln (p-u_a)$ plane
$\lambda_b$ slope of critical state line in $\nu$-$\ln (u_a-u_w)$ plane
$\nu_w = S_r \cdot \nu$ specific water content of the soil
$m, n, \phi$ and $\psi$ soil constants for the degree of saturation function by Gallipoli, Wheeler & Karstunen (2003b)
\[ \sigma_{ij}^e = \sigma_{ij} - \left[ S, u_w + (1 - S_r)u_a \right] \delta_{ij} \quad \text{Bishop’s stress by Wheeler et al. (2003)} \]

average skeleton stress by Gallipoli et al. (2003a)

\[ \xi = f(s)(1 - S_r) \quad \text{meniscus bonding factor} \]

\[ f(s) \quad \text{factor accounts for the increase with increasing suction of the stabilising} \]

inter-particle force

\[ e_s \quad \text{void ratio in a saturated condition} \]

\[ a \quad \text{fitting parameters for} \quad \frac{e}{e_s} = 1 - a \cdot \left[ 1 - \exp(b \cdot \xi) \right] \]

\[ s^*, s = n(u_a - u_w) \quad \text{modified suction} \]

\[ p^* \quad \text{Bishop’s mean stress} \]

LL \quad \text{Liquid Limit}

PL \quad \text{Plastic Limit}

SWRC \quad \text{Soil-Water Retention Curves}

NCL \quad \text{Normal compression Line}

CSL \quad \text{Critical State Line}

\[ \varepsilon_q = \frac{2}{3} (\varepsilon_a - \varepsilon_r) \quad \text{deviatoric strain} \]

\[ \varepsilon_r \quad \text{radial strain} \]

\[ \varepsilon_v \quad \text{volumetric strain} \]

\[ \varepsilon_a \quad \text{axial strain} \]

\[ \eta = q/p \quad \text{stress ratio} \]

\[ \sigma_v \quad \text{vertical stress} \]

\[ v_s \quad \text{specific volume at zero suction} \]

\[ v_1 \quad \text{and} \quad b \quad \text{fitting parameters for} \quad \frac{v_s}{v_1} = p^{-b} \]

\[ v_c \quad \text{potential collapse} \]

\[ v_{c0}, v_{c1} \quad \text{and} \quad \beta \quad \text{fitting parameters for} \quad \frac{v_c - v_{c0}}{v_{c1} - v_{c0}} = p^{-\beta} \]

\[ v_{lab} = 1 + \frac{(v_{la} - 1)}{S_r} \quad \text{critical state intercept for volumetric plane} \]

\[ b_a \quad \text{slope of critical state line in ln v-ln (p-u_a) plane} \]
\( b_h \)  
\[ b_h \] slope of critical state line in \( \ln \nu - \ln (\mu_e - u_w) \) plane

\( S_{\nu M} = \frac{e_w - e_{wm}}{e - e_{wm}} \)  
\[ S_{\nu M} \] degree of saturation of the macropores

\( e_w = S_{\nu} \cdot e \)  
\[ e_w \] water ratio

\( e_{wm} \)  
\[ e_{wm} \] microstructural water ratio (Romero & Vaunat, 2000)

\( \psi_b \)  
\[ \psi_b \] blow-through suction

\( \psi_{res} \)  
\[ \psi_{res} \] residual suction

\( S_{res} \)  
\[ S_{res} \] degree of saturation at the residual point

\( S_b \)  
\[ S_b \] degree of saturation at the second blow-through value

\( S_{r ko} \)  
\[ S_{r ko} \] degree of saturation for \( K_0 \) condition

\( S_r^d \)  
\[ S_r^d \] difference between the value of degree of saturation in any condition and that at the same void ratio in \( K_0 \) condition
CHAPTER 1

INTRODUCTION

1.1 General background

Many engineering problems are associated with soils in their partly saturated states, where both water and air are present within their voids. This type of material is very common, for instance, in arid or semi-arid areas, where the ground water level is normally relatively deep. Compacted fills, used in man made structures such as earth dams, road subgrades, and embankments, are also placed in an unsaturated state. In hillslope areas of the tropics, rainfall-induced landslides, normally occurring as shallow movements in an unsaturated zone, are one of the major causes of economic and social loss. Many collapsible materials, such as loess or loosely compacted fills can undergo large settlements as the materials are wetted at relatively large overburden pressures, bringing about damage to the overlying structures. Future climate changes, which could potentially cause significant changes in the soil moisture regime for many areas of the world, as well as rapid developments in many arid areas and the tropics, will be factors inducing further problems associated with unsaturated soils.

A sound understanding of the mechanical behaviour of this type of material is thus required, in order that safe and cost-effective solutions to these engineering problems can be arrived at. The main interest of the current research lies in the mechanical behaviour of a non-expansive collapsible material. Problems related to highly expansive or shrinkable materials such as high plasticity clays, which are also common and of great significance, are not addressed in this study.

The behaviours of unsaturated soils in general appear to be relatively complex in comparison to those of fully saturated soils. This is due to the larger number of physical phenomena involved in unsaturated soils: for example, the hydraulic hysteresis during wetting/drying cycles, the presence of meniscal water lenses as bonding agents around soil particles, the swelling and hydration of clay minerals, etc. Nevertheless, since the 1950’s significant developments have been made in understanding unsaturated soil behaviour. Early attempts were aimed at treating unsaturated soil behaviour in terms of a single effective stress (e.g. Bishop, 1959). Jennings and Burland (1962) were among
the first to recognise the limitation of this approach, pointing out the fundamental difference in the way in which total stress and suction contributes to the soil grain stability. Subsequently, many researchers proposed the frameworks in terms of two independent stress variables, net stress and suction. One of the most widely used elastoplastic frameworks was first proposed by Alonso et al. (1987) for collapsible, non- to slightly expansive soils.

This type of framework was nevertheless found not to be able to reproduce some behaviour observed in the laboratory, including, for example, that during drying/wetting cycles and the influence of these cycles on subsequent behaviour. More recent approaches (e.g. Wheeler et al., 2003, and Gallipoli et al., 2003a) then shifted towards a new choice of stress variables, which take into account the contributions of both suction and degree of saturation to the soil grain stability and to the effective stress, similar to the Bishop (1959) approach. In brief, a variety of frameworks has been proposed, most of which are still at the early stage of development, and thus need experimental validation.

1.2 Objectives

Recent research on unsaturated soil behaviour at Imperial College London has focused on developing the equipment which is capable of testing unsaturated soils at atmospheric pressure. This technique was made possible by the advent of the Imperial College tensiometer (Ridley, 1993), which is capable of measuring negative pore water pressures up to about 1500kPa. A series of research projects has been conducted using this equipment (e.g. Dineen, 1997, Colmenares, 2002, Melgarejo, 2004). The first attempt at incorporating the tensiometer into the triaxial apparatus was made by Cunningham (2000). The apparatus employed a system which was capable of controlling a constant suction in the soil sample during testing, but was not able to control wetting paths. It was thus an aim of the present research to develop further the suction-controlled triaxial apparatus, so that it was capable of performing tests with a wider range of stress paths.

From an understanding of unsaturated soil behaviour, the ideal material for the study was a collapsible, non-expansive soil, that could to be reproduced readily in the
laboratory to enable repeatable test results to be obtained. The material finally chosen was a mixture of 70% HPF4 Silt, 20% Speswhite Kaolin, and 10% London Clay. This material, Soil A, has been investigated in its reconstituted state by Colmenares (1997) and Cunningham (2000). Soil A in its reconstituted state however did not exhibit the collapse-upon-wetting behaviour. In order to make this mixture collapsible, it was formed instead by compaction at dry-of-optimum water contents. The compaction characteristics were also varied in order that the difference, if any, in the compacted fabrics could be investigated. A series of experiments has been conducted using a variety of equipment, including a suction-monitored oedometer apparatus, as well as a suction-controlled triaxial apparatus. The soil behaviour during unconfined wetting/drying was also investigated, using the filter paper technique and the suction plate. The fabrics of the material compacted with different initial properties and at different states were also examined using the petrological microscope to provide a qualitative description of the fabric, with reference to the behaviour observed in the main experiments.

To summarise, the various subjects which the current research aimed at addressing are as follows.

- Further development of the suction-controlled triaxial apparatus, incorporating the Imperial College tensiometers
- The compaction characteristics of Soil A and unconfined wetting/drying behaviour
- The collapse behaviour of the selected compacted Soil A, as observed in the suction-monitored oedometer apparatus
- The collapse and shear behaviour of the selected compacted Soil A, in the suction-controlled triaxial apparatus
- A qualitative description of the fabrics of compacted Soil A at different states
- Validations of and, where possible, improvements, to existing models based on the observed behaviour of Soil A
1.3 Thesis outlines
This thesis is arranged according to the aforementioned elements of work. A brief review of each chapter is given as below.

Chapter 2 Literature review
A review is given of the behaviour of partly saturated soils in general, of the experimental techniques required for testing them, and of their constitutive modelling. This chapter presents the necessary background for the subsequent chapters. Gaps in knowledge are discussed, together with the specific objectives of the current research.

Chapter 3 Selection of material and experimental procedures
The process of selecting the most suitable material for the experimental programme is explained. Details of the experiments are described, consisting of the sample preparation methods, the apparatus, the experimental set up and the testing procedures. Improvements made to the existing suction-controlled traixial apparatus are highlighted.

Chapter 4 Fundamental properties of compacted Soil A
The compaction characteristics and as-compacted suctions of Soil A are described. The choices for the initial properties of compacted Soil A are also explained, for the three main series of samples (Series 7-10, 7-13 & 5-10). Finally, the unconfined wetting/drying behaviour is described of samples from the three main series, as identified using the filter paper technique and the suction plate. The observed behaviour is also compared with that of reconstituted samples tested by Cunningham (2000).

Chapter 5 Suction-monitored oedometer tests
The collapse behaviour of the three main series, as observed in suction-monitored oedometer tests, is described. The behaviour is examined within the framework of Alonso et al. (1987). The Loading-Collapse surfaces are identified for the three main series. Also highlighted is the uniqueness of these surfaces as identified from loading and wetting tests. Differences in the location of the Loading-Collapse surface due to differences in as-compacted fabrics are also discussed.
Chapter 6 Fabric studies
A qualitative study of fabrics as observed through a petrological microscope was carried out. Some evidence is shown of differences in the fabrics of the samples, resulting from differences in compacted properties and subsequent loading/wetting. These differences in fabrics are also discussed in relation to the observations and hypotheses made about the soil-water retention and collapse behaviours, described in Chapters 4 and 5.

Chapter 7 Triaxial tests
This chapter describes the main testing programme of this research project. The volumetric and shear behaviours of compacted Soil A from Series 7-10 are described in detail. The collapse behaviour is investigated for samples following wetting and loading paths, under isotropic, anisotropic and $K_o$ conditions. The shearing behaviour was investigated only in compression at various suctions and cell pressures. Also included in this chapter is a discussion of the results from a series of conventional stress path shearing tests on fully saturated samples of Soil A, both in reconstituted and compacted states. The differences in behaviour between the reconstituted and compacted samples are discussed.

Chapter 8 Data interpretation
A quantitative interpretation is made of the behaviour of the compacted Soil A, as observed from all experiments with reference to the two types of elasto-plastic frameworks: those using the two conventional stress variables, and those using the two modified stress variables. Both volumetric and shear behaviours are interpreted. Limitations of models employing modified stress variables are identified. The Soil-Water Retention behaviour is also discussed, together with its dependency on the void ratio and different components of strain.

Chapter 9 Conclusions
The conclusions drawn from various parts of the thesis are summarised. Recommendations for future work are given.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction
A review is given in this chapter of the behaviour of partly saturated soils, the experimental techniques required for testing them, and their constitutive modelling. It is aimed at presenting the necessary background for the subsequent contents of the thesis. The chapter is divided into three main parts. The first part is concerned with the fundamental physics associated with soil suction and soil-water retention behaviour. Secondly, the techniques employed for testing partly saturated soils will be summarised, including suction measurement methods and triaxial testing. Lastly, the mechanical behaviour of partly saturated soils and various attempts at their modelling will be reviewed. The gaps in current knowledge will then be discussed, together with the objectives of the present research.

2.2 Soil suctions and Soil-Water Retention behaviour
Soil suction can be described as the potential of the soil for water attraction. It is the key variable that governs the mechanical behaviour of partly saturated soils as well as the flow regime within the soils. A brief review is given here, concerning the definition of types of suction and the underlying physical phenomena associated with them.

2.2.1 Physics for soil suctions and definitions
Soils above the groundwater table have an affinity for water, which, either partially or fully, fills in the space within the soil pores. This soil water above the groundwater table is normally under a tensile stress or a negative pore water pressure. This tensile stress, measured through a porous tip making an intimate contact with the soil water, is known as the matrix suction. The matrix suction, \( s \), is defined as the excess of pore air pressure, \( u_a \), over pore water pressure, \( u_w \), as expressed by Equation 2.1.

\[
s = u_a - u_w
\]  \( (2.1) \)
It thus becomes the negative pore water pressure at an atmospheric air pressure ($u_a = 0$). The matrix suction is caused by the two main physical phenomena, namely the capillarity, and the surface adsorption.

The capillarity phenomenon is directly related to the surface tension of water. As illustrated for a case of the capillary tube in Figure 2.1a, this capillary force, and thus the suction, is proportional to the curvature of the water meniscus. The smaller the tube radius, the greater the curvature, and the higher the water will rise within the tube. Similarly, the capillary rise above the groundwater level within a soil depends on the pore size, as illustrated in Figure 2.1a for the case of two idealised spheres. Fine textured soils normally have a higher capillary rise than that within coarse textured soils.

The surface adsorption is principally relevant to clay minerals and occurs as a result of the clay particles’ negatively charged surfaces, (see Figure 2.1b). This powerful electrical force around clay particles strongly attracts water molecules. The types of clay mineral also play a very important role in the adsorption of water by clay surfaces. For example, soils with high activity clay minerals, such as montmorillonite, are able to retain a greater amount of water around their particles and remain saturated even at suctions as high as 10-20 MPa. They also have a tendency for greater shrinkage or expansion in volume upon drying or wetting respectively.

Another way of expressing the affinity that a soil has for water is through the relative humidity of the ambient air close to the soil. This affinity is called the total suction, $\psi$, which, by definition, is the stress required to remove a water molecule from the soil into the vapour phase. The total suction is related to the relative humidity by the relationship,

$$\psi = -\left[\frac{RT}{V_{mol}}\right]\ln(Rh)$$ \hspace{1cm} (2.2a)

$$\psi = s + \pi$$ \hspace{1cm} (2.2b)
where $Rh$ is the relative humidity, which equals the ratio between the partial pressure of the vapour and the saturated vapour pressure of the soil air ($P/P_0$), $R$ is the universal gas constant (8.314 J.mol$^{-1}$.K$^{-1}$), $V_{mol}$ is the molecular volume of water vapour (0.01802 m$^3$), and $T$ the absolute temperature (°K).

As Equation (2.2b) shows, the total suction is the sum of the matrix suction, $s$, and the osmotic suction, $\pi$. The osmotic suction is a function of the concentration of dissolved salts within the soil water, whose presence gives rise to some additional affinity for water of the soil. In general, it is the total suction that governs the flow within unsaturated soils. As an example, provided that other potentials are the same, any gradient in the concentration of dissolved salts within soil water, or in the osmotic suction, can cause flow of water in a soil. The water content of a soil in contact with the atmosphere is also determined by the total suction, which corresponds to the atmospheric humidity.

However, it is normally considered that the strength of an unsaturated soil is determined by the matrix suction, even though any presence of salt within the soil water can cause some fundamental change in its mechanical behaviour (Alonso, Gens & Hight, 1987). Matrix suction and osmotic suction are generally considered as independent variables. In addition, thermodynamic considerations show that the total suction corresponding to zero water content is approximately 1000 MPa (Fredlund, 1991).

2.2.2 Soil-Water Retention behaviour

The Soil-Water Retention Curve (SWRC), also called the Soil-Water Characteristic Curve, is a function, which describes the relationship between suction and the corresponding state of wetness of the soil. The state of wetness can be expressed in various ways, namely, degree of saturation, $S$; gravimetric water content, $w$; or volumetric water content, $\theta$; which are all related by the equation:

$$\theta = \frac{w \cdot G}{1 + e} = \frac{S \cdot e}{1 + e}$$  \hspace{1cm} (2.3)
where \( G_s \) is the specific gravity of the soil, and \( e \) the void ratio. A general description of the SWRC is shown in Figure 2.2 in a plot between degree of saturation and logarithmic suction. To begin with, the sample is gradually dried from a fully saturated condition. As the suction increases, the sample remains fully saturated until air starts entering the largest pore at the desaturation or air-entry point. The sample then continues desaturating with increasing suction until the residual point is reached, after which degree of saturation hardly changes with suction. On rewetting, the path follows a different curve, below the drying curve, due to hysteresis. When the suction is reduced to zero, some air within the soil might still exist in an occluded form, resulting in a degree of saturation of less than 100%. For rigid soils, such as most granular soils whose void ratio hardly changes upon drying or wetting, the hysteresis occurs as a result of at least two factors, including the ink bottle effect, and the pore-fluid contact angle.

Figure 2.3a shows a schematic diagram of a pore with an ink bottle shape. Upon drying, the pore is drained once the suction exceeds the value corresponding to the meniscus of radius \( r \). However, to refill the emptied pore, the suction has to reduce to the smaller value, which corresponds to the meniscus of radius \( R \). Figure 2.3b illustrates the contact angle effect. During drying, the contact angle for the meniscus, \( \alpha_d \), is smaller than the contact angle \( \alpha_w \) during wetting, due to the movement of water. Consequently, the curvature of the drying meniscus, together with its corresponding suction, is greater than that during wetting. Thus, as a result of these two effects, at a given moisture content or degree of saturation, the suction during drying is always higher than that during wetting.

Childs (1969) gives an elaborate set of definitions for the various retention curves for rigid soils, including the boundary drying/wetting curves, the primary drying/wetting curves, and the scanning curves. These curves are illustrated in Figure 2.4. The boundary drying curve is defined as ‘a complete moisture characteristic between saturation at vanishing suction as the initial state and the maximum suction attainable’. The definition of the boundary wetting curve is the reverse of that of the boundary drying curve. The primary drying curve is defined as ‘a characteristic from an intermediate point on a boundary wetting curve and carried to maximum suction’. The
primary wetting curve can be defined in a similar way. The scanning curves are those curves beginning at an intermediate point on a primary curve.

It should be emphasized that these definitions by Childs (1969) were given for rigid soils. For deformable soils, in particular clayey soils, the volume change corresponding to a change in suction or applied stress can be more considerable, and the hysteresis not only results from the two factors already mentioned, but also from the change in pore structure accompanying the rearrangement of the soil skeleton caused by the change in suction or applied stress. Moreover, Hillel (1998) suggested that failure to attain true equilibrium during testing could also accentuate the hysteresis effect, though strictly speaking, this is not a true hysteresis. Recent developments in modelling the SWRC for deformable soils will be reviewed in Section 2.4.4.

2.2.3 Factors that influence the shape of SWRC

The shapes of retention curves are influenced by both structure and composition of a soil. In the low-suction range (0-100 kPa), the amount of water retained depends mainly on the capillary effect and the pore-size (Hillel, 1998). Therefore, the SWRC is principally affected by soil structure in this range. As the suction increases, the surface adsorption plays a greater role in retaining water around soil particles. Accordingly, at higher suction, the water retention is influenced less by the structure and more by the composition and specific surface of the soil.

The shapes of SWRCs can also be considered as an indication of the pore size distribution of the soil. For the soils with a normal pore size distribution, the typical s-shape of SWRC, as shown in Figure 2.2, is usually observed. However, if the soil is highly structured, its pore size distribution may be multi-level, resulting in the multimodal SWRC shown in Figure 2.5, for the case of a pelletized diatomaceous earth material reported by Burger & Shackelford (2001). Bi-modal SWRCs have been observed in some other structured soils such as aggregated loams, loess deposits, and collapsible clays (e.g. Othmer et al., 1991 & Camapum de Carvalho et al., 2002).

There have been a number of recent studies directed at investigating the influence on SWRCs of various factors, including stress state, compaction effort, and stress history (e.g. Vanapalli et al., 1999 and Ng & Pang, 2000). These factors essentially affect the
pore shape and their distribution within the soil. In general, as the compaction effort or the applied stress increase, the pore size would be expected to decrease, thus resulting in a shift of the SWRC to higher suctions.

A number of empirical models (e.g. Brooks & Corey, 1966 and van Genuchten, 1980) have been developed to describe the SWRCs in the area of soil science and hydrology. Since the main interests of these researches lie in the flow-related problems, the proposed formulations normally involve the assumption that the soil is rigid. A summary of various SWRC equations is given by Sillers et al. (2001). In attempting to model the hysteresis of rigid soil, the concept of hysteresis loop proposed by Childs (1969) is normally employed. Two different mathematical expressions are typically required to describe the boundary wetting/drying curves, which are then used to predict the scanning curves (e.g. Pham et al., 2003).

Recently, SWRC expressions have been adopted by geotechnical engineers for the prediction of various properties including shear strength, permeability, and thermal coefficient (Barbour, 1998). However, care should be taken when adopting this approach since, for deformable soils, the SWRC depends on many other factors, including the applied stress, void ratio and hysteresis, as explained above. Only the SWRCs that correspond to the actual field situation should be used, but this can be difficult to predict. More details of further developments to incorporate some forms of SWRC relationships within an elasto-plastic framework will be discussed in Section 2.4.4h.

In conclusion, the SWRC of a soil with a certain grading and composition can vary according to a number of factors, including the hydraulic hysteresis as well as the transient nature of the pore size distribution of a deformable soil. Any attempt to model the SWRC should take due account of these various factors.

### 2.3 Experimental techniques for testing partly saturated soils

The experimental techniques involved in the investigation of the mechanical behaviour of partly saturated soils will be reviewed in this section, categorised into four groups, namely suction measurement, suction control systems, volume measurement and water content measurement. As discussed earlier, the soil suction is a very important variable
that governs the mechanical behaviour of partly saturated soils. Attempts to measure and control the suction have thus been central to the laboratory testing of partly saturated soils. In addition, since the void ratio and water content of partly saturated soils are not coupled, their measurements need to be carried out separately.

2.3.1 Suction measurements

Ridley & Wray (1996) categorise the suction measuring devices into two types; those that measure directly and others that measure indirectly. With direct measurement, the relevant quantity under scrutiny is measured, namely the pore water energy or tensile stress. The indirect measurements are ones that measure another parameter (e.g. relative humidity, resistivity, conductivity or moisture content), which are related to the suction through calibration relationships.

For both cases, they emphasize the importance for the engineer of knowing whether total or matrix suction is being measured. In general, if no contact is made between the measuring device and the soil water, the relative humidity of the ambient air within the soil is measured and so it is the total suction. If the good contact is made between the soil water and the measuring device, and the concentration of dissolved salts can be assumed to be the same everywhere, the matrix suction is being measured. However at low degrees of saturation, the soil water might recede into the finer pores and become discontinuous, in which case it is normally difficult to guarantee that full contact is made and that the matrix suction is being measured.

A review is given here to cover only the techniques used in the current research, namely the Imperial College tensiometer and the filter paper method. For other techniques, the reader is referred to Ridley & Wray (1996) for a more complete review.

a) Imperial College tensiometer

A tensiometer is a device that measures directly the absolute negative pore water pressure in the soil water at atmospheric pore air pressure. It essentially consists of a saturated porous ceramic filter, a reservoir of water, and a pressure measuring device. When the porous ceramics filter is in good contact with the soil water, water will flow between the soil and reservoir through the filter, until equilibrium is established at a negative pressure between the water within the soil and the water in the reservoir. The
soil suction, thus transferred to the reservoir, is subsequently measured by a manometer, gauge, or electronic transducer. If a good contact is made between the tensiometer and the soil water, it is the matrix suction that is measured by the device.

The main limitation associated with the use of a conventional tensiometer is that bubble formation normally occurs when the suction reaches between 60 to 100kPa, and subsequently the suction will no longer be transmitted efficiently to the measuring device. The axis-translation technique avoids this problem by raising the ambient air pressure so that the pore water pressure, which is also raised by an equal amount, becomes measurable (Hilf, 1956). However this technique is not practicable for field measurement. Burland & Ridley (1996) also suggest that caution should also be exercised in making use of this technique in the laboratory because changing the absolute water pressure may affect air coming into or out of solution, especially when the experiments involve wetting and drying processes.

The suction probe or Imperial College tensiometer was introduced by Ridley (1993) to overcome the problem of bubble formation, or cavitation, associated with conventional tensiometers. The suction probe can measure suctions in excess of 100kPa for relatively long periods (e.g. > 1 months). Since its initial development, the suction probe has undergone further improvements (Ridley & Burland, 1993, 1995 & 1999). The design considerations for the probe include the optimisation of various components, namely a) the location of and type of the pressure sensor, b) the volume of the fluid reservoir c) the material from which the tensiometer is constructed and d) the pore size of the ceramic filter.

The final design, by Ridley & Burland (1999), is shown schematically in Figure 2.6. The probe is constructed entirely of stainless steel and fitted with a high air entry (1500kPa) ceramic filter. The very small volume of the water reservoir was designed in order to inhibit cavitation. If the probe has been preconditioned carefully, the maximum suction that can be measured is normally around 1500kPa, corresponding to the air entry value of the filter. Experience suggests that the measurement can be sustained for several weeks provided the suction does not exceed around 75% of the measured air-entry value of the filter.
The preparation procedure for the suction probe has been described by Ridley & Burland (1999) and should be followed carefully so that a reliable measurement is made. The calibration of the probe is carried out in the positive pressure range and thus extrapolation is needed for the usual operation in the negative range. Ridley (1993) and Tarantino & Mongiovi (2003) showed experimentally the justification of this extrapolation. In Section 3.4.1, the details of the use of suction probes in the present study will be given, including typical suction measurement plots, the problems encountered during the experiments, and a minor modification made to the design.

b) Filter paper technique

This technique involves placing filter papers in contact with a soil sample in an air-tight container for a period (normally around one week) in order that equilibrium is attained between the suctions in the sample and the papers. The water content of the filter papers is then measured and related to the suction using its soil-water retention relationship, i.e. the relationship between the suction and water content of the papers.

The method was first used in the field of agricultural science and has since undergone subsequent improvement in the testing procedures. Marinho (1994) provides an extensive review of these developments. In the laboratory at Imperial College, this technique has been used extensively with Whatman No.42 filter papers and the procedures have been improved upon by various researchers (Chandler & Gutierrez, 1986, Marinho, 1994, Cunningham, 2000 and Melgarejo, 2004). Ridley et al. (2003) provide a summary of the procedure currently used at Imperial College.

As for other porous media, the relationship between suction and water content of the filter paper is affected by the hysteresis and direction of flow of the water. In this respect, Ridley (1995) emphasized the importance of using different calibration curves for the initially wet and initially dry filter papers. In addition, since the technique relies on the transfer of moisture between the soil sample and the filter paper, the condition of contact between the paper and the sample is very important in determining which suction, total or matrix, is being measured. In general, when good contact is made between the paper and the sample, the matrix suction is believed to be measured. When contact is not made, it is the total suction, which is being measured. However, there is a transition region where it is not certain which suction is being measured, especially
when the soil has a low degree of saturation and soil water retreats into the finer pores. In this region, even when a contact is made, there is no guarantee that the matrix suction is being measured. In this respect, Ridley (1995) suggests that careful observation of the condition of the filter paper when it is removed from the sample can give an indication of the change from matrix suction to total suction measurement. For instance, if the filter paper sticks to the sample, it is more likely that the matrix suction is being measured.

Ridley et al. (2003) also report the influence of the salt present in the soil on the equilibrium water content in the filter papers making in-contact measurements. The equilibrium water content of the filter paper reduces when there is salt present in the soil sample. Therefore the suction in salty soils may be overestimated if the usual calibration of Chandler & Gutierrez (1986) is used.

Regarding the total suction measurement, since no contact between the soil and the papers is made, the transfer of moisture is only through the vapour phase. The vapour transfer process normally takes place over a relatively long period and consequently the equilibrium time is of great importance in determining the calibration relationship. Figure 2.7 demonstrates the influence of equilibrium time on measurements for filter papers making no contact. The 14-day measurement yields twice as much moisture content of the filter paper as for the 7-day measurement.

Other factors that can affect the calibration relationship of the filter paper include the gap distance between the soil and the filter paper for non-contact measurement, and the temperature fluctuation in the laboratory. As suggested by Ridley (1995), it is very important when using the filter paper method that a calibration is performed under conditions that will be as near as is reasonable to the conditions likely to be encountered during the subsequent tests. More details of the testing procedures for the filter paper technique used in the present studies are given in Section 3.4.2.

2.3.2 Suction control system

Three main types of system for controlling soil suction have commonly been used in research laboratories; the axis translation technique, the osmotic system, and the relative humidity control system. A recent advance has been made with the development of an
air-regulated system by Cunningham (2000), which employs a flow of dry air across the sample surface in order to increase the suction during loading and shearing.

a) Axis translation technique

As explained earlier, provided that there is no change in water content and suction, if the pore air pressure, $u_a$, is raised, the pore water pressure, $u_w$, will also increase by the same amount. If the increase in $u_a$ is larger than the initial negative $u_w$, the raised value of $u_w$ will become positive and can be measured using conventional pressure transducers.

The axis translation technique was first proposed and developed by soil scientists (Schofield, 1935 and Hilf, 1956) and its first use amongst the geotechnical community was reported by Bishop & Donald (1961). They developed a double walled triaxial apparatus as shown schematically in Figure 2.8. The system was capable of controlling the air pressure and water pressure independently, which allowed direct control of suction and tests involving wetting and drying paths to be performed. The axis translation technique was also used by many other pioneering researchers in unsaturated soil mechanics and incorporated into a variety of apparatus including, for example, an isotropic compression cell by Jenning & Burland (1962), the Rowe cell by Barden & Sides (1970) and the direct shear apparatus by Escario (1980).

The maximum suction, attainable using the axis translation system, is limited by the air pressure system, robustness of the cell and the air entry value (AEV) of the porous stone separating the soil sample and the pore pressure measuring system. One of the problems related to the axis-translation system involves the air-diffusion through the high AEV porous stone into the pore water pressure system. This leads to the formation of air bubbles, thus reducing the efficiency of the system. A flushing system is needed to overcome this problem, as developed for example by Bishop & Donald (1961), Fredlund (1975) and Sivakumar (1993).

Axis translation is normally considered to be valid only when the soil sample is at a degree of saturation less than 80%. For higher degree of saturation, Bocking & Fredlund (1980) demonstrated that the air permeability was effectively zero and all of the pore air
in the soil was most likely occluded. In this case, an increase in the air pressure will result in volumetric strains due to compression of the occluded air bubbles. Another uncertainty relating to the axis-translation technique involves the mechanism by which the soil desaturates under elevated air pressure and water pressure. More research is required to investigate the difference between the soil behaviour at elevated air pressures and at atmospheric air pressure.

Nevertheless, this technique is by far the most commonly used in the research laboratory and most of the experimental results used to validate various constitutive models in the literature had been obtained with this technique.

b) Osmotic system

Historically, the suction-controlled system using the osmosis phenomenon was first developed in the field of soil science by Zur (1966). Its first application in soil mechanics was introduced by Kassif & Ben Shalom (1971) and the technique has then been subsequently refined by Cui & Delage (1996), and at Imperial College by Dineen (1997), Cunningham (2000), Colmenares (2002), and Monroy (2005). Figure 2.9 illustrates schematically the apparatus used by Colmenares (2002). The osmotic system has an advantage that the tests can be carried out at atmospheric pressure.

The osmotic system involves the use of a semi-permeable membrane, which separates a soil sample and a solution of substance with a large molecular weight (normally Polyethylene Glycol, PEG). The semi-permeable membrane is permeable to water but not to the PEG molecules. By osmosis, the soil water will flow across the semi-permeable membrane, until the suction in the soil equals the osmotic potential of the PEG solution, which is a function of its molar concentration (the Van’t Hoff equation). The PEG solution is continuously circulated across the membrane in order to avoid dilution of the PEG. One of the problems encountered in using the osmotic system is the degradation of the semi-permeable membrane with time, which leads to a gradual reduction in suction. There is thus no guarantee that the suction calculated using the concentration of the PEG will correspond to the actual suction in the sample. Dineen (1997) was the first to employ an independent suction measurement to investigate the performance of the semi-permeable membrane and found the Van’t Hoff equation
overpredicted the actual suction in the sample. An independent suction measurement is therefore necessary if reliable test results are to be obtained from the osmotic system.

Subsequent researchers at Imperial College, Cunningham (2000) and Colmenares (2002) succeeded in identifying an appropriate semi-permeable membrane, which is resistant to biodegradation and also a suitable PEG solution. This combination enables a constant suction of 750kPa to be sustained for two weeks. Nevertheless, in an attempt to use the osmotic system with a mixture of sand & bentonite in a stress path oedometer apparatus, Colmenares (2002) found that it was extremely difficult to control the suction in the sample since the contact between the sample and the membrane was very difficult to make due to the granular nature of the material.

Cunningham (2000) also attempted to incorporate the osmotic system into a triaxial apparatus for testing a reconstituted silty clay. The system was, however, not capable of controlling the suction during shearing and suffered a problem of leakage of PEG from the base pedestal. Monroy (2005) subsequently made an improvement to the PEG circulation system and solved the leakage problem by incorporating a reversal flow and stepped control peristaltic pump. A series of osmotically suction-controlled oedometer tests has been successfully carried out by him on compacted London clay.

c) Relative humidity control system
Again this system was originally developed in the field of soil science and later adopted for application in geotechnics by Al Mukhtar et al. (1993). The approach is based on the control of the relative humidity of the air surrounding the soil sample. By this the total suction can be controlled through its relationship with the relative humidity. Normally, this technique is employed for testing soils at high suctions (above 1 MPa). Again, without an independent suction measurement it is very difficult to guarantee the equilibrium of the suction. Recent developments of these systems normally incorporate thermocouple psychrometry for suction measurement. Figure 2.10 illustrates the system used by Blatz & Graham (2003) for testing a high plasticity clay.

d) Air-regulated system
The air-regulated system, developed by Cunningham (2000) at Imperial College, incorporates suction probes for independent suction measurement and makes use of the
regulated flow of dry air across a surface of the soil sample. Figure 2.11 illustrates the schematic layout of the system. The flow of dry-air across the base of the sample causes the sample to dry out, resulting in an increase of suction. The system was used in the tests during loading or shearing stages, in which the suction normally decreases, to bring the suction back to a constant value. The measurement from the suction probe was used when controlling the air flow to maintain the suction constant. The system is, however, not capable of decreasing the suction at constant total stresses. This air-regulated system was used as a platform on which the suction-controlled system in the present research was developed. More details of the system and its subsequent development are given in Chapter 3.

2.3.3 Volume measurement system

There are two groups of techniques that are used for measuring the volume of partly saturated soils, namely global measurements, and local measurements.

a) Global measurement

The principle of this technique involves measuring the change in the volume of the cell fluid surrounding the soil sample. It was first developed by Bishop & Donald (1961) and incorporated the use of a double walled cell. The apparatus is shown in Figure 2.8. The use of a double walled cell was introduced in order to avoid the error in measurement caused by deformation of the cell chamber upon application of the cell pressure. Bishop & Donald employed an optical technique with a mercury-water interface for the volume measurement. A similar system has also been used by Cui & Delage (1996). The system has been further improved by Wheeler (1986), who incorporated the automatic volume measurement system, using the Imperial College volume gauge and an automatic burette.

The double walled system still, however, suffered from two other limitations. Firstly, the Perspex, normally used for constructing the inner cell, is porous and adsorbs water. A testing procedure of saturating the perspex cell under water when not in use can help minimise the error (Sivakumar, 1993). In addition, the magnitude of the error can be quantified by calibrating the system over different cell pressures without the sample present. Nevertheless, the hysteresis and creep involved in the water absorption of perspex give rise to uncertainties in the correction, especially under loading reversals.
Secondly, temperature fluctuations can cause the cell to expand or contract. The use of a steel cell, with the advantage of no water absorption, would be likely to suffer more experimental error from thermal variation.

In general, global systems still lack accuracy in measurement, especially at small strains due to the aforementioned sources of errors. The degree of accuracy nevertheless improves as the volumetric strain becomes larger.

b) Local measurement

Local volume measurement seeks to measure the strains at different locations on the samples. The technique involves either contact or non-contact methods.

The contact method involves attaching strain-measuring devices directly onto the sample. The devices monitor directly the strain between two fixed points. Examples of these devices are the electrolevel inclinometer for axial strain measurement used by Burland & Symes (1982), and the LVDT radial strain device, used by Maswowe (1985) and Klotz & Coop (2002). Since these devices only measure the strain between two fixed points, an assumption needs to be made regarding the shape of the sample during loading and shearing. Either a right cylinder or a barrelling shape can be assumed (e.g. Germaine & Ladd, 1988 and Klotz & Coop). The triaxial apparatus used in this research employed the contact method. The main advantage of the use of local strain devices is the accuracy of stiffness measurements in the small strain range (e.g. Burland, 1989).

The non-contact method has an advantage over the contact method in that there is no influence of the presence of the strain measuring devices on the sample. This technique has been employed, for example in the hollow cylinder apparatus, by Rolo (2003), who used proximity transducers for radial deformation measurements. Other techniques also exist, which are capable of profiling the shape of the sample along its height. For example, Alonso et al. (1995) made use of a laser beam positioned on a travelling frame to measure the shape of the sample in the triaxial apparatus. In this case, there was no need to make any assumption about the change of shape of the sample.
2.3.4 Water content measurement systems

A knowledge of the change in the degree of saturation of the sample during the test is of great importance for unsaturated soil testing. This is especially so because many recent constitutive models incorporate the degree of saturation as a component in the stress variables, as will be explained in Section 2.4.5. In order to determine the degree of saturation, the moisture content is required in addition to the volume change.

The soil water content measuring system, employed in the apparatus with the axis-translation suction-control system, normally involves either the Imperial College type volume gauge or a burette. As discussed earlier, in using these systems, there is a need to flush the system regularly to get rid of bubbles resulting from air diffusion through the drainage lines. For the apparatus that employs the osmotic suction-control system, Dineen (1997) developed a system for measuring the change in soil water, which consists of an electronic balance and a special evaporation control scheme as shown in Figure 2.9. The use of a glass dish and silicon oil floating on the PEG solution helps minimise the evaporation from the PEG solution.

In the triaxial apparatus employing the air-regulated suction control system developed by Cunningham (2000), no system existed for measuring the moisture content change in the sample during drying. Only the final and initial water contents were known in the tests.

2.4 Mechanical behaviour of partly saturated soils and constitutive models

The mechanical behaviour of partly saturated soils and their modelling reported in the literature will be briefly summarised, with particular emphasis on the behaviour of collapsible soils.

2.4.1 Stress variables

In describing the constitutive behaviour of partly saturated soils, the selection of appropriate stress variables is of great importance. Historically, three types of stress variable have been proposed, namely the single effective stress, the two stress state variables, and more recently the two modified stress state variables.
a) *Single effective stress*

In investigating unsaturated soil behaviour, early researchers attempted to combine the total stress, pore air pressure, and pore water pressure within a single effective stress variable. The most widely used relationship for the effective stress, $\sigma'$, was that suggested by Bishop (1959) as follows;

$$\sigma' = (\sigma - u_a) + \chi(u_o - u_w)$$  \hspace{1cm} (2.4)

where $\sigma$ is the total stress, and $\chi$ is a function of degree of saturation, $S_r$, varying from zero to one for 0 to 100% $S_r$ respectively. The value of $\chi$ is estimated from shearing tests, by assuming the validity of the principle of effective stress.

However, later studies, notably that by Jenning & Burland (1962), have shown that a single effective stress variable is not capable of completely describing partly saturated soil behaviour, especially the volumetric behaviour. Consider, for example, the case of silt samples loaded under oedometric conditions in Figure 2.12. The open points are for air-dried silt and the full points for identical sample soaked under zero stress. One of the air-dried samples was soaked under constant load of 400kPa, and subsequently ‘collapsed’ onto the fully saturated line at A. For another sample, soaking took place while gradually reducing the vertical load to maintain an approximately constant void ratio. The second sample reached its saturation at point B. According to the principle of effective stress, as the sample is soaked, resulting in a reduction in suction and effective stress, the samples would have been expected to swell. On the contrary, the first sample underwent ‘collapse’ to point A and the void ratio of the second sample remained unchanged at B.

The difficulties in using the single effective stress approach for describing partly saturated behaviour can be explained by considering the influence of suction on the interparticle forces. As described by Burland and Ridley (1996) using the grain column analogy shown in Figure 2.13, the influence of a suction acting through the menisci at the contact points between soil particles is to provide a stabilising effect, whereas the influence of a boundary stress is to cause grain slip. The two stress systems give rise to entirely separate mechanical effects and thus should be considered separately.
b) Two stress state variable
After the realisation of the limitation of the principle of effective stress for partly saturated soils, subsequent interpretation of experimental results and modelling were carried out in terms of separate stress variables, namely the net stress \((\sigma - u_a)\) and the matrix suction \((u_a - u_w)\) (e.g., Coleman, 1962, Bishop & Blight, 1963, and Matyas & Radhakrishna, 1968). Fredlund & Morgenstern (1977) provided a theoretical basis for this approach based on multiphase continuum mechanics, which was validated experimentally through a series of ‘null’ tests.

The advantage of using \((\sigma - u_a)\) and \((u_a - u_w)\) as the two stress state variables over the use of other two stress combinations, namely a) \((\sigma - u_w) & (u_a - u_w)\), and b) \((\sigma - u_a) & (\sigma - u_w)\), has been discussed by Wheeler & Karube (1996). The variables \((\sigma - u_a)\) and \((u_a - u_w)\) have been by far the most widely used in the interpretation of experimental results and constitutive modelling.

c) Two modified stress state variables incorporating the degree of saturation
Recently, more consideration has been given to the roles of both suction and degree of saturation in formulating constitutive models for partly saturated soils, especially to take into account the influence of hysteresis (e.g. Kohgo et al., 1993, Kato et al., 1995, Wheeler et al., 2003, and Gallipoli et al., 2003a). As discussed in Section 2.2.2, due to hysteresis, two samples of the same soil can be at the same suction, yet different degrees of saturation. The way in which the suction of these two samples contributes to the mechanical behaviour would certainly be different.

Figure 2.14 shows a schematic representation of soil water within an idealised soil skeleton. Water within a partly saturated soil can be considered as being distributed among the bulk water within water-filled voids and meniscus water at the inter-particle contacts around air-filled voids. The adsorbed water has been specified as part of the soil skeleton by Karube & Kawai (2001). The general idea of this approach is to separate the mechanical influence of the suction operating at different structural levels.
The suction within the bulk water is assumed to induce an equivalent isotropic confining stress, whereas the suction within the meniscus water is assumed to contribute to the bonding effects resulting in an increase in stiffness and yield stress. As a result, the first stress variable, accounting for the influence of net stress and bulk water suction, is normally of the form \((\sigma - u_a) + f(u_a - u_m)\), which is similar to that proposed by Bishop (1959). The second variable, accounting for the bonding effect of suction, is normally a function of suction, degree of saturation and porosity, \(f(S_v, n)\). Inevitably, in identifying the two variables, some assumptions are required, involved with specifying the distribution of meniscus water and bulk water, through the degree of saturation and porosity.

Wheeler & Karube (1996) discussed the justification of using the more complicated stress variables. There are some disadvantages that might arise from using this approach. Firstly, it would be more difficult for the engineer to think in terms of the new stress variables even when describing a relatively simple stress path, for example drying/wetting under constant applied load. Secondly, it would be more difficult to devise simple experiments to obtain the model parameters. Only if the new stress variables have a strong physical significance, resulting in considerable improvement in modelling capacity and more simplicity in stress-strain relationships, would the use of new stress variables be justified.

In the following sections, constitutive models will be discussed, which explain the mechanical behaviour of partly saturated soils in terms of the two basic stress variables, net stress and matrix suction. In Section 2.4.6, the constitutive models formulated using the more complex stress variables will then subsequently be considered.

2.4.2 Failure envelopes

Fredlund et al. (1978) proposed that the shear strength of partly saturated soils, \(\tau\), can be expressed as an extension of the Mohr-Coulomb equation for fully saturated soils, as followed:

\[
\tau = c' + (\sigma - u_a) \cdot \tan \phi' + (u_a - u_m) \cdot \tan \phi^b
\]  

(2.5)
where \( c' \) and \( \phi' \) are the effective cohesion intercept and friction angle for the soil in a fully saturated state, and \( \phi^b \) is the friction angle with respect to changes in suction. When plotted in the space \( \tau, (\sigma - u_a), (u_a - u_w) \), the relationship takes the form of a surface as shown in Figure 2.15.

Early researchers had long been aware of the non-linearity of the relationship between the shear strength and suction. Bishop et al. (1960) suggested that the \( \chi \) factor of a soil, in Equation 2.4, is dependent upon a number of factors, including the degree of saturation, the moisture hysteretic state and the stress conditions. Subsequent investigators (e.g. Escario & Saez, 1986, and Gan et al., 1988) showed that the angle \( \phi^b \) in Equation 2.5 was not constant but changed with suction and could even become negative at high suctions, resulting in a decrease in the shear strength with suction.

The non-linearity of the shear strength-suction relationship can be explained qualitatively by a physical argument. For suctions below the air entry value, the degree of saturation is approximately unity and the influence of suction is equivalent to the applied stress. In this region, \( \phi^b \) is equal to \( \phi' \). However, as the soil desaturates, the wetted contact area around the soil grains decreases and the contribution of suction to the shear strength reduces, resulting in a decrease in \( \phi^b \) with suction. Gan & Fredlund (1996) observed that the reduction in \( \phi^b \) with suction is also influenced by dilation of the sample during shearing. For samples with higher dilation, such as those tested at low confining pressures, the reduction of \( \phi^b \) with suction tends to be greater than those with lower dilation, for example tested at high confining pressure. Figure 2.16 illustrates this observation schematically in comparison with the corresponding SWRC.

The angle \( \phi' \) is normally assumed to be constant with suction. Escario & Juca (1989) showed that this assumption is approximately valid for most soils. Ng et al. (2000) also found that this is true for loosely compacted volcanic fills. Toll (1990) & Toll & Ong (2003), however, found that \( \phi' \) increases with decreasing degree of saturation based on their results from triaxial tests on a lateritic gravel and a residual sandy clay. On the other hand, Cui & Delage (1996) and Maatouk et al. (1995) found that for silty soils, \( \phi' \) decreases with increasing suction. More experimental results are obviously needed in
order to provide any conclusion regarding the dependency of the $\phi'$ variation on soil types.

Various researchers have proposed a number of models to describe the non-linear relationships of shear strength for unsaturated soils (e.g. Vanapalli et al., 1996, Oberg & Sallfors, 1997, and Toll & Ong, 2003). Features of the Soil-Water Retention Curve, including the air entry value and residual suction, have been used in the formulation of these models. Vanapalli & Fredlund (1999) provide a review of various procedures. The two formulations proposed by Vanapalli et al. (1996), which are amongst those most widely used in the literature are:

\[
\tau = \left[ c' + \left( \sigma_n - u_a \right) \tan \phi' \right] + \left[ (u_a - u_w) \left( \Theta^K \left( \tan \phi' \right) \right) \right]
\]  

(2.6a)

\[
\tau = \left[ c' + \left( \sigma_n - u_a \right) \tan \phi' \right] + \left[ (u_a - u_w) \tan \phi' \left( \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \right) \right]
\]  

(2.6b)

where,

- $K =$ fitting parameter used for obtaining a best-fit between the measured and predicted values.
- $\Theta =$ normalized water content, $\theta_w/\theta_s$
- $\theta_w =$ volumetric water content
- $\theta_s =$ saturated volumetric water content
- $\theta_r =$ residual volumetric water content

The variation of $\Theta$ with suction, as well as $\theta_w$ and $\theta_s$ can be obtained from the Soil-Water Retention relationships of the soil. Once again, while this approach is particularly useful, it should be borne in mind that the SWRC for a soil is influenced by the change in pore structure resulting from changes in net stresses and void ratio, as well as by the hydraulic hysteresis. Only the SWRCs of samples, at appropriate confining pressures, having a similar hysteresis or wetting/drying history to the field conditions, should be used.
Toll & Ong (2003) provided a formulation, which describes the shear strength at the critical state, taking a similar form to that proposed by Vanapalli et al. (1996) as shown below:

\[ q = M_a (p - u_a) + M_b (u_a - u_w) \]  
\[ (2.7a) \]

where

\[ M_a = M_b = M_s \]; for \( S_r > S_{r1} \)  
\[ (2.7b) \]

\[ \frac{M_b}{M_s} = \left( \frac{S_r - S_{r2}}{S_{r1} - S_{r2}} \right)^{k_b} \]; for \( S_{r2} < S_r < S_{r1} \)  
\[ (2.7c) \]

\[ \frac{M_a}{M_s} = \left[ \frac{M_a}{M_s} \right]_{\text{max}} - \left[ \left( \frac{M_a}{M_s} \right)_{\text{max}} - 1 \right] \left( \frac{S_r - S_{r2}}{S_{r1} - S_{r2}} \right)^{k_a} \]; for \( S_{r2} < S_r < S_{r1} \)  
\[ (2.7d) \]

\[ M_b = 0 \]; for \( S_r < S_{r2} \)  
\[ (2.7e) \]

where,

\( q \) = deviatoric stress, \( q = (\sigma_v - \sigma_h) \) for the triaxial condition \( (\sigma_v = \sigma_a, \sigma_h = \sigma_r) \)

\( p \) = mean total stress, \( p = (\sigma_v + 2\sigma_h)/3 \) for the triaxial condition

\( M_a \) = critical state stress ratio due to total stress (similar to \( \phi' \))

\( M_b \) = critical state stress ratio due to suction (similar to \( \phi^b \))

Both \( M_a \) and \( M_b \) are functions of the degree of saturation, \( S_r \), which are described by Equations 2.7b, c, d & e where,

\[ M_s \] = critical state stress ratio for fully saturated condition

\( S_{r1}, S_{r2} \) = reference degrees of saturation

\( k_a, k_b \) = fitting parameters due to total stress and suction respectively (different notation to that in Toll & Ong (2003) are used here to distinguish between the two different values of \( k \))

\[ \left( \frac{M_a}{M_b} \right)_{\text{max}} \] = maximum value of \( \frac{M_a}{M_b} \) at \( S_r = S_{r2} \).
The variation of the critical-state stress ratios with degree of saturation for Kiunyu gravel as reported by Toll (1990) is shown in Figure 2.17, together with the fitting curves.

The difference between the Toll & Ong (2003) formulation to that proposed by Vanapalli et al. (1996) is that $M_a$ (or $\phi'$) also varies with the degree of saturation, and hence suction. From the results of triaxial tests on the residual sandy clay, they found that the two reference states ($S_{r1}$ and $S_{r2}$) do not coincide with the fully saturated and residual conditions identified from the traditional SWRC, using a pressure plate apparatus in an unconfined condition, as suggested by Vanapalli et al. (1996). In this respect, the change of pore structure induced during loading and shearing might play a role in changing the SWRC from its unconfined state, but no explicit comment is given in their paper. More details of the Toll & Ong (2003) critical state model, relating the specific volume at the critical state to the suction and net stress, will be given in Section 2.4.4e.

Rassam & Williams (1999) proposed a shear strength model that is similar to the Vanapalli et al. (1996) model but also takes into account the influence of confining stress on the air entry value of the SWRC. Recently, Cabarkapa (2001) introduced a hyperbolic model, which was an extension of the model by Maksimovic (1996), to include partly saturated behaviour. As argued by Cabarkapa et al. (2002), all the parameters required for this model have physical meanings. It is of note that no degree of saturation or volumetric water content is incorporated in this model.

In conclusion, most relationships describing the shear strength of partly saturated soils are based on an extension of a Mohr-Coloumb type failure envelope to include suction as a second variable. The non-linearity of the relationships is taken into account empirically in the models either using some features of the SWRCs or some other curve fitting procedures.
2.4.3 State surface approach for volumetric behaviour

Coleman (1962) and Bishop & Blight (1963) suggested that the volumetric behaviour of partly saturated soils could be defined as a function of separate stress variables, net stress, and matrix suction. Matyas & Radhakrishna (1968) then provided experimental evidence based on isotropic and K₀-compression tests on a silt-clay mixture, defining the state surface, which relates void ratio, and degree of saturation to net stress and suction. According to the state surface concept, void ratio and degree of saturation can be expressed by the following equations,

\[
\begin{align*}
    e &= F(p - u_s, q, s, e_0, S_{ro}) \\
    S_r &= \phi(p - u_s, q, s, e_0, S_{ro})
\end{align*}
\]  

(2.8a)  
(2.8b)

where \(e_0\) and \(S_{ro}\) are the initial void ratio and initial degree of saturation, respectively. The typical state surfaces representing Equations 2.8a & b are shown in Figure 2.18. It can be seen in Figure 2.18a that the void ratio surface is warped indicating that wetting under a low applied load (AA’) induces swelling, while wetting under high load (BB’) involves collapse. The shape of the surface is dependent upon a number of factors including soil type, magnitude of applied pressure, the initial suction, void ratio and degree of saturation. In general, for sandy or silty soils, the void ratio at point A tends to approach that at A’ due to its non-expansiveness. In Figure 2.18b, the relationship between \(S_r\) and suction can be seen to vary with net stress. This is particularly so for compressible soils as discussed in Section 2.2.

The state surface has been found by Matyas & Radhakrishna (1968) to be path independent for tests involving monotonic constant suction loading and wetting, based on the results of tests on a silt/clay mix. The paths identified from loading and wetting tests appear to fall on the same surface in void ratio-stress space. However, the state surface, plotted in degree of saturation-stress space, was not as path-independent, but the discrepancy between different paths was attributed to experimental difficulties.

A number of subsequent researchers, for example, Lloret & Alonso (1985), and Fredlund (1979), then proposed various mathematical expressions for the surfaces. The disadvantage of this concept however is that no distinction is made between recoverable
and irrecoverable strains. In addition, the concept does not take into account the hydraulic hysteresis. The paths involving unloading or increasing in suction would lie below the state surface identified from loading or wetting. The uniqueness of the surface is thus only limited to those stress paths involving monotonic loading and monotonic change in suction.

2.4.4 Elasto-plastic models and the critical state framework

Alonso, Gens & Hight (1987) proposed a conceptual framework based on the theory of hardening plasticity in order to explain both the volumetric and shearing behaviour of partly saturated soils. Regarding the volumetric aspect, the framework is capable of predicting the non-uniqueness of the state surface by introducing several yield surfaces in the space of suction \(s = u_a - u_w\), net stress \(p'' = p - u_a\), and specific volume \((v = e + 1)\).

It is noteworthy that the original framework by Alonso, Gens & Hight (1987) was based on experimental results from tests carried out under both isotropic and \(K_o\) conditions. The net stress, \(p''\), represents either the net isotropic stress for isotropic conditions or the net vertical stress when the soil is under a \(K_o\) condition. In the more general mathematical formulations, such as those subsequently proposed by Alonso et al. (1990) and Wheeler & Sivakumar (1995), \(p''\) would represent the net mean stress, \(p'' = (\sigma_v + 2\sigma_s)/3 - u_a\) for the triaxial condition.

a) Loading-Collapse yield surface

The Loading-Collapse (LC) yield surface, projected onto an \(s - p''\) plot as the yield locus in Figure 2.19, predicts irreversible compressive volumetric strains for any paths moving outside the locus with either an increase of \(p''\) (path L), decrease of \(s\) (path C) or both. Path L represents loading at constant suction, while path C represents wetting at constant net stress. For the region within the yield surface, elastic behaviour is predicted for both changes in suction and stress. Any decrease or increase in suction within the elastic zone induces reversible swelling or shrinkage respectively. The irreversible strain induced by path L will be equal to that induced by path C, which is related to the hardening parameter \(p''_*\). In other words the irreversible strains induced by both paths
will be equivalent to compression along the Normal Compression Line at zero suction (fully saturated) from $p_{o1}^*$ to $p_{o2}^*$.

Figure 2.20 illustrates the volumetric behaviour predicted by the model for different stress paths. Paths L1, L2 and L3 involve loading at different constant suctions. It can be seen in Figure 2.20b that as suction increases, the yield stress also increases. If the sample L3 is unloaded after reaching the maximum $p''$, following path U3, only recoverable swelling will happen. This aspect of behaviour cannot be reproduced by the state surface approach. Similarly, for the wetting paths, C1, C2, & C3, the amount of collapse depends on the applied load. Path C1 only induces swelling since the sample was wetted at a low applied load, inside the elastic zone. Sample C2 experiences first some swelling upon wetting but as the stress path reaches the LC surface, irrecoverable strains develop. Sample C3 wetted at a “high” stress only involves irrecoverable collapse since the path immediately hits the LC surface. The largest collapse is experience by the sample C3.

The shape and position of the LC yield locus (in an $s - p''$ plot) and its evolution with the hardening parameter, $p_o^*$, can be derived from a set of compression curves for different suctions in a $\nu - p''$ plot, as illustrated in Figure 2.21. By tracing an imaginary elastic path through the compression curves, the $(s, p'')$ points at the intersections between the elastic path and the compression curves form the coordinates, through which the LC yield loci are drawn. It can also be seen that as $p''$ increases, a maximum potential collapse is reached, and subsequently the compression curve tends to converge with the fully saturated compression curve (zero suction), resulting in a more vertical LC yield locus in the $s - p''$ plot. This type of behaviour has been observed by numerous researchers (e.g., Maswoswe, 1985, Wheeler & Sivakumar, 1995 and Sivakumar & Wheeler, 2000). The results of isotropic compression tests on Speswhite kaolin samples compacted dry-of-optimum, reported by Sivakumar & Wheeler (2000) are shown in Figure 2.22 as an example.

A number of mathematical formulations describing the Loading-Collapse yield surface in the isotropic state have been proposed by a number of researchers, including Alonso
different forms of the LC yield surface are shown in Figure 2.23, plotted as a series of normal compression lines at different suctions. Figure 2.23a shows the formulation by Alonso et al. (1990). In this formulation, the compressibility gradient $\lambda(s)$ is assumed to decrease with increase in suction. In addition, it is assumed that at the reference pressure, $p_c$, the loading-collapse yield locus is a straight vertical line in the $s - p''$ plane.

According to the Alonso et al. (1990) model, the potential collapse on wetting would increase indefinitely with increasing net stress. This formulation is thus not capable of modelling the maximum potential collapse, as shown in Figure 2.21, and only valid for a range of applied net stress that is dependent upon the type of material and value of suction involved. Josa et al. (1992) subsequently extended this model to account for the maximum collapse for the larger stress range as shown in Figure 2.23b. Georgiadis (2003) made some further refinements to the model. Nevertheless, both models assumed the existence of $p^c$.

Wheeler & Sivakumar (1995) suggested an alternative formulation as shown in Figure 2.23c. Virgin compression is assumed to take place only when the applied stress, $p''$, is greater than the point of maximum potential collapse. The gradient of the normal compression line, $\lambda(s)$, then increases with increasing suction. In general, it appears that the Wheeler & Sivakumar (1995) approach is much easier to be used in practice, since the yield point would coincide with the point of maximum potential collapse. As in the discussion by de Campos and Vargas of Alonso et al. (1991), it is generally difficult to define precisely the yield point, according to Alonso et al. (1990) model, for the stress range below the point of maximum potential collapse, due to the high stiffness of the sample. The location of the loading-collapse yield locus in an $s - p''$ plane would then depend significantly on the method used to define the yield stress.

A fundamental assumption associated with the loading-collapse yield surface is that the plastic compression due to collapse on wetting and that due to loading are of the same process and thus can be described by a single unique surface. However, this assumption has been validated only based on a limited number of experiments by
Wheeler & Sivakumar (1995) on compacted samples of Speswhite Kaolin. Considering that this type of model has been increasingly in use for analysis of boundary-value problems (e.g. Gallipoli, 2000 & Georgiadis, 2003), there is an urgent need for further validation of this important assumption.

b) Suction-Increase yield surface
Alonso, Gens & Hight (1987) suggested that, for a given soil and a given applied load, an increase in suction can induce irreversible volumetric strains and this can be taken into account by a Suction Increase (SI) yield surface. Figure 2.24 shows the SI yield surface as formulated by Alonso et al. (1990), which is assumed to correspond to the past maximum suction experienced by the sample.

It should be noted that the linear relationship between $\nu$ and $\ln s$ as shown in Figure 2.24a is only realistic over a range of suctions, depending on the degree of saturation. As the degree of saturation decreases and approaches the residual state, the specific volume will converge to the value corresponding to the air-dried state. An example of this type of behaviour is shown in Figure 2.25 for a reconstituted silty clay tested by Cunningham (2000). In this respect, a framework such as that proposed by Toll (1988) & Toll (1996) may be more appropriate.

In general, soils having open fabrics associated with a well developed ‘granular’ assemblage, such as compacted soils dry-of-optimum or collapsible natural soils, experience almost negligible shrinkage upon drying (Alonso et al., 1987). For these soils, the SI yield surface might not be especially relevant in the description of their volume change upon drying.

c) Expansive soil model: double structure formulation
The behaviour of expansive soils is strongly path-dependent and cannot be fully represented by the Alonso et al. (1990) type model. It is recognized that within the ‘elastic zone’ assumed in the Alonso et al. (1990) model (Figure 2.24b), significant irreversible strains can occur. Figure 2.26 shows the suction-controlled oedometer test results for a highly expansive clay, presented by Alonso et al. (1995). Accumulated shrinkage can be observed over a number of wetting-drying cycles.
In order to model this aspect of expansive soils, Gens & Alonso (1992) introduced the double-structure concept, as an extension to the original Alonso et al. (1990) model. This concept is based on an idealization that the structure of the clay soils can be divided into two levels, namely macro- and micro-structural levels. Figure 2.27 shows schematically two kinds of micro-fabrics of compacted clays. The first type shown in Figure 2.27a consists of a clay matrix predominantly constituted by an elementary arrangement of clay platelets. The second type, shown in Figure 2.27b, consists of two structural levels; the micro-structural level, associated with the elementary particle arrangement and intra-aggregate pores, and the macro-structural level, associated with the ‘packet’, granular grain, and inter-aggregate pores. Gens & Alonso (1992) argued that the micro-fabric of the second type was very widespread both in natural and compacted soils. A detailed review of soil fabric will be presented in Chapter 6.

According to the formulation of Gens & Alonso (1992), the clay ‘packets’ are assumed to be fully saturated and their volume change can be characterized by the two yield surfaces (SI and SD) as shown in Figure 2.28. The behaviour at the macro-structural level is then assumed to be characterized by the Loading-Collapse (LC) yield curve as in the original Alonso et al. (1990) model. The interactions between the volumetric strains within the two structural levels are defined as the two functions, $f_D$ and $f_I$, as shown in Figure 2.29.

Nevertheless, recent experimental results (e.g. Sharma, 1998) suggest that non-expansive clays such as Kaolin also exhibit irreversible compression during wetting and drying cycles as shown in Figure 2.26. Wheeler et al. (2003) argued that this phenomenon was not linked to the presence of highly expansive clay minerals and could be explained by taking into account the influence of degree of saturation and hydraulic hysteresis in their elasto-plastic model. The model by Wheeler et al. (2003) will be explained later in Section 2.4.5.

Finally, as pointed out by Lloret et al. (2003), the main advantage of the double-structure elasto-plastic model was that the two levels of structure could be taken into account and the variables associated with each level could be followed throughout the tests. However, the predictive capability of the model has not been tested fully. The
model has been mainly used as a tool to gain a better understanding of the soil behaviour and of the mechanisms underlying it.

d) Extension to $q - p'' - s$ space

The LC and SI yield surfaces explained earlier may be extended to the triaxial stress state in $q - p'' - s$ space as shown in Figure 2.30. As a first suggestion, Alonso et al. (1990) employed an elliptical shape for the yield locus in the constant suction plane, as shown in Figure 2.30a. Each elliptical yield locus for a constant suction is identified by the isotropic yield stress, $p_0$, and the critical state line relating $q$ and $p''$ (or $p$ in the original paper and in Figure 2.30) at the critical state. The major axis of the ellipse goes from the yield point, $p_0$, to the intercept of the critical state with $p''$ axis. The variation of the critical state line in a $q - p''$ plot with suction has been identified in a similar manner to the failure envelope relationship proposed by Fredlund et al. (1978) (Equation 2.5). The critical state stress ratio, $M$, is assumed to be constant and the intercept of the critical state line with suction has been formulated by other researchers, such as Toll (1990), Mataouk et al. (1995), Wheeler & Sivakumar (1995) and Georgiadis (2003), in a similar manner to the discussion in Section 2.4.2.

As is the case for fully saturated soils, the shape of the yield surface for partly saturated soils is influenced by the stress history of the sample as well as the anisotropy of the fabric. Based on their results of suction-controlled triaxial tests on silty soils, Cui & Delage (1996) and Mataouk et al. (1995) found that the yield surface is of an inclined shape, as shown in Figure 2.31. The inclination of the yield surface indicates the influence of anisotropic structure, resulting from the sample preparation method carried out one-dimensionally. To account for this, Georgiadis (2003) introduced the functions for general yield surfaces, proposed by Lagioia et al. (1996) for saturated soils, to include a wide range of shapes.

Within the yield surface, elasticity is assumed and the deviatoric and volumetric strains can be calculated by the elastic stiffnesses. Upon yielding, flow rules need to be
specified in order to calculate the different components of strain. Alonso et al. (1990)
assumed non-associated flow rules in order to improve the accuracy of prediction for
\(K_s\)-compression. Wheeler & Sivakumar (1995) however found that the associated flow
rule gave reasonably good fit to their experimental data on compacted kaolin. Again this
is believed to depend on the type of material tested and its inherent structure.

e) Critical state concept for partly saturated soils
As explained earlier, the Critical State Line is employed in identifying the expansion of
the LC yield surface with suction in \(q - p' - s\) space. Firstly, it should be recalled that
in the case of fully saturated soils, at the critical state the shear strain continues
indefinitely at a constant volume and a constant \(q/p'\) ratio, where \(p'\) is the mean
effective stress. This is indicated by constant values of pore water pressure, deviatoric
stress and volumetric strain at the end of shearing.

In testing unsaturated soils, however, various researchers, including Cunningham et al.
(2003), Toll & Ong (2003) and Wheeler & Sivakumar (2000), found that at high
suctions, the samples normally tend to fail through the development of distinct shear
surfaces and it is difficult to identify the true critical states. This bears a striking
similarity to the problem of identifying critical states for bonded soils or stiff soils ‘dry’
of critical state, as reported for example by Atkinson (2000) and Georgiannou &

In terms of volumetric behaviour at the critical state, various approaches exist in
expressing the Critical State Lines for different suctions in the \((v - p'')\) plane.
According to the Alonso et al. (1990) model, the Critical State Lines can be calculated
directly from other parameters of the models, including the Normal Compression Lines
at different suctions, the variation of shear strength with suction \(k\), and the assumed
shape of the yield surface. Figure 2.32 shows an example of the Critical State Line
plotted together with the Normal Compression Line at the same suction, as assumed by

Nevertheless, Wheeler & Sivakumar (1995) found that formulation of the critical state
used by Alonso et al. (1990) did not give a good representation of actual experimental
results. They suggested that the Critical State Lines should be expressed separately as follows;

\[ \nu = \Gamma_{at}(s) - \phi(s) \ln \left( \frac{p''}{p_{atm}} \right) \]  

(2.9)

where \( \Gamma_{at}(s) \) and \( \phi(s) \) are functions of suction. Figure 2.33 shows as an example the Critical State Lines as observed from suction-controlled triaxial tests on compacted kaolin.

Toll (1990) and Toll & Ong (2003) suggested that the critical states for unsaturated soils, in terms of volume change, should be expressed as follows:

\[ \nu = \Gamma_{ba} - \lambda_a \ln(p'') - \lambda_h \ln(s) \]  

(2.10a)

\[ \Gamma_{ab} = 1 + \frac{\Gamma_s - 1}{S_r} \]  

(2.10b)

where \( \Gamma_s \) is the intercept of the Critical State Line for the saturated condition and the parameters \( \lambda_a \) and \( \lambda_h \) are functions of degree of saturation, \( S_r \). Figure 2.34 shows the variation of the parameters \( \lambda_a \) and \( \lambda_h \) with degree of saturation for compacted Kiunyu gravel.

\( f) \) Shear stiffness

An increase in suction can give rise to an increase in the shear stiffness of the partly saturated soils. Figure 2.35 shows the variation of secant stiffness at 0.01\% strain level with suction and net mean stress, during triaxial compression shearing for a reconstituted silty clay from Cunningham (2000). It is expected that, as is the case for the shear strength relationship, the stiffness cannot increase indefinitely with suction and its relationship with suction would be non-linear. Nevertheless, experimental evidence for the change of stiffness with suction is still relatively rare.
h) Incorporation of degree of saturation

For fully saturated soils, the critical state concept allows an estimation of the change in pore water pressure experienced by the soil during undrained shearing or an estimation of the change in volume during drained loading (Schofield & Wroth, 1968). However, since the soil volume and water content are not coupled for degrees of saturation less than 100%, this special feature of the critical state concept is not usable for partly saturated soils. In order to predict the change in suction during the undrained (or constant water content) loading of partly saturated soils, a form of relationship between the stress state and the water content, or another expression of the state of wetness of the soil, such as degree of saturation, is required.

Alonso et al. (1990) first proposed the concept of state surface (Section 2.4.3), which relates the stress states \((p'',q,s)\) to the degree of saturation, or water content, to be incorporated in the elasto-plastic framework. As discussed in Section 2.4.3, this approach cannot distinguish between recoverable and irrecoverable strains, and is thus only limited to certain stress paths.

Wheeler & Sivakumar (1993) and Wheeler (1996) suggested an alternative approach in which the specific water content of the soil (defined as \(\nu_w = S_r \cdot \nu\)) varies with suction and the yield stress in an elasto-plastic manner. Gallipoli, Wheeler & Karstunen (2003b) further simplified the approach by using an improved form of the SWRC relationships. They suggested the general hypothesis that, in the absence of hydraulic hysteresis, which is related to the ink bottle effect and changing wetting angle, for a given soil, there is a unique relationship between degree of saturation, \(S_r\), suction, \(s\) and specific volume, \(\nu\):

\[
S_r = S_r(s,\nu) \quad (2.11)
\]

A number of simplifying assumptions are involved in Equation (2.11). Firstly, any influence of shear strains on the degree of saturation is ignored. Secondly, no distinction is made between the influence of elastic volumetric strains and plastic volumetric strains. In practice these two components of strains are caused by different physical processes and consequently might cause different changes in void geometries. In
addition, the influence of hydraulic hysteresis, related to the ink bottle effect and changing wetting angle, is also ignored.

This hypothesis has been shown to be approximately correct based on the results of tests on compacted Speswhite kaolin by Sivakumar (1993). Figure 2.36 shows the surface and contour line for Equation 2.11, based on the test results. The experiments involved a number of stress paths including isotropic normal compression (shown as dashed lines in Figure 2.36a) and shearing to critical state (shown as data points in Figure 2.36a). The explicit mathematical form of Equation 2.11 was proposed as follows:

\[
S_r = \left[ \frac{1}{1 + \psi (\nu - 1)^n S_r} \right]^m
\]

(2.12)

where \(m\), \(n\), \(\phi\) and \(\psi\) are soil constants. This formulation is an extension of the Van Genuchten (1980) equation.

This approach has been incorporated into an elasto-plastic model by Wheeler & Sivakumar (1993) and used to reproduce the experimental results of Sivakumar (1993) and Zakaria (1995), in comparison with the state surface approach by Lloret & Alonso (1985). As shown in Figure 2.37, the new approach was capable of predicting irreversible changes of \(S_r\), experienced by a sample during loading-unloading at a constant suction, whereas the state surface approach was not.

\textit{i) Final comments on the elasto-plastic frameworks}

One of the discrepancies among various elasto-plastic models is in the formulation of the family of NCLs (or LC yield surfaces) at different suctions. The compression curves are normally curved over a wide range of applied stress and converge towards the fully saturated compression line at high applied stresses. However, many elasto-plastic models have been proposed based on test results over a limited range of stress and suction, leading to the assumption of linearity. There is a need for more experimental data from compression tests over a wide range of applied stress and suction.

The inclusion of the degree of saturation into the elasto-plastic framework has recently been a major area of improvement. The Gallipoli et al. (2003) approach appears to be
simple and yet provides a reasonable reproduction of observed results. Nevertheless, a number of simplifying assumptions made in the approach, as described earlier, might limit its usefulness to certain conditions. More experimental data are needed to investigate this aspect further.

2.4.5 Models employing new stress variables

New types of elasto-plastic models that are formulated in terms of two independent modified stress variables have been proposed by Karube & Kawai (2001), Gallipoli et al. (2003a) and Wheeler et al. (2003). The Karube & Kawai (2001) model involves more complex variables than the other two models, associated with multiple surfaces and stress. Based on the simplicity and the promising capacity of the latter two models, they will be discussed in detail here. Both the Gallipoli et al. (2003a) and Wheeler et al. (2003) models are capable of reproducing several important aspects of partly saturated behaviour, which are not satisfactorily described by the previous models employing two conventional stress variables. These aspects include:

a) an irreversible change of void ratio during wetting-drying cycle
b) a dependence of the response during virgin compression at constant suction on the previous history of suction
c) a smooth transition from fully saturated to partly saturated behaviour

Both models are based on the idea of separating the two mechanical effects, experienced by partly saturated soil, which are influenced directly by the distribution of soil water and degree of saturation. As discussed in Section 2.4.1, the first effect, imposed on the soil skeleton and the bulk water of the soil, can be treated as equivalent to the effective stress. The formulation of the first variable of the two models has the same form as follows,

$$ \sigma_{ij}^* = \sigma_{ij} - \left[ S_r u_w + (1 - S_r) u_a \right] \delta_{ij} $$

(2.13)

where $\sigma_{ij}$ is the total stress tensor and $\delta_{ij}$ Kronecker’s delta. The variable $\sigma_{ij}^*$ is called Bishop’s stress by Wheeler et al. (2003) and average skeleton stress by Gallipoli et al.
(2003a). It is equivalent to the original effective stress equation by Bishop (1959) (Equation 2.4), where \( \chi \) equals \( S_r \).

The second effect is due to the stabilising action exerted at the inter-particle contacts of the soil, provided by the water menisci. As explained earlier, the second effect is normally a function of suction, degree of saturation and porosity. The models by Gallipoli et al. (2003a) and Wheeler et al. (2003) use different approaches in taking into account this effect. Nevertheless, both evaluate this effect at a fundamental level based on the same simplified analysis proposed by Fisher (1926). In the following sections, more details of each model and their capacities will be presented separately.

a) **Gallipoli et al. (2003a) model**

The second variable, \( \xi \), proposed by Gallipoli et al. (2003a) to account for the menisci bonding effect, is defined as the product of two factors, the degree of saturation of air \( (1 - S_r) \), and a function of suction, \( f(s) \);

\[
\xi = f(s)(1 - S_r) \tag{2.14}
\]

The function \( f(s) \) accounts for the increase with increasing suction of the stabilising inter-particle force as shown in Figure 2.38, based on the analytical solution by Fisher (1926). It expresses the ratio between the value of the stabilising force at a given suction, \( s \), and the value of the stabilising force at zero suction. The function does not only depend on suction but also on the size of the spheres and the value of the surface tension for water. In their paper, a sphere radius of 1 \( \mu \text{m} \) and a value of the surface tension of water at 20\(^\circ\)C were assumed. The sphere radius of 1 \( \mu \text{m} \) was chosen based on a suggestion by Haines (1925) for materials with the texture of a compacted kaolin. Actual soils however have variable grain and pore size distributions. Consequently, the selected radius of 1 \( \mu \text{m} \) should only be viewed as a factor depending on the average size of pores of the soil.

The factor \( (1 - S_r) \) accounts for the number of water menisci per unit volume of the solid fraction. When the soil is fully saturated, the value of \( (1 - S_r) \) becomes zero and so
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does the variable, $\xi$, and there is no bonding effect. It was assumed that there exists a unique relationship between the number of water menisci per unit volume of the solid fraction and the degree of saturation. This assumption is only rigorously true if the soil is rigid. Moreover, the validity of this definition does not apply to the case of a soil in an extremely dry state, when the water menisci will start to disappear from the particle contacts. Nevertheless, due to the unavailability of experimental data, the extension of the model to cover extremely dry soils has not been attempted.

As discussed earlier, due to the bonding effect of the water menisci at the inter-particle contacts, the void ratio of the unsaturated soil during virgin loading is always greater than the value for the same soil in a fully saturated state under the same loading condition. This leads to another important modelling assumption in that, during virgin loading of an unsaturated soil, the ratio of the void ratio in an unsaturated condition, $e$, to the void ratio in a saturated condition, $e_s$, $(e/e_s)$, is a unique function of the bonding variable, $\xi$, at the same Bishop’s stress.

The validation of this assumption was given using the results of triaxial tests on a compacted Speswhite kaolin by Sivakumar (1993), on a compacted mixture of bentonite and kaolin by Sharma (1998), and on a compacted lateritic gravel by Toll (1990). Figure 2.39 illustrates the unique relationship between $e/e_s$ and the bonding variable $\xi$ together with the Normal Compression Lines at constant suction, plotted against Bishop’s mean stress, for the test results of Sharma (1998). The same unique relationship between $e/e_s$ and the bonding factor, $\xi$, was also observed for data at the critical state. The expression that fits the curves in Figure 2.39b has the following form:

$$
\frac{e}{e_s} = 1 - a \cdot [1 - \exp(b \cdot \xi)]
$$

(2.15)

where $a$ and $b$ are fitting parameters. At a degree of saturation of 100%, the bonding factor becomes zero and the value $e/e_s$ is equal to 1.
The formulation of the elasto-plastic model was proposed in a similar manner to the models by Alonso et al. (1990) and Wheeler & Sivakumar (1995). The model consists of only one Normal Compression state surface, which relates the values of void ratio, \( e \), isotropic Bishop’s stress, \( p'' \), and bonding variable, \( \xi \), during the plastic behaviour of the soil. The Normal Compression state surface is defined as follows:

\[
e(p'', \xi) = \frac{e(\xi)}{e_s} \cdot e_s(p'')
\]

\[
e_s(p'') = N - \lambda \ln p''
\]

where \( e(p'', \xi) \) is the Normal Compression state surface, \( (e/e_s)(\xi) \) is given by Equation (2.15) and \( e_s(p'') \) is the saturated Normal Compression Line. It should be noted that the parameter \( N \) in Equation 2.16b is different to that used by Schofield & Wroth (1968), since Equation 2.16b expresses the void ratio rather than the specific volume. Figure 2.40 shows three examples of Normal Compression Lines that lie on the Normal Compression state surface and correspond to constant values of the bonding variable, \( \xi \). The Normal Compression Lines at constant \( \xi \) are straight lines in the \( e - \ln p'' \) plane and intersect each other at a high stress, \( p'' = \exp(N/\lambda) \), when the void ratio tends to zero. The elastic change of void ratio is assumed to be given by

\[
\Delta e^e = -\kappa \ln \frac{p_f''}{p_i''}
\]

Where \( \kappa \) is the elastic swelling index and \( p_f'' \) and \( p_i'' \) are respectively the final and initial value of the isotropic Bishop’s stress. In this model elastic volumetric strain is independent of the variation of the bonding variable, \( \xi \). In other words, the bonding variable is considered as being only a stabilising factor and, consequently, its change does not cause an overall change in volume.

As shown in Figure 2.40, the yield locus in the \( \xi - p'' \) plane can be derived in the same fashion as for the Alonso et al (1990) model, whereby an imaginary elastic path is
traced traveling from one point on the fully saturated compression line to another on the compression line at different suctions (from points 1 to 2).

The model prediction was also demonstrated in the paper based on the experimental results by Sharma (1998). The degree of saturation measured during the tests was used in calculating the stress parameters. In formulating a more complete model, additional features relating the degree of saturation and the void ratio to suction, as well as the effect of hydraulic hysteresis (e.g. Gallipoli et al., 2003b) have to be incorporated. The paper did not mention how this can be achieved, but the coupling effect between these additional features and the main framework can be expected. In the following, some examples of the model predictions shown in the paper will be presented.

Figure 2.41 shows the stress path followed by the sample during isotropic loading at a constant suction of 100kPa. As the sample yielded, the degree of saturation increased significantly, resulting in a further decrease in the bonding effect, \( \xi \). Figure 2.42 shows the prediction for a wetting-drying cycle of a sample at a constant isotropic net stress of 50kPa. The model correctly predicts the irrecoverable strain, accompanied by drying. This occurs as a result of the increase in degree of saturation on drying due to hysteresis, giving rise to an increase in \( p'' \), which consequently expands the yield locus and causes irrecoverable compression.

b) Wheeler et al. (2003) model

In the Wheeler et al. (2003) model, the second stress variable was chosen based on the work by Houlsby (1997). The second stress variable, termed modified suction, \( s^* \), is the product of porosity and suction as follows,

\[
s^* = ns = n(u_a - u_w)
\]  

(2.18)

The increment of work input, \( dW \), for the simplified stress state of the triaxial tests, per unit volume of unsaturated soil can be written as

\[
dW = p^* \, d\varepsilon_v + q \, d\varepsilon_s - s^* \, dS_r
\]  

(2.19)
where $p^*$ is the Bishop’s mean stress. The decrease of degree of saturation, $-dS_r$, is conjugate to the modified suction, $s^*$.

The major assumption in this model lies in the consideration of the variation of the stabilising forces with suction at the interparticle contacts as analysed by Fisher (1926). As shown in Figure 2.38, the increase in this force with suction is only 50% for suction varying from zero to infinity regardless of sphere size. It was consequently argued that, to a first approximation, the stabilizing force for each meniscus could be assumed to be independent of suction. Consequently, the overall stabilizing effect would be more influenced by the number of inter-particle contacts that are affected by meniscus water lenses than the value of suction. It was then argued, as in the Gallipoli et al. (2003a) model, that the number of meniscus lenses could be related to the degree of saturation.

The LC yield curve is therefore proposed as a straight vertical line in the $s^* - p^*$ plane as shown in Figure 2.43a. During the loading path ABC at constant modified suction, plastic volumetric strains develop when the yield curve is moved from B to C. This LC yield curve is coupled with the movement of the other two yield curves, Suction-Increase and Suction-Decrease, as will be explained below.

Wheeler et al. model the hydraulic hysteresis in the water retention curve as an elasto-plastic process, as shown in Figure 2.44. Plastic changes of $S_r$ are assumed to occur when the soil state is moving along a primary drying curve or a primary wetting curve. If the soil state plots between the primary drying and wetting curves, elastic changes of $S_r$ are assumed. It is important to realise that the actual hysteresis phenomena of a soil are qualitatively different from plasticity, since they occur at any stress states without implying the violation of a yielding criterion (Gallipoli, 2000). In other words, at the same degree of saturation the scanning curves are not necessarily the same, if the wetting/drying history is different. Some degree of simplification is thus involved in this aspect of the model.

For a drying path starting at A in Figure 2.44, plastic decreases of $S_r$ commence when the primary drying curve is reached at B. Alternatively, on wetting from A, plastic increases of $S_r$ are experienced by the sample on reaching the primary wetting curve at
point D. This behaviour can be represented by the two yield curves, namely the suction increase (SI) yield curve and suction decrease (SD) yield curve, as shown in Figure 2.45a. It can be readily seen that both yield curves are coupled. As the sample is dried from B to C, both SI and SD yield curves are displaced upwards from SI1 to SI2 and from SD1 to SD2, respectively.

In Figure 2.43b, if the drying path DE, followed by the soil, involves movement of the SI yield curve, and thus a plastic decrease in the degree of saturation, more void space will be empty of water, and this will lead to a larger number of meniscus water lenses. The ultimate consequence would be an increase in the stabilising force around particle contacts, which is reflected as an outward movement of the LC yield curve from position LC1 to position LC3. On the contrary, for the wetting path DF involving a plastic increase in the degree of saturation, more void space will be filled with water, leading to a smaller number of meniscus water lenses. This ultimately results in an inward movement of the LC yield curve from position LC1 to position LC4.

The movement of the LC yield curve is accompanied by a plastic reduction in void space, which will also result in a change in the retention behaviour. As suggested in Figure 2.46, plastic compression of the soil will cause an outward movement of the retention curves. The modified suction, $s^*$, at which a plastic change in the degree of saturation occurs, changes from the values of B and D to the values of F and G respectively. As illustrated in Figure 2.45b, the movement of the stress path from A to H causes plastic volumetric strain and consequently results in the coupled movement of both the SI and SD yield curves from position 1 to position 3. The three yield curves, LC, SD, and SI, are plotted together with the conjugate strains, $\varepsilon_r$, and $S_r$ in Figure 2.47. An associated flow rule is applied on all three curves. A possible extension to anisotropic stress space is shown in Figure 2.48. An elliptical yield surface is assumed.

The predictive capability of the model is similar to that of the Gallipoli et al (2003a) model. An additional capability of the Wheeler et al (2003) model is the prediction of a smooth transition from unsaturated behaviour to saturated behaviour. Figure 2.49 illustrates a model prediction of isotropic loading at constant suction. The normal compression line is curved during the path BCEF and converges to the fully saturated
compression line at FG when the degree of saturation reaches 100%. This realistic prediction comes as a result of coupling between the SD and LC yield curves.

The Wheeler et al. (2003) model is complete without any need for an additional formulation describing the variation of the degree of saturation as would be required by the Gallipoli et al. (2003a) model. However, only qualitative predictions have been demonstrated in the paper. A number of assumptions are involved in formulating this model, in particular regarding the vertical LC yield line. The model still requires further experimental validation as well as a practical technique for identifying the model parameters from a simple experiment.

c) Final comments on the Gallipoli et al. (2003a) and Wheeler et al. (2003) models
Both models offer a potential for more realistic modelling of partly saturated soils, involving both relatively simple paths, such as monotinic loading or wetting, and more complicated paths, such as those involving wetting and drying cycles, with less complicated yield surfaces and fewer modelling parameters than would be required if the conventional stress variables were adopted. The main assumption involved in both models relates to the evaluation of the stabilising force at the particle contacts and to the relationship between the number of meniscus water lenses and the degree of saturation. The validation of the model by Gallipoli et al. (2003a) with experimental data suggests that this assumption is realistic, at least over the range of suctions up to 300kPa. More experimental data are needed in order to extend the concept to cover a wider range of suctions and degrees of saturation.

2.5 Current gaps in knowledge and the research objectives
From the literature review, areas of uncertainty in the understanding of the behaviour of collapsible non-expansive unsaturated soils can be summarised as follows.

a) Soil-Water Retention Curves: its hysteresis and variations
The soil-water retention behaviour is very important in understanding partly saturated soil behaviour. However, the hysteresis involved and its dependence on various factors including soil fabric, void ratio, current stress states and stress history make the SWRC one of the most uncertain properties of a soil. This is particularly expected to be so for collapsible soils, whose pore size distribution can be affected by the collapse.
Recently, significant advances have been made in incorporating the relationship between suction, degree of saturation and void ratio, as well as hydraulic hysteresis in the elasto-plastic framework. This significantly improves the capability of models to predict the behaviour associated with more complex stress paths such as wetting and drying cycles. Nevertheless, the relationships proposed (e.g. Gallipoli et al., 2003b) are based on experimental results over a limited range of suctions. At different degrees of saturation, different mechanisms operate in governing the suction of soils. For example, at high suctions, surface adsorption, which is dependent on soil mineralogy, will govern the value of suction. The influence on the SWRC of various factors such as void ratio might therefore be different at different degrees of saturation.

b) Elasto-plastic framework
The behaviour of collapsible unsaturated soils can be described by various elasto-plastic models such as Alonso et al. (1990), Wheeler & Sivakumar (1995) and Josa et al. (1992). These models differ in the formulation of the Loading-Collapse surface. These differences can be explained by the difference in the stress and suction ranges that are of interest and in which supporting experimental data have been used. Normally, the stress and suction ranges reported in the literature for suction-controlled tests using axis-translation technique are small. This is due to limitations associated with the raised air-pressure and the robustness of the apparatus. In addition, as already explained, the uniqueness of the Loading-Collapse (LC) surface, due to wetting and loading paths, still needs to be examined more thoroughly.

The new elasto-plastic models using modified stress variables are very promising but there is still a need for experimental results to validate them. Problems associated with the limited stress and suction ranges also exist.

c) Research objectives
The main objectives of the present research were to provide the experimental results to help clarify some of the uncertainties identified above. The main apparatus used in the research was the triaxial apparatus employing the same suction-control system as Cunningham (2000). Further improvement on the system was needed so that wetting
paths could be carried out and so that the moisture content could be monitored during testing.

Since the duration of each test on partly saturated soils is normally lengthy, so a relatively small number of tests could be performed. In parallel with the main triaxial experiments, other experiments, in particular unconfined drying/wetting, and suction-monitored oedometer tests were also carried out to supplement the results from the triaxial tests. Using both the oedometer and triaxial apparatus, attempts were made to perform tests involving both wetting and loading paths, and to cover as wide a range of suctions as possible.

The behaviour of partly saturated soils is often qualitatively explained using some description of their fabric. Many assumptions of the models are also made on the basis of fabric description. A fabric study was therefore also carried out to aide the interpretation of the results.
Figure 2.1 The two phenomena contributing to the matrix suction; a) capillarity; and b) water adsorptions by a clay particle surface (Mitchell, 1993)
Figure 2.2 General description of the soil-water retention curves

Figure 2.3 The two main causes for hysteresis in soils with rigid particles; a) the ink bottle effect, b) the contact angle effect (Hillel, 1998)
Figure 2.4 Hysteresis of the soil-water retention curves for rigid soils (Childs, 1969)

Figure 2.5 Bimodal soil-water retention curve of a pelletized diatomaceous earth material (Burger & Shackelford, 2001)
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Figure 2.6 An Imperial College tensiometer or suction probe (Ridley et al., 2003)

Figure 2.7 Influence of equilibrium time on measurements for filter papers making no contact (Ridley et al., 2003)
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Figure 2.8 Suction-controlled double-walled triaxial cell, using the axis-translation technique (Bishop & Donald, 1960)

Figure 2.9 Osmotically suction-controlled oedometer apparatus developed at Imperial College (Colmenares, 2002)
Figure 2.10 Relative-humidity suction-control system for triaxial apparatus (Blatz & Graham, 2003)

Figure 2.11 Air-regulated suction-control system for triaxial apparatus (Cunningham, 2000)
Figure 2.12 Oedometer compression curves on air dry (open points) and saturated (full points) silt showing the effects of soaking (Burland, 1965)

Boundary forces
UNSTABLE

Contact menisci
STABLE

Figure 2.13 Grain column analogy (Burland & Ridley, 1996)
Figure 2.14 Schematic representation of bulk water and meniscus water within an unsaturated soil (Wheeler & Karube, 1996)

Figure 2.15 Extended Mohr-Coulomb failure envelope (Gan, Fredlund & Rahardjo, 1988)
Figure 2.16 Schematic relationship between soil-water retention curve and shear strength versus matrix suction envelope (Gan & Fredlund, 1996)
Figure 2.17 Variation of critical-state stress ratios with degree of saturation for Kiunyu gravel (Toll & Ong, 2003)

Figure 2.18 State surface concept proposed by Matyas & Radhakrishna (1968) for void ratio and degree of saturation (Wheeler & Karube, 1996)
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Figure 2.19 Loading-Collapse yield locus (Alonso et al., 1987)

Figure 2.20 Deformation under loading and collapse behaviour as predicted by Alonso et al. (1987) model (modified from Alonso et al., 1987)
Figure 2.21 Relationship between compression curves of a collapsible soil for different suctions and yield loci (Alonso et al., 1987)

Figure 2.22 Variation in the specific volume during isotropic compression at constant suctions of kaolin samples compacted dry-of-optimum (Sivakumar & Wheeler, 2000)
**Figure 2.23** Form of Normal Compression Lines assumed by: (a) Alonso et al. (1990); (b) Josa et al. (1992) & Georgiadis (2003); Wheeler & Sivakumar (1995)
Figure 2.24 Suction-Increase yield surface as proposed by Alonso et al. (1990)

Figure 2.25 Unconfined drying/wetting volume change behaviour of a reconstituted silty clay (Cunningham et al. 2003)
Figure 2.26 Wetting-drying cycles performed on Boom clay under oedometric conditions (Alonso et al., 1995)

![Graph showing wetting-drying cycles](image)

$$\sigma_v - u_a = 100 \text{kPa}$$
$$\gamma_d = 14 \text{kN/m}^3$$

Suction, $u_a - u_w$: MPa

Volumetric strain, %

Figure 2.27 Fabric types in a compacted clayey soil. a) Clay matrix predominantly constituted by elementary arrangement of clay platelets; b) Micro-fabric of a clay predominantly made up of aggregations of elementary particle arrangements (Gens & Alonso, 1992)

![Diagram showing fabric types](image)
Figure 2.28 Graphical summary of the double-structure elasto-plastic model for expansive soils (Lloret et al. 2003)

Figure 2.29 Generic interaction functions that indicate the intensity of the interaction between the two structural levels (Lloret et al. 2003)
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Figure 2.30 Yield surfaces in \((q - p'' - s)\) space (\(p\) is equivalent to \(p''\)) (Alonso et al., 1990)
Figure 2.31 Experimental results indicating the inclination of the yield surface: a) Mataouk et al. (1995); b) Cui & Delage (1996)
Figure 2.32 Schematic representation of Normal Compression Line and Critical State Line at the same suction according to Alonso et al. (1990) model (Gallipoli, 2000)

Figure 2.33 Critical states of compacted kaolin for different suctions (Wheeler & Sivakumar, 1995)
Figure 2.34 Variation of critical-state compressibilities with degree of saturation for Kiuunu gravel (Toll & Ong, 2003)

Figure 2.35 Variation of secant stiffness at 0.01% axial strain with suction and mean net stress during triaxial shearing for a reconstituted silty clay (Cunningham, 2000)
Figure 2.36 Surface of the relationship between degree of saturation, suction and specific volume (Gallipoli et al., 2003b)
Figure 2.37 Experimental and predicted variation of $S_r$ during isotropic loading-unloading at constant suction ($s = 100\text{kPa}$) (Gallipoli et al., 2003b)

Figure 2.38 Ratio between inter-particle forces at suction $s$ and at zero suction due to a water meniscus located at the contact between two identical spheres (after Gallipoli et al., 2003a)
Figure 2.39 Experimental results by Sharma (1998) a) Normal Compression Line at constant suction ($p''$ represents Bishop’s mean stress) b) Relationship between ratio $e/e_s$ and bonding factor $\xi$ during isotropic virgin loading at constant suction (Gallipoli et al., 2003a)
Figure 2.40 Derivation of the yield locus in the isotropic plane for the Gallipoli et al. (2003a) model: a) change of void ratio; b) stress path (Gallipoli et al., 2003a)
Figure 2.41 Prediction of Gallipoli et al. (2003a) model for isotropic virgin loading at constant suction of 100kPa. a) change of void ratio; b) stress path (Gallipoli et al., 2003a)
Figure 2.42 Prediction of Gallipoli et al. (2003a) model for a wetting-drying cycle of a sample at a constant isotropic net stress of 50kPa a) change of void ratio; b) stress path (Gallipoli et al., 2003a)
Figure 2.43 LC yield curve for the Wheeler et al. (2003) model: a) direct movement caused by yielding on LC curve; b) coupled movements caused by plastic changes of $S_r$ (Wheeler et al., 2003)
Figure 2.44 Representation of hydraulic hysteresis as an elastic-plastic process by the Wheeler et al. (2003) model (Wheeler et al., 2003)
Figure 2.45 SI and SD yield curves for the Wheeler et al. (2003) model a) direct movements caused by yielding on SI curve; b) coupled movements caused by plastic volumetric strain (Wheeler et al., 2003)
Figure 2.46 Influence of plastic volumetric strain on the primary drying and primary wetting curves for the Wheeler et al. (2003) model (Wheeler et al., 2003)

Figure 2.47 LC, SI and SD yield curves for isotropic stress states for the Wheeler et al. (2003) model (Wheeler et al., 2003)
Figure 2.48 Yield surfaces for anisotropic stress states for the Wheeler et al. (2003) model (Wheeler et al., 2003)
Figure 2.49 Wheeler et al. (2003) model simulation of isotropic loading at constant suction: a) path in modified stress space; b) specific volume; c) degree of saturation (Wheeler et al., 2003)
CHAPTER 3

SELECTION OF MATERIAL AND EXPERIMENTAL PROCEDURES

3.1 Introduction

This chapter describes the selection of the most suitable material for the experimental programme of this research project, together with the basic properties of the chosen material. The sample preparation methods are also presented, which were used for creating the samples for subsequent experiments. The chapter then describes the apparatus, the experimental set up and the testing procedures employed in this research, including those of the suction-controlled triaxial tests and the suction-monitored oedometer tests, as well as the suction measurements employing the suction probe and the filter paper technique. Also highlighted is the improvement made to the existing suction control system used for the triaxial apparatus.

3.2 Material selection

The original objective of the present research was to investigate the mechanical behaviour of a partly saturated natural soil with real engineering problems. The majority of the first six months of the research project was thus spent on identifying a suitable natural soil. Two natural materials were investigated, including a Thai saprolitic sandy clay, and a Greek pumiceous gravelly sand. The saprolitic sandy clay was taken by the author as a block sample from a landslide site in northern Thailand, while several block samples of the pumiceous sand were taken by others from a historical cave that was facing a problem of collapse on the island of Milos, Greece.

A series of unconfined compression shearing tests with suction measurement were performed on samples of both soils at various water contents. The testing procedures followed were similar to those used by Colmenares (1997) and Cunningham (2000). However, due to the friable nature of the Greek sand, some special procedures were required in trimming the samples, involving freezing the samples and the use of a special cutting machine. Since the saprolitic sandy clay was also very fragile and occasionally contained fissures, samples of a square prism shape were tested, instead of
Chapter 3 – Selection of material and experimental procedures

the conventional cylindrical shape, for ease of trimming. For brevity of the thesis, the
details of their testing procedures will not be included here.

Figures 3.1 and 3.2 show respectively the results of the unconfined compression tests
for the saprolitic sandy clay and the pumiceous sand. The peak strength envelopes for
both soils are shown together with the stress paths, followed by the samples during
shearing. The envelopes appear to be non-linear, as would be expected and was
discussed in Section 2.4.2. Nevertheless, the scatter in the results for pumiceous sand is
significant and it is believed to be due to the heterogeneity of the material.

It was finally decided that both materials would not be investigated further for the
following reasons. Firstly, the pumiceous sand was of a relatively coarse nature and thus
not suitable for the suction probe technique for suction measurement. This is because
the sand desaturates quickly at a relatively low suction and the contact between the
probe and the material becomes difficult to achieve for the suction range of 500 to
1000kPa. For suctions less than 100kPa, it is believed that the accuracy of the suction
probe is not as good as other methods, such as conventional tensiometers or the axis-
translation technique. The heterogeneous nature of both the pumiceous sand and the
saprolitic sandy clay also makes their testing results less useful in validating constitutive
models. The problem of obtaining an adequate amount of material made it impracticable
to test the saprolitic sandy clay in a reconstituted state.

The objective of the research was then shifted towards testing an artificial material that
could be obtained readily and was of more repeatable properties. As a continuation from
the research by Cunningham (2000), the artificial mix was chosen of 70% HPF4 silt,
20% Speswhite Kaolin, and 10% London clay, or the so-called Soil A. The preparation
of the samples of Soil A for the main experiments was, however, carried out in a
different way to the method used by Cunningham. Cunningham prepared his samples
from a slurried state, which was consolidated one-dimensionally to a vertical effective
stress of 200kPa in an oedometer apparatus. The reconstituted samples were
subsequently dried to different suctions before testing. His test results, however,
demonstrated that the reconstituted samples did not exhibit the collapse-upon-wetting
phenomenon within the stress range investigated, which was up to 1000kPa mean net
stress. This makes the test results unsuitable for validating many aspects of the elasto-plastic models explained earlier in Section 2.4.4.

It has been recognised that clayey materials compacted dry-of-optimum demonstrate collapse-upon-wetting behaviour (Barden et al., 1973). Therefore, it was decided that samples of Soil A compacted dry-of-optimum would be used as the material for the main testing programme in this research. This choice of material also enables a comparison to be made between the behaviours of Soil A with the reconstituted and compacted fabrics. The selection of the initial properties of the compacted Soil A will be explained in Section 4.3.

The index properties and the gradation curves of the Soil A used in the present research were determined and compared with those of the Soil A used by Cunningham (2000). The test results from both studies are shown in Figure 3.3 and Table 3.1. There was a concern regarding the difference in the grading of the compacted and the reconstituted Soil A, since their preparation methods are slightly different. As will be explained in Section 3.3, the preparation of the compacted samples involves additional oven-drying at 70°C and a grinding stage. Figure 3.3, however, shows that both reconstituted and compacted samples have very similar gradation curves. In addition, it is noteworthy that the source of the London clay used in the present studies is not certain and may not come from the same site as that of Cunningham, but since the overall properties of both Soils A are very similar, they are considered as being approximately identical. Nevertheless, the Atterberg limits of the powder that was oven-dried at 70°C were not determined.

3.3 Sample preparation methods for Soil A

In the present studies, two main types of sample were tested, namely reconstituted samples, the same as those tested by Cunningham (2000), and statically compacted samples. Figure 3.4 shows diagrammatically the procedures followed in the preparation of both types of sample. In the following, the procedures used in each stage of the preparation are explained in more detail.
3.3.1 Preparation of a slurry at w/c of 1.5 Liquid Limit

The initial stage of preparation of both types of sample involves making a slurry of Soil A at a water content of 1.5 Liquid Limit. To start with, the relative proportions, by weight, of the dry powder were prepared of 70% HPF silt, 20% Kaolin, and 10% London clay. The HPF silt and Kaolin powder were obtained commercially from Hepworth Mineral and Chemicals Ltd, and Imery Minerals Ltd, respectively. The London clay powder was prepared by grinding the clay lumps, which were subsequently passed through a No.40 sieve and air-dried before use.

The relative proportions of powder of the three soils were then mixed thoroughly by hand, using spatulas. The powder mixture was added slowly to the appropriate amount of de-ionised water that would make a slurry at a water content of 1.5LL. The slurry was regularly stirred while progressively adding the powder mixture. The slurry was then transferred to the mechanical stirrer set at its slowest speed and stirred for about 3 hours. After the stirring, the slurry was ready for the next stage.

3.3.2 Preparation of the reconstituted samples

The procedure in the following was identical to that used by Cunningham (2000), Dineen (1997) and Standing (1997). After stirring, the slurry was fully hydrated in an air-tight container for at least 3 weeks before subsequent consolidation. Several samples for water content measurement were also taken both before and after the hydration period. After the hydration, the slurry was removed from the air-tight container. It was then stirred slowly under a vacuum for at least 30 minutes to ensure the slurry was deaired. Meanwhile, the oedometer apparatus was prepared for testing. In the preparation of the cake sample, a 9” (228.6 mm) oedometer apparatus was used. Otherwise, a 75mm diameter oedometer was used. The porous stone was first boiled in de-ionised water for at least 30 minutes to ensure full saturation. A disc of filter paper was prepared for each porous stone. This was to cover the loading face of the stone, preventing the soil slurry from clogging the drainage paths. The inside wall of the oedometer cell was then lightly greased. For the 9” oedometer sample, a water level of about 30mm was left standing before placing the slurry into the cell.

After it had been de-aired, the slurry was spooned slowly into the cell with care to avoid air entrapment. While placing the slurry, the cell was tapped periodically to ensure even
distribution of the slurry in the cell. Once the slurry was filled to a sufficient height, allowing for the room for a top cap, the cell was covered with cling film to prevent the slurry from drying. In the case of the 75mm oedometer cell, the sample assembly was left overnight to allow for self-weight consolidation of the slurry. For the 9” oedometer apparatus (for making the large cake sample), a period of 2 days was allowed and the drainage line from the base platen was also connected to a reservoir of de-ionised water with the water level 1m below the cell. This was to apply a suction of 10kPa to the base of the sample to accelerate the initial consolidation.

After the initial consolidation was complete, the de-aired top cap with a filter paper was positioned. The cell was left for further 24 hours before loading started. De-aired water was frequently added to keep the sample fully saturated at all times. The sample was then loaded incrementally up to a maximum vertical effective stress of 200kPa. A period of about 48 hours was allowed between each load increment to ensure full consolidation. Upon unloading, a period of 24 hours was allowed between each decrement. At the end of unloading the sample was left with only a suction of 10kPa through the drainage line for 48 hours to allow full swelling. The cake sample from the 9” oedometer apparatus was subsequently used for preparing the samples for the triaxial tests. After full swelling, the cake sample was carefully extruded from the cell. The reason for allowing the sample to swell was that it would have only a small initial suction (about 10kPa) before subsequent testing. Small off-cuts were taken from the sample for water content measurement. The cake sample was then carefully wrapped in at least three alternating layers of cling film and low melting point wax. The cake sample was then stored in a safe place for future use.

3.3.3 Preparation of the mixture for compacted samples
After the slurry was created as explained in Section 3.3.1, it was removed from the container and oven-dried at 70°C. The dried slurry thus became almost rock solid and was subsequently broken to pieces. These pieces were ground to powder and passed through a No.40 (425µm) sieve. The powder was then transferred back to the oven at 70°C. The properties of the powder created in this way were significantly different to those of powder created by merely mixing together the three dry constituents (silt, kaolin and London clay). For example, at the same water content, the same compaction
effort does not result in the same density for the samples created from the two kinds of powder. The properties of the powder that was initially created from the slurry were closer to those of the reconstituted samples than the powder that was created by dry mixing.

The powder thus created was kept within the oven at 70°C to maintain zero water content before use. In this research project, two compaction water contents of about 10 and 13.5% were chosen. The reasons underlying the selection of these water contents are explained in Section 4.3. Firstly, a fixed amount of dry powder was weighed and spread onto a tray. In this research, 800g of dry powder was always used to prepare the material for two 50mm triaxial samples. An amount of 1kg was used for the preparation of the material for five 3” oedometer samples. Afterwards, de-ionised water was then gradually sprayed directly onto the dry soil while mixing the powder and the water using spatulas. Care was taken to avoid any excessive local wetting and formation of lumps. Normally, 5 g of water was sprayed at a time, alternated with mixing the soil.

Once the weight of water reached the required value, the soil was mixed thoroughly until it appeared uniform. Afterwards, the weight of soil plus water was checked again and water was added to compensate for any evaporation during the first mixing. The soil was then mixed again using spatula for further 5 minutes. Subsequently, the soil was transferred to a resealable plastic bag, ready for passing through a 2.50mm sieve. A small portion of the soil was sieved at a time. The sieved material was transferred to another resealable bag. Once all the soil had been sieved, it was distributed to several bags, each of which had enough material for either one triaxial or oedometer test. Each bag of material was then sealed under vacuum and wrapped within a layer of aluminium foil and a layer of cling film. The seal was reinforced by three alternating layers of wax and cling film. The sealed bag was then labelled and kept for future use within a sealed container away from a direct sunlight or a source of temperature gradient.

3.3.4 Compaction procedure
Static compaction was used in forming the samples for the suction-controlled triaxial and the oedometer tests, as well as for the unconfined drying and wetting tests. In the triaxial tests, the samples had a diameter of 50mm with the height-to-diameter ratio of about 2. In the oedometer and unconfined wetting and drying tests, the samples used
were of a 3” diameter (76.2mm) with a height of about 19mm. The methods used for creating the samples of these two sizes are described below.

**a) Triaxial samples**

For the triaxial samples, the procedure used was similar to that by Sivakumar (1993) and Sharma (1998). Each sample was statically compacted within a compaction mould (Figure 3.5a) up to a certain maximum compaction pressure at a constant rate of displacement of 1.5mm/min. The compaction pressure of 800kPa with the samples’ as-compact ed moisture content of about 10%, created a set of samples that exhibited the collapse-upon-wetting behaviour as will be explained in Section 4.3. Samples compacted at this initial state were used for the main experiments in the triaxial apparatus. An Imperial College type load cell was employed to monitor the pressure during compaction (Figure 3.5b). Before compaction, some silicon oil was lightly wiped around the inside of the brass mould to aide the extrusion of the sample. Each sample was compacted in nine layers within a loading frame. A trial-and-error method was used for identifying the amount of the soil to be added for each layer in order that the final height of the sample was about 100mm. In this study, 37.9g of soil mixture was added for each layer. In between each layer, the surface was scarified before continuing with the next layer, to ensure that the adjacent layers were in a good contact. In addition, the total thickness of the sample was measured at the end of compaction of each layer. It was observed that each soil layer was of approximately the same thickness and thus the density of the final sample was expected to be uniform throughout its height.

Since the compacted samples were of a relatively low moisture content at a low degree of saturation, evaporation from the samples tended to happen relatively quickly. Care had to be taken during the compaction process to avoid any unnecessary evaporation. The plastic bag containing the soil mixture was kept sealed at all times during compaction and only opened when more material was required for the new layer. Upon completing the final (9th) layer, the top cap was maintained in the final position for further 5 minutes to reduce the rebound upon unloading. The sample was then extruded out of the mould using a mechanical press (Figure 3.5c). It was subsequently weighed and its dimensions were measured.
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b) 3” samples
The 3” diameter by 19mm height samples were used for the oedometer tests and unconfined wetting and drying tests. The samples were compacted within an oedometer ring inside a compaction mould, following the same procedure as was used by Maswoswe (1985). As will be explained in Section 4.3, three sets of samples of differing unit weights and moisture contents were used in these tests. The compaction method was the volume control type, as opposed to the stress control type used for the triaxial samples, in order that the required initial properties could be achieved more easily. One set of the 3” samples (Series 7-10) nevertheless had the same initial properties as the 50mm triaxial samples.

The compaction mould used is shown schematically in Figure 3.6. To begin with, the inside of the oedometer ring was lightly lubricated with silicon grease. The weight of the ring and its dimensions were then measured. The ring was subsequently assembled within the mould. The required amount of the soil mixture was weighed and transferred to the mould. Care was taken when placing the soil into the mould, to ensure that an even distribution of material was achieved. The top piston was then closed and the mould was transferred to the Budenberg dead-weight loading frame where the compaction was carried out.

During compaction, load was added incrementally until the gap between the top piston and the mould closed fully. After each increment, the load was sustained for 5 minutes. The gap normally closed after approximately 3 to 4 loading increments. The precise compaction pressure was however not measured. Once the gap had closed, the final load was sustained for a further 10 minutes to reduce any rebound of the material. The ring with the soil sample was then removed from the mould. Some material that protruded beyond the ring face was trimmed using a straight edge or sharp knife. The final dimensions and weight were measured again. For some unconfined drying tests, the samples were also extruded from the ring in order that the ring could be used for further samples. The extrusion was carried out in a mechanical press.
3.4 Suction measurements

As discussed in Chapter 2, two suction measurement techniques were employed in this research project, namely the suction probe and the filter paper technique. In this section, the modifications to the suction probes and the details of the testing procedures for both suction measurement methods will be explained.

3.4.1 The suction probe

a) Modification

The majority of suction probes used in this research were of the design shown in Figure 2.6 by Ridley & Burland (1999). Nevertheless, some problems occurred when a probe of this design was used inside a water-filled triaxial cell under an applied cell pressure. Only 4 triaxial tests had been carried out successfully within a water-filled cell at a cell pressure of 50 kPa before the readings of the probe drifted continuously as a result of water permeating into the back of the probe. This explanation for the drift was based on the observation that the drift reversed after the probe had been taken out of the cell and dried. This trend was consistently observed in a number of tests.

A minor modification was then made to the existing design in an attempt to prevent water from permeating into the interior of the probe. Figure 3.7 shows the three possible weak points, which could have let the water into contact with the strain gauge under a high cell pressure condition. Modifications were therefore made of these three points as shown in Figure 3.8. At point 1, the connection between the diaphragm section and the main body was made using Loctite 601 retainer. The overlapping length was made longer in the modified version to improve the seal. At point 2, the adhesive between the heat shrink tubing, the metal tube and the rubber sleeve might deteriorate over time of extensive use under water. The modification incorporated an additional PTFE tube as an inner sleeve within the usual heat shrink, which was sealed to the body of the probe (Figure 3.8). This, it was hoped, would provide another seal for the back of the probe. At point 3, the original design involved separate pieces of metal tube and the main body, which were connected together using the Loctite adhesive. In the modified version, both parts were machined from one piece. This modified version was thereafter used for the manufacture of all subsequent suction probes. Nevertheless, the problem of
drifting still occurred. As will be explained in Section 3.8, the drifting problem was finally avoided by using glycerol as an alternative cell fluid instead of water.

\textit{b) Calibration and measurement}

The procedure for calibrating the suction probes in this research was the same as that outlined by Dineen (1997). The Budenburg dead load apparatus was used for calibrating the suction probes within the positive pressure range and the calibration relationship was extrapolated for use in the negative range. A water/water interface was used between the suction probe manifold and the Budenburg apparatus in order to prevent any possible contamination of the porous stone of the probe from the water of the Budenburg apparatus.

The suction probe was used extensively in this research project for measuring the suctions of unconfined samples, in the oedometer apparatus, and in the triaxial apparatus. The same procedure as described by Ridley et al. (2003) was used for measuring the suction of unconfined samples. Figure 3.9 shows the equipment used to make suction measurements with this technique. An alternative method has also been used in this research study (Figure 3.10), which required less equipment than the use of pedestal shown in Figure 3.9. In this method, the soil sample was sealed within several layers of aluminium foil and low-melting point wax. A hole of the same diameter as the suction probe face was made in the seal, through which the suction probe was brought into contact with the soil. Some plasticine was sealed around the probe and this proved to be sufficient for preventing evaporation from the sample around the suction probe tip. A small weight was placed on top to ensure a good contact between the porous stone and the sample, as well as to keep the suction probe in place during measurement. For both techniques, a thin layer of kaolin paste was used to improve contact between the soil sample and the porous stone of the suction probe. The use of the suction probe in the oedometer and the triaxial apparatus will be covered in Sections 3.6 and 3.8.

\textit{c) Typical plots and sources of error}

In using the suction probes, it is very important to determine correctly when the suction reading has reached equilibrium. Normally, if the probe porous stone is in good contact with the soil sample, the equilibrium time should be less than a few hours. Figure 3.11a shows a typical good measurement. The water contents of the sample were very similar
before and after the suction measurement. However, if the seal around the soil sample was not good and a slow evaporation occurred from the sample, the suction reading would appear to increase slowly at an approximately constant rate. Figure 3.11b shows an example of such a measurement. The final water content was then lower than the initial water content. Nevertheless, if the rate of evaporation was slow as in this case, the suction reading over the first few hours (Figure 3.11c) could well approximate to the sample suction at the initial water content. However, during measurement, it could be difficult to determine whether the drift in the measurement resulted from the sample drying or from a shift of the zero reading. This was especially so when measuring the suction in the triaxial apparatus, where it was impracticable to dismantle the test and check the zero reading of the suction probe. A shift in the zero reading could then only be detected at the end of the test. Therefore, if possible, the use of two probes was always preferred to using only one probe.

Another source of error is the temperature effect. The materials used to construct the probe, including epoxy resin, and stainless steel, as well as the strain gauge behind the diaphragm (Figure 2.6), deform with temperature. When using the suction probe for suction measurement, it is therefore necessary to have also a temperature reading at the same time, using a thermistor sensor. Figure 3.12 shows such a comparison between suction and temperature readings. The variation of the suction reading from Probe 2 clearly corresponds to the change in temperature. Probe 1 was, however, much less temperature sensitive. It has been observed in this research and previously by Ridley (2002), that if the suction probe is not saturated properly, the first indication of this will be a slow response of the probe and the second would be a greater sensitivity of the suction reading to temperature change. This occurs as the result of the expansion or contraction of the air-bubble present in the porous stone due to temperature change, giving rise to movement of the steel diaphragm. It is worth mentioning that normally the temperature in the laboratory was controlled within $\pm 1^\circ\text{C}$, and Figure 3.12 shows an unusual case.

3.4.2 The filter paper technique
The principle of the filter paper technique and the precautions associated with its use have been summarised in Section 2.3.1b. The detailed procedure for using the filter
paper technique is explained here. This technique was used in measuring the suction for unconfined drying/wetting as well as high-suction oedometer tests. The type of filter paper used in this research was the Whatman No.42 paper and the testing procedure was the same as that followed by Marinho (1994), Cunningham (2000) and Ridley et al. (2003). Initially air-dried filter papers were used in measuring the suction of unconfined samples during drying and those of the oedometer samples after testing. Initially wet filter papers were used for unconfined samples following wetting paths.

At the time of measurement, a disc of soil (normally 3” diameter by 19 mm thick) was sandwiched between two filter papers, either air-dried or wet depending on the type of test, and two Perspex discs, as illustrated in Figure 3.13. It was ensured that the whole face of the filter paper was in a good contact with the soil. The sandwiched sample was then wrapped in three layers of cling film. Care was taken to avoid air-entrapment between the sample and the cling film. The wrapped sample was then contained within three resealable plastic bags. Where possible, a vacuum was applied to the bag while placing the sample. This technique of vacuum sealing is a new development of Monroy (2005), and not used by earlier researchers at Imperial College, such as Ridley (1993), Marinho (1994) and Cunningham (2000). The sample and the bag were then further wrapped within 2 layers of bubble wrap. The whole assembly was transferred to a sealed container and kept away from direct sunlight or sources of temperature gradient. The storage was within the laboratory with the temperature controlled within ±1°C.

After a period of approximately 6-8 days, the measurement of the water content of the filter paper was carried out. Prior to the measurement, a small resealable plastic bag for holding each filter paper was weighed to a resolution of 0.0001g. The use of the small plastic bags was to prevent any moisture loss during the weighing process. This was extremely important as the moisture in the filter paper is of a minute quantity and thus a relatively small loss can give rise to a significant error.

The cling film was subsequently cut from one surface of the sample. One Perspex disc was removed and the filter paper from beneath the Perspex disc was placed inside a plastic bag, which was then sealed and reweighed immediately. If some soil particles adhered to the paper, they were brushed off quickly using an artist’s brush. Tweezers were used at all times for handling the paper and, where possible, the plastic bags (when
it was required to touch the plastic bag, only the two edges of the bag were held). The
whole process of removing the filter paper and transferring it to the bag was kept to
within approximately 5 seconds. Nevertheless, when the process took much longer than
this, or too much soil adhered to the filter paper, note was taken and the suction
measurement was viewed with uncertainty. Moreover, as recommended by Ridley
(1995), the condition of contact was of great importance when determining whether the
total suction or matrix suction was being measured, and a note was therefore also taken
when removing the filter paper of the stickiness or the non-stickiness of the filter paper
to the soil sample. The whole procedure was then repeated for the second filter paper.
After weighing, the filter papers were removed from the bag and placed in an oven at
105°C for 2 hours. During this period, a separate piece of filter paper or tissue paper
was placed in the empty plastic bag to absorb any moisture remaining in the bag. After
two hours, the paper was removed from the plastic bags and the empty bags were
reweighed. The dry filter papers were then placed inside the same bag that they had
been removed from and resealed. Again, the process was carried out quickly. The bag
and the dry filter paper were then reweighed. The water contents of the two filter papers
were then calculated together with the corresponding suctions, using the relationships
proposed by Chandler & Gutierrez (1986) for initially dry paper and by Ridley (1995)
for initially wet paper, which are summarised in Table 3.2. The room temperature of the
laboratory was also noted at the time of measurement, which could help explain any
rogue measurements.

3.5 Unconfined wetting and drying
This series of tests involved measuring the suction, volume and water content of
compacted Soil A over some cycles of drying and wetting in an unconfined condition.
The suction measurement was carried out using the filter paper technique in
combination with the suction plate apparatus. The suction plate was used for the suction
range between 10 and 1 kPa. Samples of differing initial as-compacted properties were
tested, which are described in more detail in Chapter 4. The testing procedure is
described below.

After each compacted sample was prepared as described in Section 3.3.4b, it was then
wrapped with filter papers for the initial measurement. For the samples following a
drying path, after the first suction measurement, the sample was subsequently dried by
approximately 0.5% gravimetric water content each time before the next suction measurement. This incremental drying process went on until the water content of the soil sample remained unchanged in the atmosphere of the laboratory. The total suction was then in an equilibrium with the relative humidity of the air in the laboratory.

For samples following a wetting path, after each suction measurement, the sample was wetted up by directly spraying water onto it to increase the water content by about 0.5%. An initially wet filter paper was then placed in contact with the sample after the sample had been sprayed. This method is different to that of Cunningham (2000) and Melgarejo (2004), who added water directly to the filter paper already placed in contact with the sample. Their method was believed to cause some differences in the condition of the filter papers during measurement and during the filter paper calibration. This may be responsible for some scatter in their results. The procedure adopted in the current research project ensures that the condition of the filter paper during measurements is the same as that during calibration.

Upon wetting the sample to suctions less than 15kPa, the filter paper technique became less accurate and hence the sample behaviour in this range was investigated using the suction plate apparatus. A schematic drawing of the apparatus is shown in Figure 3.14. The principle of this apparatus is to apply a negative pore water pressure or matrix suction through a hanging column of water (1 metre applies approximately 10kPa). The value of suction can be adjusted by changing the level of the water reservoir. The sample was enclosed in the sealed chamber to maintain a constant suction over the equilibration time. The sample was wetted up incrementally from 10, 5, 2, to 1kPa. During each increment, the sample was weighed periodically to monitor the change of water content with time. Once the water content appeared constant with a change in w/c% of less than about 0.1 % per week, the volume of the sample was measured over a period of several days, if it remained unchanged over that period, the final measurement was taken as the equilibrium value. The equilibration time was at times as lengthy as 2 months for some increments. Chapter 4 will discuss more about the influence of equilibration time on the soil behaviour.
3.6 Suction-monitored oedometer tests

3.6.1 Principle and apparatus
The suction-monitored oedometer tests were carried out with the aim of investigating the loading-collapse behaviour of the compacted soil A, as a supplement to the main experiments in the triaxial apparatus. The apparatus was a simplified version of those used by Dineen (1997) and Colmenares (2002). Figure 3.15 illustrates schematically the apparatus and its components. The whole assembly was loaded in a normal oedometer frame. Figure 3.16 shows photographs of the various components of the apparatus. The main experiments carried out with this apparatus were the suction-monitored compression tests at constant water contents. The suction probes were incorporated in the apparatus through a special top cap with a flexible clamping set, as developed by Dineen (1997). The sample was sealed within the oedometer pot using a latex rubber membrane to maintain a constant water content condition.

In addition to the constant-water content compression tests, some constant–vertical stress wetting tests were also carried out. In these tests, the samples were wetted incrementally at constant vertical stress while the suction was continuously monitored. As shown in Figure 3.17, an array of perforated holes was machined through the top cap to enable direct addition of water to the sample. The procedures used for the two kinds of test will be explained below.

3.6.2 Experimental procedure

a) Initial adjustment of water content
After the sample had been formed using the static compaction method, as described in Section 3.3.4b, its dimensions and weight were taken. If the test involved an initial increase of the water content, the sample was directly sprayed with water, as for the unconfined drying/wetting samples. For tests involving initial increase in suction, the sample was normally compacted to a larger size and then dried to the required moisture content, after which it was trimmed into an appropriate ring. Before transferring the ring with the sample to the oedometer pot, the height and weight were taken again.

b) Sample set up
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The suction probes were slowly released from the high pressure in the manifold and then submerged in a container of water in order to monitor their initial zero readings for at least 30 minutes before testing. This initial monitoring was carried out to determine the stability of the zero readings and their variation with temperature. The readings of the displacement transducer and the suction probes were logged using the TRIAX data acquisition system (Toll, 1993). The top cap was subsequently assembled with the rubber membrane for evaporation prevention. Dummy probes were temporarily put into the sockets of the suction probes to prevent evaporation. The sample with its ring was then transferred to the oedometer pot and clamped into place. In order to minimise moisture loss during testing, some silicon grease was applied to seal around the gap between the base of the oedometer ring and the base platen. Some plasticine was also used to provide a further seal. Moreover, the space within the oedometer pot was filled with several pieces of wet cloth in order to keep the air moist and to avoid any loss of moisture from the sample during the test.

The top cap was then positioned and the membrane was stretched over the pot. An o-ring was then used to secure the membrane in place. Care was also taken not to overstretch the membrane, which needed to be loose enough to allow movement of the top cap during testing. The dummy probes were now replaced with the suction probes, which had a thin layer of kaolin lightly coated onto their porous stones. The clamping plate and the securing screws were then put in place to ensure that the suction probes were in good contact with the sample during the test. The clamping was not done too tightly, however, otherwise the zero of the suction probe could deviate from the initial value. After the suction probes and their clamping plates had been positioned, the whole oedometer pot was transferred to the loading frame. The lever arm was adjusted to a level slightly above the horizontal and the yoke was positioned on the top cap. The displacement transducer was put in its reading position and a zero reading was taken. A vertical stress of 5 kPa was then applied. In general, the settlement for this nominal load of 5 kPa was relatively small, and was thus taken as a bedding error. After the first load application, the suction reading was allowed to stabilise before the next load was applied.

c) Loading
For the first set of tests carried out in this research project, the load increment was always double the previous one. Before yielding took place, the period required between each increment, before equilibrium was reached, was normally less than 6 hours. After yielding occurred, the period between each increment normally became more than 12 hours. It was ensured that both the settlement and the suction reading had stabilized before each increment was applied. For the second set of tests carried out, the load increment was changed to a smaller value with an aim of identifying the compression curve with more accuracy. The rate of loading was however maintained approximately at the same value as those in the first set of tests.

\textit{d) Constant vertical stress wetting}

For the wetting tests, the perforated holes in the top cap were used as the passage through which the water was added using a syringe. The addition of water was carried out very gradually; several drops of water were added at a time. Nevertheless, it was difficult to know exactly how much water was added to the sample and thus the water content during wetting was not available.

Before the wetting stage was started, the holes were covered with a solid plate with wet cloths around it. During the first few wetting tests however the wet cloths were not used and some water from the sample evaporated before the wetting took place. This will be explained in more detail in Section 5.3.4. The loading stage took place incrementally until the vertical stress reached the required value. Afterwards, the wetting stage started and proceeded incrementally. Between each wetting stage, it was waited until the suction and vertical settlement reached stable values. The amount of the water to be added was estimated from the results of the unconfined wetting/drying tests.

\textit{e) Unloading}

For both loading and wetting tests, the unloading stage took place incrementally. After dismantling the sample, the suction probes were submerged in the container of water to check their zero readings. If necessary, the probes were submerged in the water overnight to assess the rate of drift in zero reading with time. This was of great help when analysing data from a drifting suction probe. The final moisture content of each sample was determined at the end of the test.
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f) Filter paper measurement for high suction tests
For tests carried out at suctions higher than 1000kPa, the suction measurement was carried out only at the end of test. After dismantling, each sample was weighed and then sandwiched between filter papers using the procedure explained in Section 3.4.2. After about seven days, the sample was removed, and its water content and those of the filter papers were taken.

3.6.3 Experimental limitations
The main experimental limitation associated with this apparatus is that the information about the radial stress is absent. Nevertheless, the results can be used to make a preliminary assessment of the performance of the various models. One area of possible improvement of this oedometer apparatus is related to the wetting system used in the current research. The system was still not capable of identifying precisely the amount of water change during wetting. A possible way of improving the system will be presented in Chapter 9.

3.7 Triaxial tests on fully saturated samples
The triaxial tests on fully saturated samples were performed in the initial stages of the project. They were aimed at calibrating the apparatus with the test results of Cunningham (2000) as well as at investigating further the fully saturated behaviour of Soil A in both a reconstituted and compacted state.

3.7.1 Apparatus
A number of triaxial tests were carried out on 38mm diameter samples of reconstituted Soil A. The apparatus used was the Bishop & Wesley (1975) triaxial cell (Figure 3.18), equipped with two inclinometers for local axial strain measurement. Three tests were carried out also with a radial strain belt. The pressure source for the cell, back and ram pressures was from a mains compressor in the laboratory and controlled using pressure controllers consisting of a manostat, gear box and a stepper motor. An air/water interface was used to convert the air pressure into water pressure. The movement of the axial ram was controlled either by a pressure controller box (ram pressure) or by a constant rate of strain pump (CRSP).
The electrolevel inclinometer is shown in Figure 3.19. Its operating principle has been described elsewhere (e.g. Burland & Symes, 1982). The back pressure and cell pressure were measured using Bell & Howell transducers (range 0-150 PSI). The volume change was measured using an Imperial College volume gauge, which was also used as an air/water interface. The deviatoric stress was measured using an Imperial College type Load cell (Skinner, 1975). The radial strain belt, using an LVDT, was designed in the current research project, although it is similar those developed at City University in recent years (e.g. Klotz & Coop, 2002). Figure 3.20 shows a schematic drawing of the belt. The radial belt was attached to the soil sample before the inclinometers. The design of the belt was optimised so that once attached to the sample, the inclinometers and the belt would not interfere with each other during testing.

The control and monitoring system included an MSL unit for data logging, pressure controllers for the control of the back, cell and ram pressures, and the TRIAX program developed by Toll (1993) for data retrieval and control. The system allowed all the transducers to be continuously monitored and also allowed the control of each pressure and of the constant rate of strain pump, by means of a feedback loop.

The last test on a reconstituted sample and those on compacted samples were carried out with a sample diameter of 50mm and height of approximately 100mm. A modification was made to the base platen, but the triaxial cell was otherwise unchanged.

3.7.2 Calibration

The cell pressure and back pressure transducers were calibrated with a Budenburg dead-weight calibration unit using the technique explained in Dineen (1997) and Standing (1997). The calibration was performed over a pressure range of 0-860 kPa, using three cycles of loading and unloading in order to examine hysteresis. The regression analysis of the calibration points was then carried out using the TRIAX program to determine the calibration coefficients, which were then stored in the computer and used for conversion of the readings in subsequent tests. The load cell was calibrated with the Budenburg loading frame up to the maximum load of 3460N. A number of cycles of loading and unloading were applied before the actual calibration to minimise any hysteresis.
The calibrations of the LVDT displacement transducers and the inclinometers were carried out using the micrometer calibration unit as explained in Kuwano (1999) and Dineen (1997). The volume gauge was calibrated using a previously calibrated Bishop hand pump. The calibration of the radial strain belt was carried out in a micrometer rig, which applied a known distance between the two pads as shown in Figure 3.21. The linear range of the calibration curve was for distances of 47-53mm. During calibration, the distance was increased with an increment of 1 mm from 47 to 53 mm.

3.7.3 Testing procedure

a) Sample preparation and cell assembly
In testing the reconstituted samples, the sample of 38mm diameter (or 50mm diameter for Test TR7) was cut from the large soil cake, and then trimmed in a soil lathe using a cheese wire. In testing the compacted samples, each sample was prepared as explained in Section 3.3.4a. The sample was then weighed to the nearest 0.01g and the dimensions measured to the nearest 0.1mm before the next stage.

Meanwhile, a sintered porous stone was de-aired by boiling in de-ionised water for 30 minutes. The sample was then mounted in the cell with the de-aired porous stone placed between the sample and the base platen. A Perspex cap was subsequently positioned on top of the sample. Silicon grease was applied lightly around the sides of the top cap and the base platen. A latex membrane was then placed around the sample using a stretcher. The membrane was further sealed with two rubber o-rings at each end of the sample. The local strain instruments were then attached to the sample using ‘super glue’. Before positioning the cell cover, some silicon grease was applied around the large o-ring at the cell base. Care was taken before placing the cell cover not to trap any electrical leads from the local strain devices. It was also ensured that the load cell was at its uppermost position to avoid damaging the sample upon mounting the cell chamber.

b) Saturation
Sample saturation was carried out gradually with the mean effective stress maintained at a nominal value of 25 kPa. The back pressure was progressively increased from zero to 200kPa at a rate of 10kPa/hour, after which a B-test was carried out. The saturation stage was continued until the B-value was greater than 0.95. It was at times necessary to
increase the back pressure to greater than 200 kPa in order to achieve this value. In the case of the compacted samples (Tests AC1 and AC2), the back pressure was required to be as high as 500 kPa at the end of saturation.

c) Consolidation
Consolidation was carried out at a rate of 5 kPa/hour. Most tests involved isotropic consolidation except for two tests (TR3 and TR5), which involved K₀-consolidation. The combination of the external axial strain and global volumetric strain, from the volume gauge, was employed in calculating the radial strain, which was used as a control parameter for the K₀-consolidation. The radial strain was maintained within ±0.002%, using the procedure suggested by Toll (1993). After the target effective stresses had been reached, a period of time was waited until the rate of change in the volumetric strain fell below 0.05%/day.

d) Shearing
Three types of shearing tests were carried out: drained compression, undrained compression and constant-deviatoric stress drained shearing. For the drained compression and undrained compression, shearing was carried out under a strain-controlled condition in stages. At the start of the test, the shearing was carried out at a rate of 0.05%/hr, and the rate was doubled every 8 hour up to a maximum rate of 0.4%/hr. The shearing stage normally took 2-3 days to complete. The purpose of this scheme for shearing stage was to improve the resolution of the readings in the small strain range, while avoiding excessively long tests. It was acknowledged that differences in rate of strain might result in a difference in the stiffness. Nevertheless, within the small strain region (0-0.4% axial strain), the rate of shearing was consistently at 0.05%/hr, and therefore the behaviour at small strain was not affected by the change in shearing rate thereafter. Typically, shearing was stopped at as large a level of strain as possible, i.e. the end of the travel of the apparatus, but care was taken to ensure the local strain devices were not damaged, especially when the sample deformed non-uniformly at large strains.

For the constant-deviatoric drained shearing, the deviatoric stress was kept constant using the Constant-Rate-of-Strain Pump, while reducing the mean effective stress at a
rate of 1 kPa/hour. As the stress path touched the failure envelope, the deviatoric stress was then allowed to decrease, while maintaining a constant rate of axial strain.

e) Dismantling
Upon dismantling, the whole sample was taken out for the determination of water content. For the compacted samples, their final shape after shearing was also measured, including the diameter of each of the nine soil layers. These diameters were later used for correcting the radial strain measurement to account for the barrelled shape during shearing.

3.8 Suction-controlled triaxial tests

3.8.1 Operating principles
The development of the suction-controlled triaxial apparatus was one of the central objectives of the present research. It was built upon the design of the previous system by Cunningham (2000). Further development made during the current research included the system for estimation of the water content change during drying tests and the wetting system. The apparatus was a modified version of the Bishop & Wesley type triaxial cell described in the previous section. As shown in Figure 3.22, two suction probes were incorporated into the triaxial cell. To measure the total volume change of the sample during the tests, the local axial and radial strain measurements were used. Therefore, the back pressure system and volume gauge were not used when testing partly saturated samples. The majority of tests were carried out under a constant water content condition, and thus did not require the system for measuring water content. Nevertheless, as the testing programme progressed, the apparatus was gradually modified and therefore different versions of the apparatus were developed.

The first modification made onto the original apparatus was the size of the base platen, which was changed from 38mm to 50mm. The change of the sample size was made for the following reasons. Firstly, the 38mm sample was too small for the relatively large amount of local instrumentation: two suction probes, two inclinometers and one radial strain device. Secondly, the compacted samples had a fabric that was coarser than that
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of the reconstituted samples. For the compacted samples, the soil mixture had been passed through a 2.50mm sieve before use. This would indicate probably the maximum possible size of the fabric, which would therefore be better investigated with 50mm samples. In the following sections, a number of modifications made to the apparatus will be explained.

3.8.2 General layout

The two suction probes were attached to the side of the sample at a distance of one-quarter of the sample height from the top and base of sample, as illustrated in Figure 3.22. In some tests, however, only one suction probe was employed. As described in Section 3.4.1, the suction probes initially suffered from a problem of drifting when used under higher pressures within the water-filled cell. This was demonstrated to be due to water leaking into the strain-gauged body at the back of the probe. Even after various attempts at improving the sealing of the suction probe body had been made, the probe still occasionally drifted within the water-filled cell. To solve this problem, it was then decided to change the cell fluid rather than to change the design of the suction probe. The use of air as a cell fluid was first attempted. Nevertheless, due to safety reasons, it was not used at pressures higher than 200 kPa. Different kinds of vegetable and mineral oils were also tried. However, most types of oil appeared to attack the latex rubber membrane and were thus unsuitable. An alternative type of membrane material, such as butile, was more resistant to attack but was considered too stiff for the range of cell pressures used for the tests. Only two kinds of fluid, Castor oil and glycerol, appeared not to attack the latex membrane. The glycerol was preferred over the Castor oil as the glycerol is soluble in water. This was believed to be more appropriate especially when there might be a chance that the cell fluid could contaminate the porous stone of the suction probe.

Glycerol is considerably more viscous than water and much less electrically conductive than water. For these reasons, the drift problems of the suction probe did not occur when using them within a glycerol-filled cell. However, with the high viscosity of the glycerol, a modification was needed to the procedure for filling and emptying the cell. Figure 3.23 shows a schematic layout of the suction-controlled triaxial apparatus. The air-glycerol interface was also used as the container for the glycerol when the cell was
emptied and therefore needed to be of a large capacity. The procedure followed during testing will be described in Section 3.8.4.

3.8.3 Drying system
The air-regulated system developed by Cunningham (2000) was adopted in this research. An improvement was made in that a relative humidity sensor was incorporated into the air-out line. In addition, a coarse sintered porous disc was used in the base platen instead of the fine woven mesh used by Cunningham. Figure 3.24 illustrates the drying system. In order to dry the sample in a suction-controlled manner, an air-flow valve was used for switching on and off the flow, depending on the suction reading from the probe. During drying, if the suction reading exceeded the required value, the computer would send negative pulses to the air flow valve, which then automatically switched off the air-flow. The pressure transducer adjacent to the valve was employed for indicating when the flow occurred and when it did not. This information was used for calculating the water content during the test.

With this system, the water content of the sample was estimated based on the following hypothesis. Provided that the flow rate of air-in is constant with a constant relative humidity, the relative humidity of the air-out, which was measured, would be proportional to the amount of moisture loss from the soil sample. Appendix 1 shows the calculation adopted in the estimation of the moisture loss.

3.8.4 Wetting system
A development of the existing apparatus so that wetting paths could be followed was made in this research. A variety of systems was tried. The first idea was to circulate moist air with 100% relative humidity across the surface of the sample, using a similar system to that shown in Figure 3.24, except for the use of moist air instead of dry air. This system, however, proved to be ineffective. The sample water content remained relatively constant even after a flow of moist air across its surface for several days. The second idea was then to pass the moist air through the sample, i.e. the air flowed into the base and out of the top of the sample. Again, this method proved to be ineffective, possibly due to the relatively low air-permeability of the soil. In order to get an air flow across the sample at an appreciable rate and to be able to decrease the suction at a
practicable rate, the pressure gradient for the air flow needed to be high. The gradient in
the air pressure also caused a gradient in the net stress across the sample, which was
highly undesirable.

The final wetting system developed is shown in Figure 3.25. The system employed the
circulation of water and air via a transparent Tygon® plastic tube using a peristaltic
pump, across the top surface of the sample. During the circulation, water was gradually
absorbed into the soil sample. Between the top surface of the sample and the top cap
were placed a plastic woven mesh and a piece of filter paper, in order to ensure that
wetting happened evenly over the sample surface. Provided that the amount of water
within the tube before and after the circulation were known and it was made sure that no
water was left within any other parts of the system, the amount of water added to the
sample could be calculated. The wetting was carried out incrementally. After each
wetting increment, it was waited until the readings in suction and the local strains were
both stable before starting the next wetting stage. Details of testing procedures will be
presented in Section 3.8.6e.

3.8.5 Influence of cell pressure on the suction probes
It has been consistently observed during the testing programme that the difference
between readings from the two suction probes that were used in the triaxial apparatus
became more significant (up to 50 kPa) as the cell pressure increased to 800kPa. This
was believed to be due to the straining of the diaphragm of the suction probe, caused by
the application of the cell pressure. Accordingly the shift of zero readings of the two
probes was investigated within the cell, using a PVC dummy sample that was drained
internally to the atmosphere so that the probes read only zero pressure during the cell
pressure application. The other experimental arrangements were exactly the same as
shown in Figure 3.22. Figure 3.26 shows the change of zero reading of the two probes
during this test. It can be seen that while the zero reading of Probe 14 remained
relatively constant during the application of cell pressure, Probe 11 showed a significant
drift in its zero reading, which reached a maximum value of about 35kPa at a cell
pressure of 800kPa. The curve fitting for the drift of Probe 11 shown in the figure was
subsequently used for the interpretation of suction measurements made using this probe.
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It has not been known why Probe 11 showed the drift, while Probe 14 did not. The design of the two probes was identical but there might have been some inconsistencies in their manufacturing. Further investigation is needed for this.

3.8.6 Experimental procedure

a) Sample mounting

Before compacting the sample, the membrane was prepared, together with the grommets for holding the suction probes. Two holes of a 4mm diameter were cut into the membrane at the positions of the two suction probes (Figure 3.22). The two rubber grommets were pushed through the holes from inside of the membrane. The grommets were also inserted with dummy buttons with their screw threads facing outwards. The screw threads were used for future removal of the buttons. The membrane was also marked with the positions in which the local strain devices would be attached. Figure 3.27 shows the layout for attaching the various local instruments. Figure 3.28 shows a photograph of the various equipment used for sample mounting.

The suction-controlled triaxial tests were carried out exclusively on compacted samples that were prepared using the method described in Section 3.3.4a. The experimental programme involved testing the samples at different suctions. In order to speed up the tests, the modification of suction for different tests was carried out outside the triaxial cell, before the start of each test. After the compaction of each sample, the dimensions and weight of the sample at the as-compacted state were first taken. The moisture content of each sample was then modified to the desired value by directly spraying water onto it. The new dimensions of the sample after wetting were then taken. Again, the Soil-Water Retention Curve as identified from the unconfined wetting/drying tests was used as an initial guide in deciding how much water to add to bring about the required change in suction.

After the modification of its water content, the sample was mounted onto the base platen. An appropriate disc was placed between the sample and the platen, according to which system was being used (Figures 3.22, 3.24 and 3.25). Since the apparatus was
progressively modified as the research project proceeded, different versions of the configuration were used. Some silicon grease was lightly applied around the perimeter of the base platen and the top cap to provide a seal around the membrane. The membrane was then placed around the sample. In putting on the membrane, a stretcher with an inner diameter of 65 mm was used, in conjunction with a suction source. A continuous supply of suction was necessary since the membrane was stretched to a greater extent than usual. A venturi system, used extensively at the Imperial College laboratory (e.g. Qadimi, 2005) was used for creating the suction for this purpose. After the membrane was on the sample, two o-rings were placed around the membrane at each end of the sample. A split-ring was used for stretching the o-rings in the wetting tests, when the apparatus configuration (Figure 3.25) involved drainage lines coming from the top cap. A half-steel ball was subsequently positioned in place on the top cap.

b) Attachment of local instruments

Prior to attaching the local instruments, three layers of latex were applied over the area between the grommet and the rubber membrane to provide a seal (Figure 3.27a). Before the application, the area to which it was applied was first cleaned with methanol. After each application of latex a period of at least 45 minutes was allowed before the next application. Meanwhile, the suction probes were released from the high-pressure manifold and submerged in a reservoir of water to monitor their zero readings. At least 45 minutes after the last application of latex, the suction probes were transferred to the triaxial apparatus. The two small o-rings were then stretched around the ring stretcher (Figure 3.27a), into which the suction probe was then inserted.

The dummy button was then removed from the hole using a screw. A small amount of water was sprayed into the hole to compensate for some possible drying during the attachment of the suction probe and to prevent cavitation during setting up. The inside of the grommet was then wiped dry and if necessary cleaned with methanol. One suction probe was removed from the reservoir and smeared with a layer of wet kaolin paste on its porous stone. The perimeter of the tip of the suction probe was then wiped dry and cleaned with methanol before vulcanised rubber solution was applied onto it. The suction probe was then inserted into the grommet, which was then squeezed to make sure the contact between the probe and the grommet was tightly made. The small o-rings were then carefully moved from the stretcher onto the grommet to provide a
tight seal for the suction probe. Two layers of rubber solution were then painted around the suction probe up to the end of adhesive heat shrink and also around the grommet using an artist’s brush. At least five minutes were waited between each application. If necessary, the area to which the rubber solution was applied was cleaned before the application.

The radial strain belt, followed by the inclinometers, were in turn attached on the sample using ‘super glue’. The cell cover was then positioned over the sample. In filling the cell with glycerol, a pressure of about 800kPa was used for pushing the glycerol from the air-glycerol interface into the cell (Figure 3.23). During filling, valves a and b were open, while valve d was closed (i.e. the glycerol flowed in from the top). The pressure was then gradually released once the cell was nearly full. Once the cell was full, valve a was closed, followed by valves b and c. The zero reading of the cell pressure was taken considering that the sample mid-height was the datum.

c) Compression and shearing stages
During subsequent testing, valve c was kept closed, while valves a and d were open. This arrangement was meant to minimise the time lag between the pressure application and the reading from the cell pressure transducer. A nominal cell pressure of 50kPa was then applied to the sample gradually. A period of at least 24 hours was waited at this pressure of 50kPa for the equalisation of suction readings and of the local strain measurements. The compression stage was subsequently carried out at the rate of 5kPa/hour. For K_o-compression, the radial strain from the local measurement was used as the control parameter, with the radial strain maintained within ±0.002%. It is worth reminding that this procedure was different to that used for the fully saturated tests described earlier, which made used of the combination of global volume and axial strain measurements for radial strain control. The shearing procedure followed was the same as that used for the fully saturated tests.

d) Constant suction control and drying
The air-regulated system was used for drying and for the control of constant suction during consolidation or during shearing. The same procedure as that used by Cunningham (2000) was followed with the addition of the use of the relative humidity
sensor and the flow indicator, shown in Figure 3.24. The suction reading from the probe that was nearer to the drying face was used as the control parameter. The tolerance between the required suction and the reading was maintained within ± 10 kPa.

e) Wetting

As explained in Section 3.8.4, wetting was carried out incrementally. For each increment, the required amount of water (normally around 0.4% w/c) was added to the transparent tube using a syringe. Both air and water were present in the tube. The tube plus water was sealed at both ends using metal bulldog clips and then the whole assembly was weighed to the nearest 0.01g. The tube plus water was then connected to the triaxial apparatus and the peristaltic pump as shown in Figure 3.25 and the circulation of water started. As the circulation continued, water was adsorbed into the sample and less water was therefore visible in the transparent tube. Once no water could be seen in the tube, the circulation was continued for at least further several hours to ensure no more water was trapped inside the system. The tube was then taken out, sealed with the same metal clips, and reweighed. The difference between the initial and the final weight was assumed to be equal to the increase in the weight of water within the sample. As the suction decreased, it generally became more difficult to increase the water content of the sample and thus required longer times for the water in the tube to disappear.

f) Dismantling

Upon dismantling, the cell pressure and deviatoric stress were reduced to zero. The pressure line from the controller was disconnected from valve e and connected to valve b. Valve c was then opened and valve a closed. A pressure of 200kPa was applied to push the glycerol back into the air-glycerol interface. With this method, the cell was emptied within two hours. After the cell chamber was removed, the membrane was washed with water and wiped dry before detaching the suction probes. This was to avoid any contamination of the porous stone with glycerol. The dummy button was again inserted in place of the suction probe. The sample was then carefully removed from the platen. The membrane was quickly removed from the sample to avoid evaporation. The sample was then weighed immediately. If the sample exhibited a
distinct barrelling shape, the diameters of various layers of the sample were taken. This
dimension was later used in the correction of the volume change during shearing.

**g) Corrections during shearing**

As discussed in Section 2.3.3, since the local strain devices only measure the strain
between two fixed points, an assumption needs to be made about the shape of the
sample during compression and shearing. During isotropic- and K<sub>0</sub>-compression, the
right cylinder assumption that is usually made did not deviate much from the actual
shape of the sample. Upon shearing, for the samples with high suction, i.e. those greater
than 700 kPa, a distinct shear plane normally formed at a relatively small strain and the
right cylinder assumption also appeared to be valid up to that point. After the formation
of shear plane, indicated by a rapid reduction in deviatoric stress, the correction method
proposed by Chandler (1966) was used.

For samples that failed in a barrelling mode, the right cylinder assumption normally
overpredicts the overall volume change of the sample during shearing, and the
assumption can predict unrealistic dilation of the sample during shearing. A method was
therefore adopted to make a correction for the barrelling effect. With this method, the
final diameters of the sample were measured at different elevations. The smallest final
diameter measured was then assumed to correspond to a threshold value of the radial
strain for the sample. For the radial strains below this threshold value, the right
cylindrical shape was assumed to be valid. For the radial strains greater than the
threshold value, the barrelling assumption was used. It was assumed that, during
barrelling, there was a constant ratio between the part of radial strain that was in excess
of the threshold value, at a certain height, and that at the mid-height. The variation of
this ratio with height was measured at the end of test, and was assumed to remain
constant throughout the barrelling. This method was similar to that adopted by Klotz
and Coop (2002). Appendix 2 explains this method in more detail.

In order to validate this correction technique, the volumetric strain calculated with this
method was compared with the global measurement from a volume gauge, using test
results for one fully saturated drained and one undrained shearing test. As shown in
Figure 3.29, the volumetric strains based on the barrelling correction method are very
similar to the values from the global measurements. This correction method was thus
believed to provide a good estimation of the global volume change for the barrelled samples during shearing.

Table 3.1 Atterberg Limits of the soil A used in the present studies and in Cunningham (2000)

<table>
<thead>
<tr>
<th></th>
<th>LL</th>
<th>PL</th>
<th>Clay content, %</th>
<th>Activity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current studies</td>
<td>29</td>
<td>18</td>
<td>26</td>
<td>0.42</td>
</tr>
<tr>
<td>Cunningham (2000)</td>
<td>28</td>
<td>18</td>
<td>26.6</td>
<td>0.38</td>
</tr>
</tbody>
</table>

Table 3.2 Calibration relationships for the filter paper suction measurement

<table>
<thead>
<tr>
<th>Filter paper water content, $w_f, %$</th>
<th>Relationship for suction, $s, \text{kPa}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initially dry paper</td>
<td></td>
</tr>
<tr>
<td>$w_f \leq 47$</td>
<td>$\log_{10}s = 4.84-0.0622(w_f)$</td>
</tr>
<tr>
<td>$w_f &gt; 47$</td>
<td>$\log_{10}s = 6.05-2.48\log_{10}(w_f)$</td>
</tr>
<tr>
<td>Initially wet paper</td>
<td></td>
</tr>
<tr>
<td>$w_f \leq 15.47$</td>
<td>$\log_{10}s = 4.84-0.0622(w_f)$</td>
</tr>
<tr>
<td>$15.47 &lt; w_f \leq 57.2$</td>
<td>$\log_{10}s = 4.573-0.0449(w_f)$</td>
</tr>
<tr>
<td>$w_f &gt; 57.2$</td>
<td>$\log_{10}s = 2.904-0.0158(w_f)$</td>
</tr>
</tbody>
</table>
**Figure 3.1** Results for unconfined compression tests on the Saprolitic sandy clay
Figure 3.2 Results for unconfined compression shearing tests on the Pumiceous sand
Figure 3.3 Gradation curves for the Soil A used in the present studies and in Cunningham (2000)

Figure 3.4 Schematic procedures for sample preparation of reconstituted and compacted samples of Soil A
Figure 3.5 Equipment used for the compaction of 50mm diameter triaxial samples, a) equipment, b) compaction, c) extrusion
Figure 3.5 Equipment used for compaction of 50mm diameter triaxial samples (continued)
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Figure 3.6 Compaction mould for 3” samples (Maswoswe, 1985)
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**Figure 3.7** Three possible weak points of the suction probe where water could permeate into the probe body under water at high pressure

**Figure 3.8** The suction probe modified to reduce the weaknesses of the three locations
Figure 3.9 Equipment used to make suction measurements on unconfined sample in the laboratory with the suction probe (Ridley et al., 2003)

Figure 3.10 Alternative procedure for making suction measurements in the laboratory with the suction probe
Figure 3.11 Typical equilibrium times & suction measurements on compacted soil A, using suction probes: a) Sample 1, good measurement, b) Sample 2, measurements on a slowly evaporating sample, c) Sample 2, showing only measurements for the first 5hrs.
Figure 3.11 Typical equilibrium times & suction measurements on compacted Soil A, using suction probes (continued)
Figure 3.12 Influence of temperature fluctuation on suction readings
Figure 3.13 A soil sample sandwiched between two filter papers and two Perspex discs

Figure 3.14 The suction plate apparatus
Figure 3.15 Experimental set up of the suction-monitored oedometer apparatus for constant water content loading tests
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Figure 3.16 Equipment used for the suction-monitored oedometer tests

Figure 3.17 Top cap used for wetting tests in the suction-monitored oedometer apparatus
Figure 3.18 Bishop & Wesley (1975) triaxial stress path cells for 38mm samples (diagram from Head, 1982)

Figure 3.19 Electrolevel inclinometer (diagram from Kuwano, 1999)
Figure 3.20 The new radial strain belt

Weight of strain belt plus LVDT = 39 g
Figure 3.21 Arrangement for the calibration of the radial strain belt
Figure 3.22 The 50mm diameter stress path triaxial apparatus for constant water content tests.
Figure 3.23 Schematic layout of the suction-controlled triaxial apparatus
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Figure 3.24 Layout of the drying system used for the suction-controlled triaxial apparatus
Figure 3.25 Layout of the wetting system used for the suction-controlled triaxial apparatus

- Peristaltic pump
- Transparent Tygon tube (R3603, I.D.=1/8", O.D.=1/4"
- Filter paper
- Plastic woven mesh
- The connection is tightly sealed but can be removed by hand
- Vent to atmosphere
Figure 3.26 Drifting of zero readings from the two suction probes used in the triaxial apparatus

\[ y = -2\times10^{-5}x^2 - 0.0278x \]
Figure 3.27 Layout for the attachment of various local instruments onto a 50mm diameter triaxial sample, a) suction probe, b) top view of an instrumented sample
Figure 3.28 Various equipment used for sample mounting
Figure 3.29 Comparison between local and global measurements, a) during drained shearing (Test TC1), b) during undrained shearing (Test TR7)
CHAPTER 4

FUNDAMENTAL PROPERTIES OF COMPACTED SOIL A: COMPACTION CHARACTERISTICS AND SOIL-WATER RETENTION CURVES

4.1 Introduction

As discussed in the previous chapter, the artificial silty clay, Soil A, compacted dry of optimum, was considered to be the most suitable for this investigation. The logical next step was to examine further other fundamental properties, namely the compaction characteristics and the soil-water retention curves. This chapter describes these elements of the investigation.

4.2 Compaction characteristics

A variety of compaction methods have been employed in the laboratory, which are aimed at achieving different purposes. These methods include dynamic compaction, kneading, and static compaction (Lambe & Whitman, 1979). Dynamic compaction is one of the standard procedures, normally employed in practice for the purpose of duplicating conditions in the field. The kneading and static compaction methods are generally used for other specific purposes such as in research. In particular, static compaction has proven consistently to produce more uniform soil samples, which is normally used for soil element testing, see for example, Sivakumar (1993), Sharma (1998), Cui & Delage (1996). The static compaction method was chosen as the sample preparation technique in this study.

4.2.1 Dynamic and static compaction curves

In order to provide a reference state for static compaction, and for the initial states of the soil samples, the two compaction curves, for ‘heavy’ and ‘light’ compaction tests, have been identified for Soil A according to BS1377: Part 4:1990. These are shown in Figure 4.1.

As explained in Section 3.3, concerning the sample preparation method, the static compaction tests were carried out with compaction pressures of 400 kPa and 800 kPa. The initial properties of the samples statically compacted at both pressures are also plotted in Figure 4.1.
4.2.2 As-compacted suction

For each of the samples compacted statically, the as-compacted initial suction was measured using the suction probe technique and the results are shown in Figure 4.2, plotted against the as-compacted moisture content. Initially, there was concern regarding the influence of hydration time on the initial suction, and therefore, a study was performed to investigate this influence. As described in the sample preparation technique in Section 3.3, the mixture of moist soil was left to hydrate for a certain period of time before compaction. The time of hydration was varied between one day and four weeks in order to investigate its influence on the initial suction.

The results shown in Figure 4.2 indicate a scattered and unclear trend for the influence of hydration time on initial suction. Nevertheless, the relationship between water content and log suction is linear, as normally observed for compacted clays (Ridley & Perez-Romero, 1998). This apparently insignificant influence of hydration time on initial suction is expected, considering the low activity of the clay content of this material, which is largely kaolinite. However, a hydration time of one week was chosen, in order to allow the moisture time to equilibrate throughout the mixture.

4.3 Choices for the samples as-compacted states

Once the compaction characteristics of soil A had been established, the next step was to decide the initial properties of the samples, i.e. their moisture content and void ratio, for the main testing programme. The criteria for suitable initial properties were twofold.

a) The initial suction is approximately 600 kPa to enable investigation of soil behaviour both in the wetting (suction < 600 kPa) and drying (suction > 600 kPa) ranges, given that the maximum working range of the suction probe is around 1200-1500 kPa.

b) The samples exhibit a collapse-upon-wetting behaviour.

A preliminary double oedometer test, described in Section 3.4, was carried out on two samples, compacted dry of optimum, at a dry density of ~1.56 g/cm³. The results are shown in Figure 4.3 and indicate the collapse-on-wetting behaviour, thus suggesting suitability for the second criterion.
The initial properties finally chosen for the main testing programme were those of a void ratio of 0.70 and water content of 10% (Series 7-10). This is a compromise between the criteria mentioned previously and ease of handling the samples. Samples drier or looser than the chosen conditions were very difficult to handle due to their tendency to crumble. In addition to the samples for the main testing programme, two other series of samples were also tested, namely Series: 5-10 and Series: 7-13. The properties of all three Series are summarised in Table 4.1, and plotted in Figure 4.1 in terms of dry density and water content.

<table>
<thead>
<tr>
<th>Series</th>
<th>Specific volume</th>
<th>Gravimetric water content, w/c, %</th>
<th>Degree of saturation, Sr, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-10</td>
<td>1.70</td>
<td>10</td>
<td>38</td>
</tr>
<tr>
<td>7-13</td>
<td>1.70</td>
<td>13.5</td>
<td>51</td>
</tr>
<tr>
<td>5-10</td>
<td>1.50</td>
<td>10</td>
<td>53</td>
</tr>
</tbody>
</table>

Table 4.1 Initial properties of the three main series

The purpose of testing the additional Series 7-13 was so that the influence of aggregate size, and hence the fabric of the sample on its unsaturated properties could be investigated for samples with similar void ratios. It has long been appreciated that samples compacted at different moisture contents will possess different fabrics (e.g. Lambe & Whitman, 1979, and Alonso et al., 1987). The influence of void ratio on the loading-collapse curve and soil-water retention curve was also investigated by comparing test results of Series 5-10 with those of Series 7-10.

To summarise, the initial properties of the compacted samples, which will subsequently be tested in the main experiments, have been identified and meet the two main criteria, the sample’s collapsibility and having a reasonable initial suction for the suction probe. In the subsequent sections, various experimental results for these three sets of samples will be explained.
4.4 Soil-Water Retention Curves (SWRC)

4.4.1 Soil-Water Retention Curves of Collapsible Soils
Many collapsible soils arrive at their natural state having always been partially saturated. Such materials may have never experienced full saturation. Examples are compacted soils and some natural deposits, such as loess and some residual soils. These collapsible soils exist at large void ratios, and would be as soft as slurry if they were in a fully saturated state at the same void ratio. They can exist at such a high void ratio in a partially saturated state as a result of the suction and clay bonding effects.

Normally, description of the SWRC is given such that the soil samples start drying from their fully saturated state and desaturate at a specific suction (see e.g. Fredlund & Rahardjo, 1993). Slurried samples, especially of a clayey type, tend to shrink on drying, and therefore, it is unlikely that a dried slurried sample will arrive at the same void ratio as that of a collapsible sample at the same suction. For these reasons, it has been speculated that the SWRC of collapsible soil A would be rather different to that of reconstituted soil A, prepared from a slurry. The experimental programme described in this section was therefore intended to explore this aspect of behaviour and provide additional insight into the collapse behaviour of the material.

4.4.2 Testing programmes
Soil-Water Retention Curves (SWRC) were identified for the three main series of compacted samples (7-10, 5-10, and 7-13), tested under unconfined conditions. Table 4.2 gives a description of all the unconfined drying/wetting tests. All samples were compacted in a 3” mould. More details of the testing procedures can be found in Chapter 3. The results will be discussed with reference to the SWRC for reconstituted Soil A identified by Cunningham (2000).

For each series of samples, two main types of test were carried out on two samples, involving two different stress paths, one drying from the initial state and another wetting (see Table 4.2). The last letter of the sample number in Table 4.2 indicates the type of initial path followed. The samples also underwent additional wetting/drying cycles afterwards as detailed in the table. It should be recognized that the complete
SWRC for each series is not from a single sample, but from a number of practically identical samples. This is inevitable since each test can be relatively lengthy. For example Test 7-10W2 lasted nine months. The presentation of results in the following sections will be given according to the different topics being investigated.

In presenting the results, given that partially saturated soils are composed of three-phases, at least two state variables are required to represent fully their states. In order to clarify the complete states of the samples during wetting/drying, the results will be presented in the following plots.

1. Degree of saturation, $\text{Sr}$, vs. log suction
2. Gravimetric moisture content, $w/c$, vs. log suction
3. Volumetric moisture content, $\theta$, vs. log suction
4. Void ratio, $e$, vs. log suction

By plotting the SWRC using a number of different variables, it is believed that the pattern of behaviour during drying and wetting can be visualised better.

4.4.3 First drying and first wetting: Influence of initial void ratio on the SWRC

a) $\text{Sr}$- log suction

The SWRCs for Series 7-10 and 5-10 are plotted in terms of the degree of saturation against log suction in Figure 4.4a. The curves were followed by the samples during the first drying and first wetting from their as-compacted states. The arrows shown indicate the direction of the paths. Evidently, the retention curve for the 7-10 Series has a distinct bi-modal shape. Conversely, the SWRC for 5-10 Series, which was compacted to a lower initial void ratio at a higher compaction stress, is more uni-modal, apart from a small kink in the wetting curve at a suction of about 10kPa, although this kink might have been simply due to experimental error.

Bi-modality of the retention curves can be explained by the presence of structures at two scales in the samples, which will be referred here as the macro- and micro- voids. For compacted clays, it has been proposed (see e.g. Alonso et al., 1987, and Burland &
Ridley, 1996) that aggregations of clay form and interact in a granular manner. These aggregations represent the macro-structure, whereas within the aggregates exists the micro-structure. As for the 5-10 samples, since most of their macro-voids were probably compressed during compaction, the division between two scales of structure was less pronounced, leaving the pore size distribution more well-graded. The retention curve thus becomes more continuous with less bimodality.

For suctions above 2000kPa, the SWRCs for both series appear to converge, suggesting a unique relationship between degree of saturation and suction, regardless of void ratio or soil structure. At high suctions and low degrees of saturation (<30%), the contact of water in the liquid phase between the sample and the filter paper is difficult to maintain, and the moisture transfer would be mainly through the vapour phase. Thus, it would be the total suction that was measured instead of the matric suction.

b) w/c – log suction
The first drying and first wetting curves are plotted in terms of gravimetric water content versus log suction in Figure 4.4b. The comparison between the two curves is rather different to that of the Sr-log suction plots. For the suction range between 50 and 1000kPa, the relationship between w/c and suction appears remarkably similar for both series. At higher suctions, the two curves converge only when the suction is greater than about 4MPa.

c) θ – log suction
The same curves are shown on a volumetric water content against log suction plot in Figure 4.4c. The two series follow different curves before converging at suctions greater than about 5000kPa.

d) e- log suction
The plot of void ratio against log suction is shown in Figure 4.4d. Some difficulties in interpretation arise due to the limited accuracy of measurement, resulting in scatter of the data. Nevertheless, the observed volume changes were relatively small, suggesting the non-expansiveness of the samples. However, the sample expansion becomes more substantial when the suction reduces to around 10 to 20kPa.
4.4.4 First drying and first wetting: Influence of fabric on the SWRC

a) Sr-log suction

Figure 4.5a shows the SWRCs for Series 7-10 and 7-13 on an Sr-log suction plot. The bi-modality can be seen clearly for both retention curves, indicating the macro- and micro-structural scales. However, the difference in fabric induced by the differing as-compacted moisture contents, as well as in void ratio, causes some deviation in their retention curves. Again, the difference is believed to be related to the pore size distributions of the two series.

For suctions between 1 and 30 kPa, the 7-13 retention curve locates to the left of the 7-10 curve. A possible explanation is that the macro-voids of the 7-13 samples have a larger average size than that of the 7-10 samples. Visual inspection appears to support this speculation. This will be confirmed by the more detailed fabric studies presented in Chapter 6.

b) w/c – log suction

From Figure 4.5b, the relationship between w/c and suction for both series appears to be remarkably similar for suctions greater than about 70 kPa. However, the relationships deviate for suctions below 70 kPa, as observed in the Sr-log suction plot. The same explanation may also apply to this plot.

c) $\theta$ – log suction

The same curves are shown for volumetric water content, $\theta$, plotted against log suction in Figure 4.5c. A similar trend as observed in Figure 4.5a and b can also be seen in this plot.

d) e-log suction

Figure 4.5d shows the plot of void ratio against log suction. Again the scatter of data poses some difficulties in interpretation. However, the trend is very similar for both samples. The volumetric changes appear insignificant, except in the suction range from...
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200 to 2000kPa, as well as when the suction is below 10kPa. These stages in volume change are believed to correspond to the two structural scales discussed earlier.

4.4.5 Hysteresis loop: Series 7-10

As indicated in Table 4.2, several tests were carried out involving wetting/drying followed by redrying/rewetting over a number of cycles. The main purpose was to investigate the hysteresis of the retention curves.

Childs (1969) gives an elaborate set of definitions for various retention curves in a hysteresis loop for non-shrinking soils, including the boundary drying/wetting curves, the primary drying/wetting curves, and the scanning curves, of which more details are given in Chapter 2. In this section, the identification of these curves for this material is attempted.

a) Boundary drying curve and Primary drying curve

The investigation of the boundary and the primary drying curve was made with samples S-7-10W2 and S-7-10W3, wetted from an as-compacted state to a suction of 1 kPa and subsequently redried to an air-dried state. Figures 4.6a to 4.6e show the drying curves for both samples in various plots.

It was intended to bring sample S-7-10W2 to a suction of 1kPa from an as-compacted state in one large step. However, since the sample was wetted without any retaining ring it became almost as soft as a slurry once the suction was reduced to 1 kPa. It was therefore decided to allow the sample to absorb water only for three days before redrying, in order to avoid losing the sample. Upon redrying the sample did not retain its original shape and numerous cracks appeared around its edge. The sample was then subsequently trimmed down to 50mm diameter by pushing a standard oedometer ring through it. During trimming, it appeared that the sample contained some air bubbles, suggesting inadequate saturation. The measurement of the sample dimensions was also inaccurate due to its distorted shape and the presence of cracks. The degree of saturation at the suction of 1kPa could not therefore be determined with certainty.

The redrying path (AB) followed by sample S-7-10W2 in Figure 4.6a-d therefore did not start from the full saturation, and more appropriately represents the primary drying
curve, as defined by Childs (1969) as ‘a characteristic from an intermediate point on a boundary wetting curve and carried to maximum suction’. Whether the sample starts from the boundary wetting curve will be discussed later.

The primary drying curves AB, plotted in terms of either $Sr$, $w/c$, or $\theta$ against log suction are generally very similar as indicated in Figures 4.6a, b, and c. The influence of soaking the sample to the suction of 1 kPa is to reduce the bi-modality of the retention curve. It is expected that the swelling of sample from a suction of 10 kPa to 1 kPa has changed the more open as-compacted fabric to a more slurry-like fabric. This slurry-like sample, being softer than the as-compacted soil, reduced in volume more during drying, as indicated in Figure 4.6d. This reduction in volume affects the macro-pores and hence the bimodality.

Another sample, S-7-10W3, was subsequently investigated. It was decided to wet the sample slowly to 1 kPa, within a retaining ring, to protect the sample from excessive distortion, and also to ensure that most of the air bubbles escaped from the sample. During wetting and drying within the suction range between 1 and 10 kPa, the equilibrium between each step required a much longer period than that for the filter paper (1 week), as indicated in Figure 4.7. Between each stage, the sample was allowed to rest until the rate of change in gravimetric water content fell below 0.01%/day over several days. This rate was chosen arbitrarily to give realistic measurements of water content within a reasonable equilibrium time.

The retention curve of this sample, for the redrying path, is shown in Figures 4.6a-d. In Figure 4.6a, the drying curve for sample S-7-10W3 on the $Sr$-log suction plot lies above the drying curve for sample S-7-10W2, before converging at a suction of about 1500 kPa. This was clearly because of the longer time of saturation used for sample S-7-10W3. On plots between w/c-log suction and $\theta$-log suction (Figures 4.6b & c), however, the redrying curves do not coincide until the suction exceeds about 4000kPa. The S-7-10W3 curve instead lies slightly below the S-7-10W2 curve for suctions between ~400 to 4000kPa.
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The redrying curve for sample S-7-10W3 can be considered as the boundary drying curve, indicating ‘a complete moisture characteristic between saturation at vanishing suction as the initial state and the maximum suction attainable’, which is the definition proposed by Childs (1969). This boundary drying curve (S-7-10W3) appears to enclose the primary drying curve (S-7-10W2) on the plot of $Sr$ against log suction in Figure 4.6a. This observation is in accordance with the typical hysteresis of the Soil Water Retention Curve as proposed by Childs (1969) for non-shrinking granular material (see Figure 2.4).

However, from Figures 4.6b & c, it can be seen that the S-7-10W3 drying curve does not bound the S-7-10W2 over the entire range of suctions. This deviation from the theory should not come as a surprise, given that Childs (1969) proposed the definitions for non-shrinking soils. Figure 4.6d shows that sample S-7-10W3 shrinks more on redrying than sample S-7-10W2 does. The influence of the longer period of saturation used for sample S-7-10W3 is once again believed to be responsible for this. The longer the period of saturation, the less air entrapment and the more slurry-like the sample would be.

b) Boundary wetting curves

The rewetting curves followed by Samples S-7-10D and S-7-10W2 after they reached the air-dried state can be regarded as the boundary wetting curve according to the definitions by Childs (1969). Figures 4.8a-d show these rewetting curves on various plots. From Figure 4.8a, the boundary wetting curves on the $Sr$-log suction plot for both samples appear to follow different paths, once again indicating the strong influence of structure and thus void ratio on the retention curves. Figure 4.8d shows the difference in voids ratio of both samples. Nevertheless, upon rewetting, it can be seen that the voids ratio of both samples hardly increased. Figures 4.8b & c also demonstrate the difference in the two boundary wetting curves of the samples on the $w/c$ and $\theta$ against suction plots.

e) Scanning curves

After rewetted to a suction of about 15 kPa, sample S-7-10D was subsequently redried to a suction of approximately 2000 kPa (see Table 4.2). The results are shown in
Figures 4.9a-d. It can be seen from Figures 4.9a-c that the redrying curve of sample S-7-10D rejoins its first drying path at a suction of around 2000 kPa. By definition, this redrying curve starts from the boundary wetting curve and thus can be considered as the primary drying curve. From this information the scanning curve can thus be identified from the plots as being the first drying/wetting curve of samples S-7-10D & S-7-10W in the suction range between 200 to 2000 kPa.

Figure 4.9d shows the change in void ratio for samples S-7-10W & S-7-10D during wetting/drying. Both samples underwent very minimal changes in void ratio. It is therefore not surprising that the observed behaviour of these two samples is well in accordance with the hysteresis framework of Childs (1969) given that the framework was proposed for non-shrinking soils.

**4.4.6 Hysteresis loop: Series 7-13**

Samples S-7-13D and S-7-13W followed several wetting/drying cycles, as described in Table 4.2. Figures 4.10a-d show the results on various plots. From Figures 4.10a-c the hysteresis loops of both samples, for the suction range above 1500 kPa, appear to coincide. Nevertheless for suctions below 1500 kPa the hysteresis loop seems to depend considerably on the void ratio as well as the wetting/drying history.

Figure 4.10d shows the plot of voids ratio against log suction. It is interesting to note that even though the changes in void ratio of Series 7-13 during the wetting/drying loops are minimal (~ 0.02 in voids ratio), and in fact very similar to the change undergone by the samples of Series 7-10, as was shown in Figure 4.9d, the hysteresis loops of Series 7-13 appear to be more dependent on the void ratio and the wetting/drying history. It is thought that the samples of Series 7-13 have a structure that is more prone to change upon air-drying and wetting. Despite the marginal change in overall void ratio over the loops, the change in pore size distribution for Series 7-13 might have been greater.

**4.4.7 Comparison with the reconstituted Soil A**

The hysteresis loops identified earlier for Series 7-10 are compared with that of the reconstituted sample in Figures 4.11a-d from tests carried out by Cunningham (2000). Details of the tests on reconstituted samples can be found in Cunningham.
reconstituted sample had been consolidated from the slurry, at a moisture content of 1.5 times the Liquid limit, to a vertical stress of 200 kPa before unloaded and drying started in an unconfined state.

For suctions above about 5000 kPa, it can be seen from Figures 4.11a-c that the hysteresis loops are practically identical for all samples. The boundary drying/wetting curves from the reconstituted sample however lie above those of Samples S-7-10W3 and S-7-10W2 until converging with them at suctions above 1000 kPa on a plot of Sr against log suction in Figure 4.11a. This once again indicates the influence of structure induced by the sample preparation method. The reconstituted sample had undergone one-dimensional compression to 200kPa vertical effective stress and was therefore in a denser state, as shown in Figure 4.11d. No definite trend, however, could be observed between the retention curves on plots of w/c and $\theta$ against suction. Nevertheless, the differences between the loops on the w/c-suction plot appear to be less than those on the $\theta$-suction plot.

4.5 Final comments on the SWRCs

Engineers and soil physicists are aware of a number of factors that influence the SWRCs of a soil, for example soil fabric, hydraulic hysteresis, void ratio/applied stress (see e.g. Hillel, 1998, and Barbour, 1998). In fact, it has been a challenge to model this aspect of the change in SWRCs and a number of models have been proposed. As was seen from the data, there are difficulties in using the hysteresis concept proposed by Childs (1969) as the boundary/primary retention curves do not remain stationary but move as the void ratio changes during the course of drying and wetting. Indeed, the hydraulic hysteresis has to be separated from the mechanical hysteresis, which occurs as a result of changes in void ratio, in order to render the concept useful.

In this respect, Gallipoli et al. (2003b) suggested the hypothesis that in the absence of hydraulic hysteresis effects, there is, for a given soil, a unique relationship between the degree of saturation, $Sr$, suction, $s$, and specific volume, $\nu$. It is proposed here that the hydraulic hysteresis concept of Childs (1969) could also be incorporated into the void ratio dependent retention curve concept of Gallipoli et al. The hysteresis loops should also be dependent on voids ratio. As seen from the test results, as the void ratio becomes
smaller, as a result of either shrinkage on drying or applied stresses, the hysteresis loops tend to move to higher suctions on the Sr-suction plots.

Nevertheless, the comparison between the retention curves of Series 7-10 and 7-13 suggests that despite the samples having similar void ratios, their pore size distributions may be different, depending on the fabric induced by the compaction water content. The shape of the hysteresis loops for samples of differing fabrics can be very different especially for suctions less than 1000 kPa when the capillary effect dominates the wetting/drying process. In general, however, for suctions greater than 5000 kPa, the hysteresis loops are largely unaffected by the void ratio or fabrics, due to the dominance of adsorption in the wetting/drying process.

Another important aspect is the bimodality of the retention curve, which results from the fabric and, less clearly, to the void ratios, of the soils. The new Soil-Water Retention Surface will also need to accommodate for the bi-modality. Finally, it should be emphasized again that the concept of hysteresis loops, initially proposed by Childs (1969) for non-shrinking soils, should be applied only when the change in void ratio with suction is minimal and should not be extend for use with more compressible soils such as soft clays or collapsible soils.
# Summary of unconfined drying/wetting testing programme

<table>
<thead>
<tr>
<th>Test series</th>
<th>Sample no.</th>
<th>Void ratio, $e$</th>
<th>Moisture content, w/c, %</th>
<th>Paths followed after compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-10</td>
<td>S-7-10D</td>
<td>0.708</td>
<td>9.96</td>
<td>1) Drying until air-dried. 2) Wetting until 15kPa suction. 3) Drying until 2000kPa suction</td>
</tr>
<tr>
<td></td>
<td>S-7-10W1</td>
<td>0.706</td>
<td>10.02</td>
<td>1) Wetting until 20kPa suction</td>
</tr>
<tr>
<td></td>
<td>S-7-10W2</td>
<td>0.707</td>
<td>10.42</td>
<td>1) Wetting until ~1kPa suction (see text). 2) Drying until air-dried. 3) Wetting until 10kPa suction</td>
</tr>
<tr>
<td></td>
<td>S-7-10W3</td>
<td>0.709</td>
<td>10.27</td>
<td>1) Wetting until 1kPa suction, with accurate measurement of SWRC for low suction range (sample retained in the oedometer ring) 2) Drying until air-dried.</td>
</tr>
<tr>
<td>5-10</td>
<td>S-5-10-D</td>
<td>0.510</td>
<td>9.96</td>
<td>1) Drying until air-dried</td>
</tr>
<tr>
<td></td>
<td>S-5-10-W</td>
<td>0.517</td>
<td>9.79</td>
<td>1) Wetting until 40kPa suction (Test stopped due to presence of fungi)</td>
</tr>
<tr>
<td></td>
<td>S-5-10-W2</td>
<td>0.504</td>
<td>9.98</td>
<td>1) Wetting until 1kPa suction, with accurate measurement of SWRC for low suction range (sample retained in the oedometer ring)</td>
</tr>
</tbody>
</table>

**Table 4.2** Summary of unconfined drying/wetting testing programme
### Table 4.2 Summary of unconfined drying/wetting testing programme (continued)

<table>
<thead>
<tr>
<th>Test series</th>
<th>Sample no.</th>
<th>Void ratio, e</th>
<th>Moisture content, w/c, %</th>
<th>Paths followed after compaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>7-13</td>
<td>S-7-13-D</td>
<td>0.678</td>
<td>13.48</td>
<td>1) Drying until air-dried. 2) Wetting until 10kPa suction. 3) Drying until 1000kPa suction</td>
</tr>
<tr>
<td></td>
<td>S-7-13-W</td>
<td>0.684</td>
<td>13.14</td>
<td>1) Wetting until 10kPa suction. 2) Drying until air-dried. 3) Wetting until 10kPa suction</td>
</tr>
<tr>
<td></td>
<td>S-7-13-W2</td>
<td>0.693</td>
<td>13.57</td>
<td>1) Wetting until 1kPa suction, with accurate measurement of SWRC for low suction range (sample retained in the oedometer ring)</td>
</tr>
</tbody>
</table>

Table 4.2 Summary of unconfined drying/wetting testing programme (continued)
Figure 4.1 Compaction characteristics of Soil ‘A’

Figure 4.2 Relationship between as-compacted suction and moisture content of Soil ‘A’
Figure 4.3 Results from the preliminary double oedometer test on compacted Soil A
Figure 4.4 SWRC for Series 7-10 and 5-10 during first drying and wetting
Figure 4.4 SWRC for Series 7-10 and 5-10 during first drying and wetting (continued)
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Figure 4.5 SWRC for Series 7-10 and 7-13 during first drying and wetting

a)

b)
c) Figure 4.5 SWRC for Series 7-10 and 7-13 during first drying and wetting (continued)
Figure 4.6 SWRC for Series 7-10 during redrying
Figure 4.6 SWRC for Series 7-10 during redrying (continued)
Figure 4.7 Equilibrium time plot of gravimetric water content and suction of sample S-10-W3 for suction between 1 and 10kPa
Figure 4.8 SWRC for Series 7-10 during rewetting
Figure 4.8 SWRC for Series 7-10 during rewetting (continued)
Figure 4.9 SWRC for Series 7-10 in hysteresis loop
Figure 4.9 SWRC for Series 7-10 in hysteresis loop (continued)
Figure 4.10 SWRC for Series 7-13 in hysteresis loop
Figure 4.10 SWRC for Series 7-13 in hysteresis loop (continued)
Figure 4.11 SWRC for Series 7-10 and reconstituted sample
Figure 4.11 SWRC for Series 7-10 and reconstituted sample (continued)
CHAPTER 5

SUCTION-MONITORED OEDOMETER TESTS

5.1 Introduction

As shown in Chapter 4, the results of the preliminary double oedometer tests indicated that soil A, compacted dry of optimum, exhibited the collapse-upon-wetting phenomena and was thus suitable for further studies. The next step was to investigate this collapse behaviour in the elasto-plastic framework. Suction-monitored oedometer tests were considered to be a quick and easy means for this purpose. Although only information about the vertical stress and suction can be obtained as the stress variables from these oedometer tests, their results can be beneficially compared with the triaxial test results to gain confidence in the data interpretation.

Suction probes have been incorporated into the stress path oedometer apparatus in tests carried out by Dineen (1997) and Colmenares (2002). This testing technique has been simplified for the conventional oedometer apparatus used in the present studies. A variety of stress paths can nevertheless be followed during the tests. More details of the testing procedures and equipment have been given in Chapter 3. For the tests at high suctions (4000-30000 kPa), the filter paper technique has been used to measure suction. The assumptions regarding the use of this technique will be discussed subsequently.

5.2 Objectives of the tests and testing programme

As discussed in Chapter 2, one of the main assumptions of the elasto-plastic framework that needs validation is the uniqueness of the Loading-Collapse (LC) yield surface, as identified from wetting and loading paths. Moreover, information about the LC yield surface over the whole range of saturation, i.e. from full saturation to the air-dried state, is still scarce.

The testing programme was designed so that these two aspects could be thoroughly examined. It included tests on the three main series of samples (7-10, 7-13 & 5-10), thus enabling a discussion to be made of the influence of fabric and compaction stress on the LC yield surface.
For each series, two main types of tests were carried out, namely constant-water-content loading tests, and constant-vertical-stress wetting tests. For the loading tests, the application of stress was done incrementally, with every subsequent load added after the void ratio and suction had stabilized. The wetting tests involved reducing the suction incrementally by direct addition of small amounts of water to the sample at a constant vertical stress.

The purpose of the wetting tests was to check whether when following a wetting path, the sample would yield on the same LC surface as that identified by the loading tests. In other words, it was to verify the path independency assumption of the LC surface. Admittedly, there is uncertainty regarding this testing technique since the horizontal stress is unknown and the stress state in the wetting tests might not be the same as that in the loading tests in a more generalised stress space (such as p-q space), if the variation of \( K_o \) with suction in the loading and wetting tests differed. Even so, it serves as a valuable preliminary investigation prior to the more rigorous wetting tests in the suction-controlled triaxial apparatus. In the following sections, the test results and interpretation for each of the three series will be presented in turn.

### 5.3 Test series 7-10

Table 5.1 gives the description of the tests performed on the sample Series 7-10, including their as-compacted properties and the stress paths followed. All samples were prepared in the same way, of which details are given in Chapter 3. Each group of these tests, and their results are explained as follows.

#### 5.3.1 Soak & Load Test (7-10-SL)

In this test, the sample was soaked at a vertical stress of 11 kPa to its full saturation, after which loading was continued incrementally. A period of about half to one day was allowed between each load increment. Figure 5.1 shows the specific volume plotted against vertical stress for Test 7-10-SL, together with results from other fully saturated loading tests, which will be described later. Upon soaking, the sample swelled marginally by about 0.20 of specific volume, before yielding at a stress of 65 kPa on loading. The yield stress was estimated using the Cassagrande (1936) method and is
shown in Figure 5.1 as the arrow. This suggests that the sample 7-10-SL was wetted well ‘inside’ the LC yield surface.

### 5.3.2 Constant-water-content loading test

The results of five successful constant-water-content loading tests, namely 7-10-D, G, H, K, & T, are presented here (see Group 2, Table 5.1). In Test 7-10-K, the sample was loaded at its as-compacted water content. In Tests 7-10-D, G & H, however, after compaction, the sample suction was reduced to a desired value by directly spraying water onto it. The Soil-Water Retention Curve identified in Chapter 4 was used as a guide for estimating the amount of water to be added and the corresponding suction that would result. Afterwards, each sample was loaded to a maximum vertical stress of about 3200 kPa in an approximately constant water content condition. In Test 7-10-T, the sample was dried to a moisture content of 9.2% (corresponding to ~940 kPa suction) prior to loading. It was observed that Sample 7-10-T still adhered to the oedometer ring after the initial drying, hence ensuring the K₀ condition was still maintained.

Results from Tests 7-10-D, G, H, K & T are shown in Figures 5.2-5.6 respectively, in terms of vertical displacement, vertical stress, and pore water pressure plotted against time. From the Figures, the decrease/increase in suction can be seen corresponding to each loading/unloading step. In Figure 5.2 the sudden increases in pore water pressure, when the sample was loaded to the vertical stresses of 1717 and 3219 kPa most probably took place in the kaolin paste rather than in the sample, as indicated by the rapid reduction of suction to the equilibrium value. This feature occurred occasionally in some other tests.

The constant water content condition of the sample during loading/unloading in each test was checked by comparing the initial and final water contents. Table 5.1 shows the difference between the two, \(\Delta w/c = w/c_i - w/c_f\), for each test. Considering that the accuracy of water content measurement is about ±0.2 %, it can be suggested that some slight evaporation occurred in tests 7-10-D, G, & H. The evaporation might have happened slowly through the membrane and/or during sample handling before and after the test. When calculating the degree of saturation, the average value of moisture content was used.
Even though the constant water content condition in some tests was only approximate, the suction measurement was considered to be reliable, judged from their relatively stable readings with time seen in Figures 5.2-5.6. In Test 7-10-G, nevertheless, the water content of the sample reduced during the test by 1.5% (see Table 5.1). The suction equilibration plot in Figure 5.3 suggests that evaporation probably worsened during the last stage of unloading as indicated by the steady increase in suction. The reason for this was unclear but believed to be deterioration of the rubber membrane with time, particularly on unloading. The water content was consequently assumed to be constant until unloading started, after which it was assumed to reduce linearly towards the final measured value.

The final values of displacement and suction from each loading/unloading step were then used to prepare the plots of specific volume, degree of saturation, suction and net vertical stress shown in Figures 5.7a, b, and c. In Figure 5.7a, the yield point for each test was estimated using the Cassagrande (1936) method, indicated as an arrow. These yield points were then included in Figure 5.7c, showing the stress paths followed by the samples on a net vertical stress-suction plot. Note that for tests in which two suction probes were employed, the average value of measurement was used, and suctions are plotted with error bars. For tests in which only one probe was used, no error bar is shown.

Through these yield points the Loading-Collapse curve was then drawn, shown in Figure 5.7c. By definition, this curve indicates the boundary across which a large reduction in volume (‘plastic’ strain) commences upon either a reduction in suction or an increase in net stresses. However this boundary is approximate in nature since a number of different methods for identifying the yield point exist and all can give different results or are operator dependent (e.g. Wang & Frost, 2004). The Cassagrande method used here was chosen as a first approximation only. In order to identify the Loading-Collapse characteristic of a soil more fully, the yield surface in suction-vertical stress-specific volume space needs to be constructed. Such a surface will be shown later in Section 5.6 and Chapter 8.
5.3.3 Influence of movement of the stress path in the ‘elastic’ zone

According to the Elasto-Plastic framework, such as the Basic Barcelona model, any movement of the stress path ‘inside’ the Loading-Collapse surface will not affect the position of the surface. Test 7-10-L was carried out to investigate this assumption, and involved wetting the sample at the as-compacted water content and a constant vertical stress of 215 kPa, to a suction of around 140 kPa, a state which was still ‘inside’ the previously identified LC curve. Before the LC curve was reached by wetting, the sample was loaded instead, at a constant moisture content, so the sample crossed the yield curve by loading. The results are shown in Figure 5.8 in terms of vertical displacement, vertical stress, and suction plotted against time. The specific volume plotted against vertical stress and the stress path followed are shown in Figure 5.13. Regarding the degree of saturation during the test, however, the testing technique used did not enable the precise water content change during the wetting test to be monitored. Therefore, no data is presented for the degree of saturation.

It can be seen from Figures 5.8 and 5.13a that upon wetting at a vertical stress of 215kPa, the sample showed practically no change in void ratio. The yield point identified from Figure 5.13a, appears to coincide with the Loading-Collapse curve from the previous loading tests of Group 2, as shown in Figure 5.13c. The result of this simple test was thus believed to confirm the assumption stated above, at least for a monotonic loading case.

5.3.4 Wetting tests

The objective of this group of tests is to compare the Loading-Collapse curve, previously identified from the loading paths in test Group 2 and Test 7-10-L, with the yield points identified from the wetting paths. Four tests were accordingly performed which involved collapse-on-wetting. In these tests, the suction was reduced incrementally by direct addition of water at constant vertical stresses of 108, 215, 430 and 594 kPa in Tests 7-10N, J, I and Q respectively (see Section 3.6 for more details about the testing procedure). The test results are shown in Figures 5.9, 5.10, 5.11, and 5.12, in terms of vertical displacement, vertical stress, and pore water pressure plotted against time.
From these figures, the steady drift of the suction reading during the first stage of loading can be seen and suggests some evaporation from the samples. However, after the sample was wetted, the suction at equilibrium appeared relatively stable. Most importantly, the suctions, at which yielding of the sample took place and those measured afterwards, appear stable, and are believed to be representative of the actual suction of the sample.

The reason for the initial drift of suction was believed to be the fact that the perforated holes in the top cap provided a large space where vapour could escape despite the use of the covering brass top caps (see Section 3.6.2d). The evaporation drift of the suction reading was accentuated especially in tests at high stresses, and high suctions (e.g. 7-10-Q) as a result of non-linearity of the retention curve. In other words, the change in suction will be larger at higher suctions, for a certain amount of moisture change. Nevertheless, after the wetting started, the drift of the suction reading was greatly minimised and virtually disappeared as the sample started yielding. This was because of the relatively moist air prevailing above the perforated holes (coming from the residue of water in the holes, see Figure 3.17) and also because of the non-linearity of the retention curve.

There was also concern regarding the actual equilibrium of the suction over the height of the sample, since both suction measurement and wetting took place at the top surface, and the suction at the base might be different to that at the top even though the top suction appeared to have reached equilibrium. In order to check this, in Ttest 7-10-J, the suction measurement was made on the base of the sample after the reading at the top surface had become stable. As seen in Figure 5.11, the difference ranges from 5-15%. This, together with the relatively small height of the oedometer sample (~20mm), suggests that the suction measured at the top was a reasonable approximation of the average value within the sample. The results from the triaxial tests performed later also confirm the validity of this testing technique.

The final void ratios, suctions, and vertical stresses at each loading & wetting stage were then plotted in Figures 5.13a, b & c. The degree of saturation is again not available due to uncertainties in the amount of water actually absorbed into the sample and the evaporation that occurred before wetting. In Figure 5.13b, the yield points are estimated
as indicated by arrows. These yield points, replotted in Figure 5.13c, together with the stress paths, suggest that the onset of collapse-upon-wetting approximately coincides with the LC yield curve identified by the loading tests.

5.3.5 Constant-water content loading tests at high suctions

Four tests (7-10-P, U, V & W) were carried out with the aim of identifying the LC surface for suctions above 1500 kPa up to the air-dried state, which are beyond the operating range of the suction probe. The method selected for suction measurement was the filter paper technique. Each sample was sealed in the oedometer to ensure constant water content conditions during the test, as described in Section 3.6. The sample was loaded to a maximum vertical stress of 7520 kPa. After the test was finished, the sample was sandwiched between two filter papers for the suction measurement following the procedure explained in Section 3.6.

There was concern regarding the representativeness of the suction measurement, which was made at the end of the test, as opposed to the actual value at the time of yielding, since when the sample yielded, there would be changes in pore structure, which might affect the suction. However, it was impracticable to use the filter paper technique during the test, therefore ruling out the option of measurement at the time of yield. Even so, the inspection of the Soil-Water-Retention Curves in Figures 4.4b, 4.5b & 4.6b suggests that for suctions above 4000 kPa there is a unique relationship between the gravimetric water content and suction, regardless of void ratios of the samples. It is therefore expected that, above 4000 kPa, the suction remained largely constant throughout the constant water content loading tests, and its measurement at the end of test is representative of the value during the test.

The results of Tests 7-10-P, U, V & W are shown in Figure 5.14, plotted in terms of specific volume against net vertical stress, together with the yield points identified using the Cassagrande (1936) method, and the suction measured at the end of each test. It can be seen in Figure 5.14a that as suction increases the vertical yield stress increases.
5.3.6 Summary for oedometer tests on 7-10 series
The two objectives of the testing programme stated earlier have been achieved. The path independency of the yield surface appears to be a valid assumption. The Loading-Collapse surface has also been identified over the entire range of saturation.

5.4 Test series 5-10
The testing technique used with the sample Series 7-10 has been shown to be sufficiently accurate for the test objectives, and thus was also used for Series 5-10. The samples of Series 5-10 were essentially the 7-10 samples that had been further compressed to a lower void ratio, and therefore it would be of interest to compare the Loading-Collapse surfaces of the two series. The path independency of the LC surface of Series 5-10 was also investigated in the same fashion as for Series 7-10. The description of test Series 5-10 is given in Table 5.2, including their initial properties and stress paths followed.

5.4.1 Soak & Load Test (5-10-SL)
As in Test 7-10-SL, Sample 5-10-SL was soaked at a vertical stress of 11 kPa to its full saturation, and loaded/unloaded in the same way. The test results are shown in Figure 5.1. Upon soaking, Sample 5-10 swelled considerably more than Sample 7-10. A possible explanation is that since the structure of Sample 7-10 was more open than that of Sample 5-10, the swelling clay packets of Sample 7-10 could fill up those open spaces, resulting in smaller overall swell than the 5-10 series. Considering that both samples had the same initial suction, as demonstrated in Chapter 4, it follows that they also have differing stiffness parameters for changes in suction.

After saturation, the sample was loaded to a maximum stress of 3300 kPa. The yielding of Sample 5-10-SL appears more gradual than that of Sample 7-10-SL. The Normal Compression Lines for both tests, however, appear to fall on the same line, if the error in specific volume of up to 0.02 can be accepted as a range of experimental error.

5.4.2 Constant water content loading tests
Four successful tests were carried out (5-10B, E, F, & H) (see Group 2 in Table 5.2). After compaction, the water content/suction of each sample was modified as explained earlier for 7-10 series. The modified water content for each test is given in Table 5.2.
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The results for the four tests are given in Figures 5.15-5.18, presented in a similar manner to the 7-10 samples. Note that, due to an electricity cut in the laboratory, the data file of readings during the equilibration time of Test 5-10B was lost. Only the final readings at each stage for this test are available.

Figures 5.18 a, b & c show the plots of specific volume, degree of saturation, and suction against net vertical stress for the four tests. From Figure 5.18a, it is clear that the 5-10 samples are much less collapsible than those of Series 7-10, as would be expected from the denser initial state of Series 5-10. Also included in Figure 5.18a are the yield points estimated using the Cassagrande (1936) method. Figure 5.18c shows the Loading-Collapse curve drawn through these yield points.

5.4.3 Wetting tests

The uniqueness of the Loading-Collapse surface was also investigated for the 5-10 series. Two wetting tests were performed, namely, 5-10D & G (Group 3, Table 5.2), in which suction was reduced at constant vertical stresses of 1184 and 1691 kPa respectively. The same procedure as followed in test series 7-10 was used. The test results of the two tests are shown in Figures 5.19 & 5.20. As in Series 7-10, during the tests, the suction measurement before wetting is not considered truly representative due to evaporation. After wetting up, however, the evaporation from the sample was minimised, and the reading of suction appeared stable, and was thus considered to be a reliable measurement.

The final specific volumes and suctions, plotted against net vertical stress at each loading & wetting stage for the two tests are shown in Figure 5.21. In Figure 5.21c the scatter in the stress path for Test 5-10-D before wetting started was due to evaporation from the sample as seen in Figure 5.19. The yield points were estimated from Figure 5.21b as indicated by the arrows. In Figure 5.21c, these yield points are in a good agreement with the previously identified LC curve from the constant water content loading tests. This observation is the same as that for test series 7-10.

An inspection of Figure 5.21a, however, indicates that, after Sample 5-10-D was wetted to zero suction, its specific volume did not collapse to the fully saturated compression line (Test 5-10-SL), but lies above it by 0.35. Sample 5-10-G, upon wetting to zero
suction, had a specific volume lying above the fully saturated compression line by 0.2. The reason for this is unclear but could be because of the different kind of base plate employed in Test 5-10-D & G and Test 5-10-SL. In Test 5-10-SL, a coarse porous stone was used, whereas a solid plate was used for Tests 5-10-D & G. Some air bubbles might still have been trapped in Samples D and G when the suction reached zero. Nevertheless, the difference might have been contributed to by a scatter in the data due to experimental error.

5.4.4 Constant-water content loading tests at high suction
Following the same technique as used for Series 7-10, three loading tests (Group 4 in Table 5.2, Tests 5-10I, K, & L) were carried out at high suctions. However, after drying out, the 5-10 samples were trimmed to the size of 1.5” in diameter, as opposed to 2” for the 7-10 series, in order that larger stresses could be imposed on the samples during testing and the Normal Compression Lines could be reached.

The results, plotted in terms of specific volume and degree of saturation against net vertical stress, are shown in Figure 5.22, together with the yield points identified using the Cassagrande (1936) method. The same trend can be observed in Figure 5.23a, that as suction increases, the Normal Compression Line of the sample displaces to the right.

5.4.5 Summary for oedometer tests on 5-10 series
Similar observations were made regarding the uniqueness of the Loading-Collapse curve. The 5-10 samples, being denser than the 7-10 samples, have the LC curve displaced more to the right in the suction-net vertical stress plot. They are also less collapsible than the 7-10 samples.

5.5 Test series 7-13
The purpose of this test series 7-13 is to investigate if there was any influence of the as-compacted fabric on the collapsible character of the material. The uniqueness of the Loading-Collapse surface for sample series 7-13 was also examined as in the other two series. The testing programme is summarised in Table 5.3.
5.5.1 Soak & Load Test (7-13-SL)

Figure 5.1 shows that Sample 7-13-SL, fully soaked at a vertical stress of 11 kPa, followed closely the same compression line as that of Sample 7-10-SL. Another fully saturated oedometer test was also carried out on a reconstituted sample, prepared from slurry following the procedure recommended by Burland (1990). The data for this test is also shown together in Figure 5.1. It can be seen that all four samples follow the same intrinsic Normal Compression Line. Nevertheless, there is evidence of structure shown by Samples 7-10-SL and 7-13-SL in the range of vertical effective stress up to 200 kPa, where their compression lines lie above the intrinsic NCL. Samples 7-10-SL and 7-13-SL then appear to destructure towards the reconstituted at the stresses higher than 200 kPa.

5.5.2 Constant water content loading tests

Four tests were carried out (Group 2 in Table 5.3, Tests 7-13-I, J, K & L). Some differences in the testing procedure to that used in the 7-10 series was followed in Test 7-13-K. This sample was initially compacted in a 100mm diameter ring and subsequently dried to a moisture content of about 10%. After drying the sample, it was trimmed into the 3” ring, ready for testing. This was to ensure the $K_o$ condition was maintained after the sample had shrunk upon drying.

The results of the four tests are plotted in the same manner as for the previous series and are shown in Figures 5.23-5.26. The plots of specific volume, degree of saturation, and suction against net vertical stress are shown in Figures 5.27, together with the estimated yield points and Loading-Collapse curve. Also plotted in Figure 5.27c is the LC curve for Series 7-10. By comparison, the Loading-Collapse curve for Series 7-13 appears to be displaced to the right of that for Series 7-10 for suctions higher than 150 kPa. This is an indication of some difference in the collapse characteristics of the two series. More detailed comparisons will be presented in the subsequent sections.

5.5.3 Wetting tests and constant-water content loading tests at high suctions

The uniqueness of the Loading-Collapse surface was also examined for Series 7-13 in order to confirm the observations made earlier for the other two series. Only one wetting test was performed, namely, 7-13-M (see Table 5.2), in which suction was reduced at a constant vertical stress of 303 kPa, using the same procedure as followed in
the other wetting tests. The test results are shown in Figures 5.28 and 5.29. As shown in Figure 5.29b, the uniqueness of the LC curve is again demonstrated by this test.

Two high suction loading tests (7-13D, & G) were carried out, as in the other two series and their results are plotted in Figure 5.30, together with the estimated yield points. The same trend as in other series can be observed.

5.5.4 Summary for oedometer tests on 7-13 series

As shown in Figure 5.27c, the Loading-Collapse curve of Series 7-13 appears to be to the right of the 7-10 LC curve. The results of the high suction loading tests from both Series 7-10 and 7-13 have been also used to construct the LC curves in the higher suction range as shown in Figure 5.31. A similar trend is observed as for the LC curves in the lower suction range.

The reason for this displacement of the LC curve could be the difference of fabric. However, it could also be argued that this displacement might be merely due to the differences in specific volume caused during drying of the 7-13 sample. The specific volumes of the 7-13 samples were about 0.02 lower than those of the 7-10 samples at the same suction and the LC curve is dependent upon the specific volume. Although this difference in specific volume could fall within the accuracy of the measurements, it is very difficult to compare the Loading-Collapse character of the two series based on the LC curves alone. The Loading-Collapse surfaces need be constructed to make the comparison more realistic. These are presented and discussed in the next section.

5.6 Comparison between the Loading-Collapse surfaces of the three series

The Loading-Collapse surface can be described as consisting of a series of Normal Compression Lines at different suctions. The oedometer tests were, however, not conducted at constant suctions and thus an estimate of the contour lines was needed in order to construct the surface. As a first approximation, the data beyond the yield points identified earlier was considered to represent the LC surface. The contour was produced from these data using the Kriging Gridding method in the software Surfer©. More details on how the contours were constructed are presented in Appendix 3.
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Figure 5.32 shows the contours of constant suction NCLs for Series 7-10. Included in the plots are the data points used to create the contours. The NCLs for suctions between 10 and 100 kPa appear to be slightly curved and converge towards the fully saturated (zero suction) NCL at high vertical stresses. It is expected that the NCLs for higher suction would also converge if the samples had been loaded to high enough stresses.

The contours of NCLs for Series 7-13 are shown in Figure 5.33. The lines again appear to be slightly curved. By comparison, it can be seen that the NCLs for series 7-13 lie above those of series 7-10 for suctions up to 10000 kPa. For suctions exceeding 10000 kPa, the Loading-Collapse surfaces for both series seem to coincide. This observation indicates the influence of fabric on the Loading-Collapse behaviour. It can be speculated that the 7-13 fabric type (larger aggregates) yields samples with a greater bonding strength between the silt/sand grains and clay packets, and thus their Loading-Collapse curve locates to the right of that for the 7-10 series at the same void ratio. However, this influence of fabric seems to lessen once the suction exceeds 10000 kPa and degree of saturation fall below about 10%.

The contours for the series 5-10 is shown in Figure 5.34. The fitting of the contour lines to the test results is not as good as the contours for the previous two series. This is because the settlements involved in test series 5-10 are much smaller and fall within the range of accuracy of the measurement of specific volume (about 0.02). Nevertheless, the trend is similar to that of the previous two series, and the compression lines tend towards the fully saturated compression line at high stresses. The 5-10 contour lines also appear to be in a good agreement with those of series 7-10. More quantitative comparisons between the different series will be given in Chapter 8.
### Table 5.1 Summary of oedometer tests on 7-10 series

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
<th>As-compacted properties</th>
<th>Stress paths followed after compaction</th>
<th>Diff. w/c, %</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7-10-SL</td>
<td>0.70 10.4</td>
<td>1) Soaked at a vertical stress of 11 kPa. 2) Loaded incrementally to 7336 kPa $\sigma_v$. 3) Unloaded to 11 kPa $\sigma_v$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>7-10-D</td>
<td>0.71 10.2</td>
<td>1) Wetted to 10.6 %w/c at zero kPa $\sigma_v$. 2) Loaded to 3220 kPa $\sigma_v$. 3) Unloaded to 54 kPa</td>
<td>+0.38</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>7-10-G</td>
<td>0.70 10.1</td>
<td>1) Wetted to 14.8 %w/c at zero kPa $\sigma_v$. 2) Loaded to 3220 kPa $\sigma_v$. 3) Unloaded to 54 kPa</td>
<td>+1.50</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>7-10-H</td>
<td>0.69 10.2</td>
<td>1) Wetted to 13.5 %w/c at zero kPa $\sigma_v$. 2) Loaded to 3220 kPa $\sigma_v$. 3) Unloaded to 54 kPa</td>
<td>+0.54</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>7-10-K</td>
<td>0.72 10.6</td>
<td>1) Loaded to 3220 kPa $\sigma_v$ at as-compacted water content 2) Unloaded to 54 kPa</td>
<td>-0.19</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>7-10-T</td>
<td>0.69 10.0</td>
<td>1) Dried to 9.2 % w/c at zero kPa $\sigma_v$. 2) Loaded to 3220 kPa $\sigma_v$. 3) Unloaded to 54 kPa</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td>7-10-L</td>
<td>0.73 10.1</td>
<td>1) Loaded to 215 kPa $\sigma_v$ at as-compacted water content 2) Wetted to suction of ~ 140 kPa at 215 kPa $\sigma_v$. 3) Loaded to 3220 kPa $\sigma_v$. 4) Unloaded to 54 kPa</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>7-10-I</td>
<td>0.71 10.3</td>
<td>1) Loaded to 430 kPa $\sigma_v$ at as-compacted water content 2) Wetted to suction of ~ 130 kPa at 430 kPa $\sigma_v$. 3) Loaded to 3220 kPa $\sigma_v$. 4) Unloaded to 54 kPa</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>7-10-J</td>
<td>0.71 10.3</td>
<td>1) Loaded to 215 kPa $\sigma_v$ at as-compacted water content 2) Wetted to suction of ~ 51 kPa at 215 kPa $\sigma_v$. 3) Sample collapsed. 3) Loaded to 861 kPa $\sigma_v$. 4) Unloaded to 54 kPa</td>
<td>-</td>
<td>Top measurement compared with base</td>
</tr>
<tr>
<td>10</td>
<td>7-10-N</td>
<td>0.71 10.0</td>
<td>1) Loaded to 108 kPa $\sigma_v$ at as-compacted water content 2) Wetted to suction of ~ 10 kPa at 108 kPa $\sigma_v$. 3) Sample collapsed. 3) Unloaded to 54 kPa</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>7-10-Q</td>
<td>0.70 10.2</td>
<td>1) Loaded to 594 kPa $\sigma_v$ at as-compacted water content 2) Wetted to suction of ~ 40 kPa at 594 kPa $\sigma_v$. 3) Sample collapsed. 3) Unloaded to 54 kPa</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Diff. w/c, %, is the difference between the water content prior to loading and the final water content, w/c_i-w/c_f, indicating the accuracy of the constant water content condition.
### Table 5.1 Summary of oedometer tests on 7-10 series (continued)

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
<th>As-compacted properties</th>
<th>Stress paths followed after compaction</th>
<th>Diff. Water content, %*</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>7-10-P</td>
<td>0.69 10.1</td>
<td>1) Dried to air-dried state (0.81 % w/c) at zero kPa σv  2) Loaded to 7523 kPa σv  3) Unloaded to 125 kPa</td>
<td>+0.06</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-10-U</td>
<td>0.70 10.1</td>
<td>1) Dried to 3.9 % w/c at zero kPa σv  2) Loaded to 7526 kPa σv  3) Unloaded to 125 kPa</td>
<td>+0.43</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-10-V</td>
<td>0.70 10.2</td>
<td>1) Dried to 4.3 % w/c at zero kPa σv  2) Loaded to 7523 kPa σv  3) Unloaded to 125 kPa</td>
<td>+0.81</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-10-W</td>
<td>0.70 10.1</td>
<td>1) Dried to 2.0 % w/c at zero kPa σv  2) Loaded to 7523 kPa σv  3) Unloaded to 125 kPa</td>
<td>-0.10</td>
<td>-</td>
</tr>
</tbody>
</table>

*Diff. w/c, %, is the difference between the water content prior to loading and the final water content, w/c<sub>f</sub>-w/c<sub>i</sub>, indicating the accuracy of the constant water content condition.
### Table 5.2 Summary of oedometer tests on 5-10 series

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
<th>As-compacted properties</th>
<th>Stress paths followed after compaction</th>
<th>Diff. w/c, % *</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5-10-SL</td>
<td>0.51 10.2</td>
<td>1) Soaked at a vertical stress of 11 kPa. 2) Loaded incrementally to 3240 kPa $\sigma_v$. 3) Unloaded to 54 kPa $\sigma_v$.</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>5-10-B</td>
<td>0.51 9.8</td>
<td>1) Wetted to 13.5 % w/c at zero kPa $\sigma_v$. 2) Loaded to 3220 kPa $\sigma_v$. 3) Unloaded to 54 kPa.</td>
<td>+0.53</td>
<td>Data for equilibration time was lost</td>
</tr>
<tr>
<td>2</td>
<td>5-10-E</td>
<td>0.51 10.0</td>
<td>1) Wetted to 11.8 % w/c at zero kPa $\sigma_v$. 2) Loaded to 3232 kPa $\sigma_v$. 3) Unloaded to 54 kPa.</td>
<td>+0.28</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>5-10-F</td>
<td>0.49 9.9</td>
<td>1) Wetted to 10.9 % w/c at zero kPa $\sigma_v$. 2) Loaded to 3238 kPa $\sigma_v$. 3) Unloaded to 54 kPa.</td>
<td>+0.20</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>5-10-H</td>
<td>0.50 10.1</td>
<td>1) Dried to 9.7 % w/c at zero kPa $\sigma_v$. 2) Loaded to 3239 kPa $\sigma_v$. 2) Unloaded to 54 kPa.</td>
<td>+0.10</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>5-10-D</td>
<td>0.50 10.0</td>
<td>1) Loaded to 1184 kPa $\sigma_v$ at as-compacted water content. 2) Wetted to zero suction at 1184 kPa $\sigma_v$; sample collapsed. 3) Unloaded to 54 kPa.</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>5-10-G</td>
<td>0.50 10.0</td>
<td>1) Loaded to 1619 kPa $\sigma_v$ at as-compacted water content. 2) Wetted to zero suction at 1619 kPa $\sigma_v$; sample collapsed. 3) Unloaded to 54 kPa.</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>5-10-I</td>
<td>0.51 10.1</td>
<td>1) Dried to air-dried state (1.2 % w/c) at zero kPa $\sigma_v$. 2) Loaded to 13 MPa $\sigma_v$. 3) Unloaded to 223 kPa.</td>
<td>+0.20</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>5-10-K</td>
<td>0.49 10.1</td>
<td>1) Dried to 4.0 % w/c at zero kPa $\sigma_v$. 2) Loaded to 13.4 MPa $\sigma_v$. 3) Unloaded to 223 kPa.</td>
<td>+0.51</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>5-10-L</td>
<td>0.49 10.1</td>
<td>1) Dried to 2.2 % w/c at zero kPa $\sigma_v$. 2) Loaded to 13.4 MPa $\sigma_v$. 3) Unloaded to 224 kPa.</td>
<td>+0.05</td>
<td>-</td>
</tr>
</tbody>
</table>

*Diff. w/c, %, is the difference between the water content prior to loading and the final water content, w/ci-w/cf, indicating the accuracy of the constant water content condition.
### Table 5.3 Summary of oedometer tests on 7-13 series

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
<th>As-compacted properties</th>
<th>Stress paths followed after compaction</th>
<th>Diff. w/c, % *</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7-13-SL</td>
<td>0.71 13.6</td>
<td>1) Soaked at a vertical stress of 11 kPa. 2) Loaded incrementally to 3215 kPa $\sigma_v$. 3) Unloaded to 54 kPa $\sigma_v$</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>7-13-I</td>
<td>0.71 13.5</td>
<td>1) Loaded to 3235 kPa $\sigma_v$ at as-compacted water content 2) Unloaded to 54 kPa</td>
<td>+0.23</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-13-J</td>
<td>0.70 13.6</td>
<td>1) Dried to 12.6 % w/c at zero kPa $\sigma_v$, 2) Loaded to 3214 kPa $\sigma_v$, 3) Unloaded to 54 kPa</td>
<td>+0.23</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-13-K</td>
<td>0.72 13.7</td>
<td>1) Dried to 9.9 % w/c at zero kPa $\sigma_v$, 2) Loaded to 3238 kPa $\sigma_v$, 3) Unloaded to 54 kPa</td>
<td>+0.13</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-13-L</td>
<td>0.70 13.6</td>
<td>1) Wetted to 15.1 % w/c at zero kPa $\sigma_v$, 2) Loaded to 3239 kPa $\sigma_v$, 2) Unloaded to 54 kPa</td>
<td>+0.43</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>7-13-M</td>
<td>0.71 13.5</td>
<td>1) Loaded to 303 kPa $\sigma_v$ at as-compacted water content 2) Wetted to zero suction at 303 kPa $\sigma_v$; sample collapsed 3) Unloaded to 54 kPa</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>7-13-D</td>
<td>0.71 13.9</td>
<td>1) Dried to air-dried state (1.3 % w/c) at zero kPa $\sigma_v$, 2) Loaded to 7.5 MPa $\sigma_v$, 3) Unloaded to 125 kPa</td>
<td>+0.06</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>7-13-G</td>
<td>0.70 13.7</td>
<td>1) Dried to 2.89 % w/c at zero kPa $\sigma_v$, 2) Loaded to 7.5 MPa $\sigma_v$, 3) Unloaded to 126 kPa</td>
<td>+0.82</td>
<td>-</td>
</tr>
</tbody>
</table>

*Diff. w/c, %, is the difference between the water content prior to loading and the final water content, w/c_i-w/c_f, indicating the accuracy of the constant water content condition
Figure 5.1 Specific volume against vertical stress (effective stress, after soaking) for fully saturated oedometer tests
Figure 5.2 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-D
Figure 5.3 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-G
Figure 5.4 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-H
Figure 5.5 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-K
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Figure 5.6 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-T
Figure 5.7 Results from oedometer tests 7-10-SL, D, G, H, K & T
Figure 5.7 Results from oedometer tests 7-10-SL, D, G, H, K & T (continued)
Figure 5.8 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-L
Figure 5.9 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-N
Figure 5.10 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-J
Figure 5.11 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-I
Figure 5.12 Plots of vertical displacement, vertical stress and suction with time from Test 7-10-Q
Figure 5.13 Results from oedometer tests 7-10-SL, L, N, J, I & Q
Figure 5.13 Results from oedometer tests 7-10-SL, L, N, J, I & Q (continued)
Figure 5.14 Results from oedometer tests 7-10-SL, V, P, W & U
Figure 5.15 Plots of vertical displacement, vertical stress and suction with time from Test 5-10-E
Figure 5.16 Plots of vertical displacement, vertical stress and suction with time from Test 5-10-F
Figure 5.17 Plots of vertical displacement, vertical stress and suction with time from Test 5-10-H
Figure 5.18 Results from oedometer tests 5-10-SL, B, E, F & H
Figure 5.18 Results from oedometer tests 5-10-SL, B, E, F & H (continued)
Figure 5.19 Plots of vertical displacement, vertical stress and suction with time from Test 5-10-D
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Figure 5.21 Results from oedometer tests 5-10-SL, D & G
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Figure 5.21 Results from oedometer tests 5-10-SL, D & G (continued)
Figure 5.22 Results from oedometer tests 5-10-SL, I, K & L.
Figure 5.23 Plots of vertical displacement, vertical stress and suction with time from Test 7-13-I
Figure 5.24 Plots of vertical displacement, vertical stress and suction with time from Test 7-13-J
Figure 5.25 Plots of vertical displacement, vertical stress and suction with time from Test 7-13-K
Figure 5.26 Plots of vertical displacement, vertical stress and suction with time from Test 7-13-L
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Figure 5.27 Results from oedometer tests 7-13-SL, I, J, K & L

a)

b)
Figure 5.27 Results from oedometer tests 7-13-SL, I, J, K & L (continued)
Figure 5.28 Plots of vertical displacement, vertical stress and suction with time from Test 7-13-M
Figure 5.29 Results from oedometer tests 7-13-SL & M
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Figure 5.31 Comparison between the Loading-Collapse curves for Series 7-10 and 7-13
Figure 5.32 Contour of Normal Compression Lines at constant suctions for Series 7-10
Figure 5.33 Contour of Normal Compression Lines at constant suctions for Series 7-13
Figure 5.34 Contour of Normal Compression Lines at constant suctions for Series 5-10
Chapter 6 – Fabric studies

CHAPTER 6
FABRICS STUDIES

6.1 Introduction

Chapter 4 shows that a number of factors, including the as-compacted properties, stress history, and wetting/drying history, can have a significant influence on the soil-water retention curves. All these factors affect the fabric and pore size distribution of the material, which in turn control its drying and wetting behaviour.

The Loading-Collapse behaviour was shown in Chapter 5 to be affected by the fabric induced at as-compacted water contents. Differing parameters would be needed in order to model samples compacted at different water contents and a similar void ratio. This chapter describes a qualitative study of the fabrics, through petrological micrographs, aiming to supplement and confirm some of the observations and hypotheses made in the previous chapters.

6.2 Background

It has been shown by Barden et al. (1973) that collapse in a soil, whether it is a sand, a silt or a clay, is related to its open metastable structure, which is of a bulky granular type. In the case of clays, the granular ‘grains’ are composed of aggregates of clay plates or ‘packets’. The collapsible silt or sand grains also normally have clays as a dominant bonding agent, which clothes the surface of the grains. Where concentrated in a local area, the clays give a buttress type of support to the bulky grains.

Burland (1965) and Burland & Ridley (1996) highlighted this, as shown in Figure 6.1. A highly idealised mechanistic model of a partly saturated silty-clay consists of silt grains and ‘packets’ of clay particles bonded together by high curvature menisci of water. Volume change can take place as a result of contact slip between grains and/or ‘packets’ as well as by shearing, swelling and shrinkage of the ‘packets’ themselves.

It is thus of interest to investigate the actual arrangement of these grains and ‘packets’ in Soil ‘A’ and to correlate them with the observed behaviour. The wetting/drying behaviour is also related to the distribution of the pores, including the macro- and
micro-pores. A petrological microscope study was carried out; chosen on the basis of the scale of interest (i.e. that of the sand/silt particle and clay packets size) and its availability in the laboratory.

6.3 Experimental programme

Five thin sections of different samples were prepared for the study. A summary of the sample properties is presented in Table 6.1. As explained in earlier chapters, all these five samples behaved very differently and each one can be considered to represent its own type of fabric.

<table>
<thead>
<tr>
<th>sample</th>
<th>slide</th>
<th>as-compacted properties</th>
<th>void ratio, $e$, at time of impregnation</th>
<th>w/c, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-7-10-D</td>
<td>A</td>
<td>0.708 10.0</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td>reconstituted</td>
<td>B</td>
<td>- 1.5LL</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>5-10-G</td>
<td>D</td>
<td>0.498 10.0</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>S-7-10-W2</td>
<td>E</td>
<td>0.707 10.4</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>S-7-13-D</td>
<td>F</td>
<td>0.678 13.5</td>
<td>0.66</td>
<td></td>
</tr>
</tbody>
</table>

Table 6.1 Summary of samples used for fabric studies

The procedures of sample impregnation are summarised in Figure 6.2, and follow those recommended by Murphy (1986). All samples were oven-dried at 40°C, before being impregnated with polyester resin. The polyester resin was diluted with acetone before mixing with dye and the catalyst. Red dye was used first in order to help with the identification of pores under the microscope. The proportion of resin, catalyst, dye, and solvent is summarised in Table 6.2. Before impregnation, the sample was kept in a desiccating container (i.e. a container filled with dried silica gel) to prevent it from absorbing moisture from the atmosphere.

Each sample was then submerged in a bath of resin under a small vacuum for a few hours. The vacuum should not be too large that the acetone boils and this was confirmed by placing a dummy resin mixture in the chamber. No bubbles should come out of the
dummy mixture during evacuation. After the evacuation, the sample was cured in an oven at 40°C for several days. Once relatively hard, the sample was cut in half to investigate the impregnation. Upon bisecting, all the samples appeared coherent (i.e. all the particles were cemented with resin) and no particles were dislodged under running water. Even so a second impregnation was carried out using resin dyed with blue stain to ensure complete impregnation.

<table>
<thead>
<tr>
<th>Polyester-resin Resin ‘A’ Scott Bader (an 1866)</th>
<th>Catalyst</th>
<th>Acetone</th>
<th>Dye</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 g</td>
<td>1 ml</td>
<td>40-50 ml</td>
<td>~0.025 g</td>
</tr>
</tbody>
</table>

* Resin starts gelling in approx. 2 hrs

Table 6.2 Proportion of resin mixture

The red resin impregnated the finer pores. However some air still existed in the larger pores, which were later filled with blue resin. Indeed, the proportion of red to blue resin absorbed depended on the location within the sample, whether it is at the centre of the sample or around the edge, possibly due to non-uniformity of resin wetting. Even so, this technique proves particularly useful in qualitatively identifying the micro- and macro- pores, provided that consistent technique and interpretation is used for all samples.

6.4 Identification of mineralogy of particles

Soil ‘A’ is an artificial soil made of 70% HPF4 silt, 20% Kaolin, and 10% London clay. Therefore, the composition and mineralogy of the material is fairly well known. HPF4 silt is mainly composed of Quartz; the kaolin of Kaolinite, and the London clay of Illite and Montmorillonite with few accessory minerals. Zdravkovic (1996) studied the particle shape of HPF4 and concluded that the HPF4 silt particles are highly angular and elongated.

Quartz

The quartzitic particles appear colourless under plane-polarised light. Their size varies from that of fine sand to silt (see the grain-size distribution curve in Figure 3.3). The individual fine silt size particles are more difficult to detect than the sand size particles, since they are frequently coated with clay. Quartz mineral is considered to have a low
birefringence, Gribble & Hall (1992). The birefringence is a property, which indicates the variation of colours of an anisotropic mineral under crossed polars. Such variations are called the interference colours. The same mineral shows different colours depending on its crystallographic orientation. On the other hand, isotropic minerals, such as resins, remain dark under crossed polars whatever their optical orientation.

In the present studies, with the section thickness of 60µm, quartz grains have the maximum interference colours of first order, from white, yellow to red. The anisotropism of Quartz also means that the particles go in to extinction four times during a complete rotation of the section under crossed polars. These properties help in distinguishing between quartz and other features or particles.

Clay minerals
It is impossible to see an individual clay-sized particle under an optical microscope. However, in this study, the clays were almost invariably impregnated with red resin and they appeared orangish to dark brown under plane-polarized light. Under crossed-polarization, they did not exhibit any extinction angle.

Clay minerals are most readily identifiable under reflected light mode due to their high reflectance. Their flaky structures reflect light and appear bright, whereas the quartz and resin do not. This property is particularly useful in confirming the presence of clay-silt aggregations.

6.5 Fabric of Sample S-7-10-D (Slide A)
A conceptual model of particle arrangements was proposed by Collins & McGown (1974), and consists of 3 types of features. These include ‘elementary particle arrangements’, ‘particle assemblages’, and ‘pore spaces’ as shown in Figure 6.3, 6.4, and 6.5 respectively. The nomenclatures used in this study of fabrics will be in line with these definitions.

Figure 6.6 shows a typical micrograph of slide A under plane-polarized light and under crossed-polarized light. There is evidence of multi-structural levels. A variety of particle assemblages, as illustrated conceptually in Figure 6.4, are present; namely the regular
aggregations of silt and clay; connectors or clay buttress; irregular aggregations, as well as the granular matrix. Figure 6.6b also confirms the presence of clay minerals in the silt-clay aggregations, viewed under reflected light. An interpretation of the micrograph is shown in Figure 6.6c. The inter-assemblage pore space (as conceptually shown in Figure 6.5) is generally filled with blue resin. These multi-structural levels confirm the hypothesis made regarding the bi-modality of the soil-water retention curve and the presence of micro- and macro-structural levels. Moreover, it demonstrates the open structure, which is a significant feature of collapsible materials.

6.6 Fabric of reconstituted sample (Slide B)

Slide B is from a reconstituted sample prepared in a conventional way, that of remoulding a sample at a water content of 1.5 LL%. The sample was consolidated in a conventional oedometer to a void ratio of 0.70, after which it was then air-dried to a void ratio of about 0.48.

The micrograph in Figure 6.7 shows a fabric that is far more uniform than that of slide A. The fabric can be described as a uniform matrix of well-mixed clay, silt, and sand particles. No aggregations, nor intra-assemblage pores, can be observed at this scale (Figure 6.7c). This in fact confirms the definition of material reconstitution as suggested by Burland (1990), and studied further by Fearon & Coop (2000).

6.7 Fabric of Sample 5-10-G (Slide D)

Sample 5-10-NG followed a more complicated stress history. The sample was compacted to a void ratio of 0.50 and a water content of 10%, then loaded to a vertical stress of 1620kPa, soaked to zero suction, and unloaded. The final void ratio was about 0.48, very similar to the reconstituted sample in slide B. However, the micrograph of slide D, shown in Figure 6.8, indicates that its fabric is significantly different to that of the reconstituted sample. The regular silt-clay aggregations appear uniformly throughout the sample, together with a dense matrix of granular particles (Figure 6.8c). As would be expected from its low void ratio, there are very few large macro pores.

6.8 Fabric of Sample S-7-10-W2 (Slide E)

Sample S-7-10-W2 was soaked to a suction of 1kPa from the as-compacted state in an unconfined condition and followed by drying. Upon drying, the void ratio reduced to a
value lower than the as-compacted state and the SWRC became more uni-modal. More details of the stress paths followed can be found in Chapter 4.

A micrograph of slide E is shown in Figure 6.9. Under plane polarized light, evidence of aggregation is almost absent. The clay appears to be interspersed all over the granular grains when seen in the reflected light mode. Some irregular aggregations of silt & clay are, however, partly discernible, but their boundaries are not clear (Figure 6.9c). This fabric is therefore less multi-structural than that of slide A (as-compacted structure), while being more aggregate-like than that of slide B (reconstituted structure). This observation confirms the hypothesis made earlier that the silt-clay aggregates expanded during soaking and thus replaced the intra-assemblage pore space. This consequently reduced the multi-structural fabric and hence the bimodality of the SWRCs. The observation also suggests that the influence of reducing suction to nearly zero is not fully equivalent to reconstituting the sample in a conventional way.

6.9 Fabric of sample S-7-13-D (slide F)
Sample S-7-13D followed a similar stress path to that of Sample S-7-10D with the only difference being its initial properties. The fabric of slide F is practically representative of the sample Series 7-13 at their as-compacted state. Even visual inspection could identify that the sample series 7-13 was composed of larger aggregates than the Series 7-10.

This observation becomes clearer at the microscopic level as shown in Figure 6.10. At a similar void ratio, Sample 7-13, compacted at a higher water content, has more regular and larger aggregations of silt and clay than Sample 7-10. The presence of inter-assemblage pore space, or macro- pores, is also more frequent in Sample 7-13. As a corollary, this suggested that while the overall void ratio of both Series 7-10 and 7-13 is similar, the intra-aggregate void ratio of Sample 7-13 might be smaller than that of Sample 7-10.

This observation explains the difference in SWRCs observed for Series 7-13 and 7-10. As shown in Figure 4.5, at the macro level of the retention curves, the curve for the 7-13 series is displaced to the left, which suggests a larger macro- pores than that for 7-10. In addition, the displacement of the Loading-Collapse surface for the 7-13 series to higher
net stresses, as shown in Section 5.6, might be explained by a speculation that the aggregates in the 7-13 samples were denser at the same suction and overall void ratio, and thus a larger stress is needed to displace them. However, more studies are needed before any conclusions can be drawn regarding the mechanism that governs the influence of fabric on the Loading-Collapse surface.

6.10 Conclusions
Observations from a petrological microscopic study confirm the conclusions reached for the soil-water retention curves and the results for the oedometer tests. The evidence of aggregations and the multi-structural level can be clearly seen for the samples in their as-compact ed states (Slides A & F). The as-compact ed water content appears to govern the average size of the aggregations and the distribution of the macro- pores. This in turn influences the SWRC at low suctions (0-100kPa).

The various stress paths followed by a sample after compaction can influence the fabric in a variety of ways. For example, the influence of reducing the suction to zero in an unconfined condition (Slide E) was to swell and, to a certain extent, destroy the silt-clay aggregations. However, the destructuration of the aggregates is not as vigorous as that obtained with the conventional reconstitution method (Slide B). In addition, by wetting an already densely compacted sample (Slide D), the aggregates swelled but did not disappear. This may be because of the lack of inter-assemblage pore space or macro-pores for the aggregates to fill in.
Figure 6.1 Idealised mechanistic model of a partly saturated silty clay (Burland & Ridley, 1996)
a) The sample was contained in an aluminium foil basin. The space between the basin and sample was minimised to avoid cracks due to resin shrinkage.

b) After the red-dyed resin mixture had been made, it was then poured down the side to half-cover the sample. A period of about 5 minutes was waited so that the resin could fill up the pores through capillary action.

c) More resin was poured so that the sample was slightly submerged. It was then transferred to a vacuum chamber and a small vacuum was applied. Note the bubbles coming out during the first part of evacuation. The evacuation was then carried out for several hours. Afterwards, the sample was cured in an oven at 40°C for several days.

d) The bisected sample was impregnated again with blue-dyed resin. It was then stuck to a glass plate and ground down to a thickness of about 60 µm. This thickness was suggested by Coop (2004) as more information can be obtained than from the standard thickness of 30 µm. As the thickness reached about 100 µm, the grinding was carried out by hand.

Figure 6.2 Procedure for sample impregnation
Figure 6.3 Schematic representation of elementary particle arrangements (Collins and McGown, 1974). a) Individual clay platelet interaction. b) Individual silt or sand particle interaction. c) Clay platelet group interaction. d) Clothed silt or sand particle interaction. e) Partly discernible particle interaction.
Figure 6.4 Schematic representations of particle assemblages; (Collins and McGown, 1974) (a) (b) (c) connectors; (d) irregular aggregations linked by connector assemblages; (e) irregular aggregations forming a honeycomb arrangement; (f) regular aggregations interacting with silt or sand grains; (g) regular aggregation interacting with particle matrix; (h) interweaving bunches of clay; (j) interweaving bunches of clay with silt inclusions;(k) clay particle matrix: (l) granular particle matrix
Figure 6.5 Schematic representation of pore space types (Collins and McGown, 1974)
Figure 6.6 Micrograph of compacted Sample S-7-10-D: Slide A. a) under plane-polarized light. b) under reflected light. c) interpretation
Figure 6.6 Micrograph of compacted Sample S-7-10-D: Slide A. a) under plane-polarized light. b) under reflected light. c) interpretation (continued)
**Figure 6.6** Micrograph of compacted Sample S-7-10-D: Slide A. a) under plane-polarized light. b) under reflected light. c) interpretation
Figure 6.7 Micrograph of the reconstituted sample: Slide B. a) under plane-polarized light. b) under reflected light. c) interpretation
Figure 6.7 Micrograph of the reconstituted sample: Slide B. a) under plane-polarized light. b) under reflected light. c) interpretation (continued)
Figure 6.7 Micrograph of reconstituted sample: Slide B. a) under plane-polarized light. b) under reflected light. c) interpretation
Figure 6.8 Micrograph of Sample 5-10-G: Slide D. a) under plane-polarized light. b) under reflected light. c) interpretation
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CHAPTER 7

TRIAXIAL TESTS: fully saturated and suction-controlled tests

7.1 Introduction

The central objective of this research was to investigate the partly saturated behaviour of compacted Soil A in the suction-controlled triaxial apparatus. The main testing programme was intended to explore both volumetric and shearing behaviours of compacted samples of Soil A from Series 7-10. Regarding the volumetric behaviour, the collapse of the compacted samples was studied under various stress paths: isotropic compression, anisotropic compression, $K_o$-compression, as well as wetting at constant net stresses. The shearing behaviour was investigated only in compression (deviatoric stress, $q > 0$) at various suctions and cell pressures.

A series of conventional stress path triaxial tests was also carried out on fully saturated samples of Soil A. Both reconstituted and compacted samples were tested. The test objective was to identify the soil behaviour at full saturation, and also to highlight the differences between the behaviours of the reconstituted and the compacted samples. This chapter presents first the test results for fully saturated samples, which are then followed by those for partly saturated samples. More in-depth interpretation of the results with reference to elasto-plastic frameworks will be given in the next chapter.

7.2 Fully saturated triaxial tests

7.2.1 Objectives of the tests and testing programme

Cunningham (2000) carried out a series of triaxial tests on the fully saturated reconstituted samples of Soil A. During the first year of the present research, a triaxial apparatus was just set up and various tests were carried out in order to confirm the results of Cunningham as well as to refine his observations. Two additional triaxial tests were also performed on compacted Soil A from Series 7-10 and 7-13 in their fully saturated states. This enabled a comparison to be made between the fully saturated behaviours of the reconstituted and the compacted samples. Table 7.1 gives a description of all the fully saturated triaxial tests.
7.2.2 Tests on reconstituted samples
As shown in Table 7.1, seven triaxial tests were carried out on reconstituted samples of Soil A. The sample preparation method used for all reconstituted samples was the same as that of Cunningham (2000). All samples were trimmed from the cake sample to the size of 38mm in diameter, apart from Sample TR7, which was trimmed to a diameter of 50mm. In Test TR7, direct radial strain measurement was made using the radial strain belt. Local axial strains were measured in all tests, using the inclinometers. Details of the experimental procedures have already been given in Section 3.7.3.

7.2.3 Isotropic and $K_o$ compression
The isotropic compression behaviour was mainly investigated in Sample TR6, which was compressed isotropically to a mean effective stress, $p'$, of 650 kPa before shearing. Other samples were compressed to lower stresses. The compression paths are shown in Figure 7.1. Tests TR3 and TR5 involved $K_o$ compression, but did not reach a stress that was high enough to construct the virgin $K_o$ compression line. During $K_o$ compression, the radial strain was calculated using the axial strains from the inclinometers, and the volumetric strain from the volume gauge. This radial strain was maintained within $\pm 0.002\%$ by the procedure explained in Section 3.7.3. The paths followed by Samples TR3 and TR5 are shown on a $q-p'$ plot in Figure 7.2.

After consolidation, most samples were either sheared drained or undrained. The sequence of shearing rates used has been explained in Section 3.7.3d. Sample TR5 was sheared by reducing mean effective stress, $p'$, while maintaining a constant deviatoric stress, $q$. Figures 7.3 and 7.4 show the paths followed by all samples during compression and shearing. In the next section, the behaviour of each sample during shearing will be explained in detail.

7.2.4 Shearing behaviour of the reconstituted samples

$a)\ TR1$
Due to an electricity failure, information during the compression of Sample TR1 was lost and the control system malfunctioned. As a result, the stress history of the sample prior to shearing is not clearly known. From Figure 7.4, it might be inferred that Sample TR1 had been lightly overconsolidated before undrained shearing. The stress path in
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Figure 7.3a shows that the phase transformation occurred at a stress ratio slightly below that at the critical state. The stress-strain relationship during shearing is shown in Figure 7.5. The axial strains plotted are the average values from the two inclinometers. This sample failed in a barrelling mode.

b) TR2

Sample TR2 was sheared drained, after isotropic compression to a $p'$ of 100kPa. Since the sample had been $K_o$ compressed in the 7” oedometer to an effective vertical stress of 200kPa, before tested in the triaxial apparatus, it was only lightly overconsolidated prior to shearing. The sample volume accordingly reduced towards the critical state (Figure 7.4a). Figure 7.6 shows the stress-strain relationship during shearing. The sample bulged noticeably into a barreled shape upon failure.

c) TR3

Sample TR3 was $K_o$ compressed before being sheared undrained. As shown in Figures 7.3a and 7.7, the failure mode of the sample was noticeably brittle, as would normally be observed during undrained shearing of an anisotropically consolidated low plasticity clay (e.g. Gens, 1982).

d) TR4

After isotropic compression to a $p'$ of 200 kPa, Sample TR4 was sheared under undrained conditions. The sample failed in a contractant mode as indicated by the rise in pore water pressure upon shearing (Figure 7.8). A slight decrease in pore water pressure towards the end of shearing might be due to a marginal overconsolidation of the sample. Such an overconsolidation might have resulted from sample disturbance or drying during its preparation.

e) TR5

Test TR5 underwent $K_o$ compression to a $p'$ of 67 kPa before shearing. The sample was sheared by reducing $p'$, while maintaining a constant $q$ (Figure 7.3a). Figure 7.9 shows the stress-strain relationship during shearing. Once the sample reached the critical state line on a $p'$-$q$ plot that had been already identified from the previous tests, $p'$ was reduced while maintaining a constant rate of strain, instead of a constant $q$. The sample then onwards followed the critical state line.
TR6
Test TR6 was isotropically consolidated to a $p'$ of 650kPa before shearing (Figures 7.3b & 7.4). The normally consolidated sample showed a typical contractant mode of failure, as seen in Figure 7.10. The volume reduced upon shearing, accompanied by barrelling of the sample shape. Nevertheless, after dismantling the test, a faint indication of a failure plane was observed. In Figure 7.3b, it can be seen that the sample does not quite end up on the critical state line extrapolated from the tests performed at lower stresses. It is not clear whether the critical state line is actually curved, or Sample TR6 was not sheared to a sufficiently large strain, or whether a small localisation of strains prevented it reaching the true critical state.

TR7
By comparing the results of Test TR4 with that following the same total stress path performed by Cunningham (2000), as shown in Figure 7.11, a discrepancy in the results was observed. Cunningham’s result demonstrated a more brittle behaviour, indicated by a fall in the deviatoric stress after a peak, accompanied by the development of a failure plane in the sample. At first, the reason for this discrepancy was unclear and therefore Test TR7 was carried out in order to repeat Test TR4.

Test TR7 was however carried out on a 50mm diameter sample and also incorporated the radial strain belt. As explained in Section 3.8.4d, the volumetric strain calculated from the local strain measurements was compared with the volumetric strain measured directly by the volume gauge. This comparison supports the assumption made regarding the correction for the barrellled shape of sample upon shearing, which was then used for the subsequent suction-controlled triaxial tests.

Otherwise, the testing procedure of Test TR7 was the same as that of Test TR4. The results are shown together in Figure 7.11. The stress-strain relationships for Test TR7 are also shown in Figure 7.11/2. It can be seen that both samples followed closely the same effective stress stress path. The difference between the paths for the two tests was due to the fact that shearing of Sample TR4 was started prematurely, before the sample was fully consolidated. Some excess pore water pressure would have still existed at the
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start of shearing stage for Test TR4. The results of Tests TR4 and TR7, therefore, confirmed the repeatability of the tests carried out in the current research.

A possible explanation was then sought after for the discrepancy between the results of the present study and that of Cunningham (2000). Cunningham’s results show that even for isotropically normally-consolidated reconstituted samples tested at zero suction, upon undrained shearing, the deviatoric stresses reduced after reaching a peak value, followed by the development of a failure plane. This behaviour was not observed in the present studies and is also not usually reported in the literature. It was suspected that there might have been more segregation between silt and clay particles in the samples of Cunningham, and perhaps the failure plane observed in his studies occurred along the clay-rich part of the sample. In addition, the shearing rate used by Cunningham was maintained at 0.53%/hr during the tests, which was faster than the shearing rates followed in the present research as described in Section 3.7.3d.

7.2.5 Conclusions for the fully saturated behaviour of the reconstituted samples

The test results of the present study show a typical behaviour of a low plasticity reconstituted clay. The reasons for the differences in the behaviour observed in the present study and the rather unusual behaviour seen by Cunningham are uncertain, but are suspected to be associated with a slight difference in the materials due to segregation between silt and clay particles, as well as some difference in the shearing rate. Nevertheless, the repeatability of results from Tests TC7 and TC4 confirm the validity of the present study.

The Critical State and Normal Compression Lines are plotted in Figures 7.3 and 7.4. The parameters defining them are summarised in Table 7.2. The failure envelope given by Cunningham (2000) and Cunningham et al. (2003) is associated with the peak states and is thus different to the Critical State Line here. The Normal Compression Line from Cunningham (2000) is different to that obtained in the present research and is again believed to be due to the segregation of the soil.
7.2.6 Tests on compacted samples

Two conventional triaxial tests were carried out on compacted samples of Series 7-10 and 7-13. The descriptions of both tests, TC1 and TC2, are shown in Table 7.1. In both tests, the radial and the axial strains were measured locally, and as in Test TR7, the volumetric strains measured by the local instruments were compared with the global volumetric strain by the volume gauge. The method of correcting the volume for the barrelled shape of samples on shearing was also verified using the results of these two tests.

a) TC1

Sample TC1 had similar initial properties to those of Series 7-13. It was consolidated to a \( p' \) of 200kPa before drained shearing. The results are shown in Figure 7.12. During sample saturation, the back pressure was increased gradually to 500kPa while \( p' \) was maintained at 10kPa. It was observed from the local instruments that the specific volume reduced by about 0.01 from its initial state during saturation. This reduction in volume upon saturation is different to the slight swelling-upon-wetting with an increase of specific volume of approximately 0.002 at a vertical stress of about 10kPa seen in an oedometer sample (Figure 5.1). A reason for this difference might be that the way in which the sample was saturated in the triaxial cell was very different to that in the oedometer apparatus. The triaxial test involved a direct application of a high positive water pressure, whereas the oedometer sample was wetted at atmospheric pressure. The triaxial sample might have undergone some collapse during the saturation due to this high positive pressure. In addition, the triaxial sample was under the isotropic condition, whereas the oedometer the \( K_0 \) condition. Another explanation might be that the oedometer sample still had some occluded air-bubbles after wetting had been complete.

From Figure 7.12a, during consolidation, the compression line of the compacted samples lies above the reconstituted Normal Compression Line, before moving below it at a \( p' \) of 40 kPa. This trend is different to that observed from the oedometer tests (Figure 5.1), which showed a convergence between the compression lines of the reconstituted and compacted samples. Upon drained shearing, the sample contracted towards a Critical State Line, which is significantly lower in location than that of the reconstituted soil. The global volume measurement was used in this calculation for consistency with the results of the reconstituted samples. The results on a \( q-p' \) plot in
Figure 7.12b shows that the critical state for both reconstituted samples and Sample TC1 were on the same envelope ($M = 1.32$). Figure 7.13 shows the stress-strain relationship during shearing.

b) TC2

Sample TC2 was from Series 7-10 (Table 7.1). It was consolidated to a $p'$ of 200 kPa, before undrained shearing. On saturation, the specific volume of the sample reduced by 0.02, which is again different to the behaviour observed in the oedometer test (7-10-SL). The results during compression and shearing are also shown in Figure 7.12. The compression behaviour was similar to that of Test TC1. Upon shearing, the effective stress path on a $q$-$p'$ plot shows a different pattern to that of Tests TR4 and TR7, which followed the same total stress path. The stress-strain relationship during shearing is shown in Figure 7.14.

c) Discussion of the behaviour of fully saturated compacted samples

The structure and stress history of the reconstituted samples are different to those of the compacted samples. The results on a $q$-$p'$ plot (Figure 7.12b), however, show that their critical states are at the same stress ratio, $M$. The CSL on a $\nu$-$\ln p'$ plot (Figure 7.12a) appears to have a similar gradient as that for the reconstituted sample. Due to the limitations of the apparatus, the Normal Compression Line of the compacted samples could not be defined beyond the mean effective stress of 200kPa. This was because a relatively high back pressure (>500kPa) was required in order to saturate the compacted samples, and at 200kPa neither tests seemed to have reached a unique NCL.

It can nevertheless be seen that both the NCL and CSL for compacted samples lie below those of the reconstituted samples. In addition, it might be expected that, at a mean effective stress greater than 200kPa, the compression paths for both compacted samples would converge towards the NCL, which lies parallel to the CSL. For $p'$ below 200kPa, the compression path thus lies above the extrapolated NCL and this is believed to be due to their compaction-induced structure. It is worth noting that this interpretation is based on the assumption that both compacted samples from Series 7-10 and 7-13 have the same NCL and CSL.
7.3 Unsaturated triaxial tests

After the tests on fully saturated samples had been completed, the triaxial apparatus was modified for testing the unsaturated compacted samples. The modification of the apparatus was ongoing, while the testing programme proceeded, and thus different versions of the apparatus were used for different tests, as explained in Section 3.8. Table 7.3 gives a description of all the tests performed on unsaturated samples. All samples tested were of Series 7-10. After compaction, the moisture content of each sample was increased by directly spraying water onto it. The sample was then mounted onto the triaxial cell, ready for testing. The modified moisture content and void ratio prior to testing are given for each sample in Table 7.3. The final moisture content of the sample was then measured again at the end of test.

The constant moisture content condition was checked by comparing the initial and final moisture contents. If the difference between the two was significant (more than 0.1%), especially when the test duration was long (e.g. 1 month) a linear variation was assumed in the data interpretation. The change in moisture content during testing occurred as a result of absorption of water through membrane. This adsorption could be particularly considerable especially when the cell was filled with glycerol as a result of osmosis through the membrane.

A variety of stress paths were followed in the suction-controlled triaxial tests. Table 7.3 categorises the tests according to the type of stress path followed. In the subsequent sections, the results will be presented according to the different topics of investigation. The volumetric behaviour is discussed first, in particular within the Loading-Collapse framework. The shearing behaviour is then presented together with its discussion.

7.4 Volumetric behaviour of unsaturated compacted soil A

7.4.1 Isotropic compression

The behaviour of the compacted Soil A during isotropic compression at different suctions was investigated in the tests of Group 1. Three tests were carried out, namely Tests TC16, 18 & 29. Each sample was wetted to a different water content and then compressed to the maximum cell pressure of 800kPa at a constant water content. In order to identify the isotropic Normal Compression Lines at different suctions, the
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results have to be interpreted using the same contour technique as that employed for the oedometer test results described in Section 5.6 and in Appendix 3.

The results of Group 1 are shown in Figure 7.15, together with the compression path of the fully saturated test TC2, in terms of specific volume, suction, degree of saturation, deviatoric strain, \( \varepsilon_q = \frac{2}{3}(\varepsilon_a - \varepsilon_r) \), and net mean stress. The trend observed in this figure is similar to that from the oedometer test results (Figure 5.7). From Figures 7.15a & b, as suction increases, the compression line displaces to the right, indicating a stiffer response. The compression lines in Figure 7.15a are shown together with the yield points estimated using the Cassagrande (1936) method, shown as arrows, as well as estimated contours for compression lines at constant suctions.

Figure 7.15b shows that the reduction in suction during constant water content loading was relatively small in Tests TC16 & 18. In Test TC29, the sample was dried to a higher suction of about 950kPa prior to the constant water content loading. Upon loading, the suction reduced more considerably than was observed in Tests TC16 & 18. The LC yield curve had been drawn through the estimated yield points and the corresponding suctions in Figure 7.15b. Figure 7.15c shows the degree of saturation plotted against mean stresses. In Figure 7.15d, the deviatoric strains during isotropic compression are plotted against the mean stresses. The anisotropy of the compacted samples is evident as indicated by the reduction of the deviatoric strains upon loading. The rates of reduction in the deviatoric strains of Samples TC2 and TC16 nevertheless seem to decrease with the applied stress. This suggests that the anisotropic structure was changing towards an isotropic one as the samples were loaded isotropically.

7.4.2 Anisotropic compression

The tests of Groups 2 and 3 were carried out with the aim of investigating the compression behaviour under anisotropic condition. Table 7.3 describes the testing details of the two groups. Group 2 (Tests TC14, 15 & 30) involved compressing the samples under a \( K_0 \) condition with constant water contents. The \( K_0 \) condition was ensured during the tests using the procedure explained in Section 3.8.4c. In Group 3 (Tests TC19 & 20), the compression was carried out at a constant stress ratio \( (\eta = q/p = 0.5) \) again with constant water contents.
The test results from Group 2 are presented in Figure 7.16. The data for Test TC30 are, however, incomplete. In Test TC30, glycerol permeated into the sample through a leak in the top cap during $K_0$-compression, and it was unsure when the leakage started. Upon dismantling the test, the sample was examined and its top part appeared to be soaked with glycerol, but the lower part, where the suction probe was attached, was unaffected by the glycerol. Figure 7.16b shows the reading from this suction probe during Test TC30. The significant reduction in the suction during compression was most likely caused by an increase in the degree of saturation of the lower part of the sample, since the soil water was pushed downward by the positive glycerol pressure at the top part of the sample. The suction reading was assumed to be approximately representative of the whole of the sample, while the water content and the degree of saturation were considered to be unreliable and not included in Figure 7.16c.

As shown in Figures 7.16a & b, the higher the suction, the more the NCL displaces to the right. Also shown in Figure 7.16a are the constant suction NCLs estimated using the contour technique as described in Appendix 3. The change in suction on loading for Tests TC14 & 15 appears to be only marginal. Figure 7.16d shows the stress paths followed by the samples during $K_0$-compression on a $q$-$p$ plot. The unload-reload loops in the stress path of Sample TC14, for net mean stresses between 450 and 600kPa, occurred accidentally, as a result of an unstable control system. Nevertheless, the $K_0$-compression path of the sample with a higher suction lies above that of a lower suction. The gradient of the path, $q/p$, reduced as yielding took place. After yielding, the compression lines for samples at higher suctions still lie above those at a lower suction.

This decrease of the stress ratio, $q/p$, upon yielding is similar to the behaviour of bonded soils, such as an artificially cemented carbonate sand (Coop & Atkinson, 1993) and a calcarenite (Lagioia & Nova, 1995). This trend is also in a good agreement with observations from a discrete element modelling study by Liu et al. (2003). This similarity confirms that the suction, together with the soil fabric, contributes to a form of bonding force at the contact points between soil particles.

The results of tests from Group 3 (TC19 & 20) are presented in Figure 7.17. Figure 7.17a shows the stress path followed by the samples on a $q$-$p$ plot. After each sample
was compressed at a constant stress ratio, \( \eta = q/p = 0.5 \), to the maximum net mean stress, \( p \), of 958 kPa, it was unloaded until the cell pressure was equal to 200kPa and then \( q \) was subsequently reduced to zero, before the shearing stage started. Figures 7.17b, c, d & e show the specific volume, suction, degree of saturation and deviatoric strains plotted against the net mean stress. Again the sample with a higher suction (TC20) is stiffer and its compression path lies to the right of the sample with a lower suction. The post-yield compression paths of Samples TC19 & 20 coincide with the NCLs for constant suctions of 150 and 250 kPa respectively. From Figure 7.17c, it can be noticed that the suction change during loading was only marginal, as was observed in the previous loading tests. The change in suction of Sample TC 20 for mean stresses below 50kPa was not representative, since the reading is not consistent with the other tests. It was speculated that this was because for the cell pressure less than 50kPa the suction probe was not in a good contact with the sample. The dashed line had been drawn to show the estimated initial suction of the sample.

From the plot of deviatoric strains against mean net stresses in Figure 7.17e, it can be observed that before the gross yielding took place, the deviatoric strain decreased upon loading. After the sample yielded, however, the deviatoric strain started to increase. This increase in the deviatoric strain reflects a significant rearrangement of the soil particles upon the gross yield.

### 7.4.3 Wetting tests: isotropic

The wetting tests were performed in the suction-controlled triaxial apparatus on the samples of Group 4. The details of the testing procedure have been presented in Section 3.8.4e. The sample was wetted incrementally at a constant stress by adding known amounts of water directly to it. After each addition of water, a certain period of time, normally around one to two days, was allowed until the suctions near the top and the base became approximately equal, and until the change in volumetric strain was less than 0.05% /day.

Four wetting tests were performed, namely TC25, 26, 27 & 28. Table 7.3 describes the details of each test. Tests TC25 and 26 involved wetting the sample under isotropic stresses condition at cell pressures of 200 and 100kPa respectively. In Test TC25, the
sample was wetted from its base. As a matter of fact, the first stage of wetting in this test happened accidentally. The drainage tubes from the base of the sample were submerged in a container of water overnight. When the temperature differed between inside the sample and the water container, the gradient in air pressure caused the water from the container to move in to the soil and cause a reduction in suction. The change in suction with time during wetting for Test TC25 is shown in Figure 7.18. After the first accidental wetting, the sample was wetted by passing a mixture of air/water from the base, through the sample, and out from the top, using a peristaltic pump. The amount of water added to the sample was measured directly by weighing the tube filled with water before and after wetting.

As described in Section 3.8.4, this wetting system was, however, not very effective since it was found very difficult to pass water and air from the base to the top of sample without changing the air pressure in the sample. It can be seen in Figure 7.18 that during the third wetting stage, the top pore water pressure temporarily reduced by 40kPa. This occurred as a result of air being sucked out of the sample top by the peristaltic pump, which reduced the air pressure around the top of the sample. According to the axis-translation principle, this reduction in air pressure, $u_a$, brought about an equal amount of reduction in the pore water pressure, $u_w$, as shown in Figure 7.17. Once the circulation of water/air was stopped, the air pressure returned to zero and the pore water pressure also returned to its normal value. Even though this reduction in air pressure only caused a temporary change in pore water pressure and not in suction, the net stress, $(\sigma_n = \sigma - u_a)$, was increased and this could have given rise to extra compression of the sample. In addition, when the sample became nearly saturated, its air permeability reduced dramatically and this made it very difficult to pass water and air through the sample to reduce the suction to zero. Sample TC25 could therefore only be wetted to a suction of around 70kPa at a degree of saturation of 73%.

The final version of the wetting system was consequently developed and used for Test TC26. The details of the system have been described in Section 3.8.4. It only differed from the system for Test TC25 in that the circulation of water/air occurred along the top of the sample (see Figure 3.25). The air within the circulation line thus only passed along the top of the sample, and not through the sample. The water was absorbed into
the sample during this circulation. The wetting stage was done incrementally and each step was allowed to come into equilibrium in both suction and strains. The amounts of added water were measured by directly weighing the water and the tube before and after each circulation. The system proved to be effective and enabled wetting to be carried out to nearly zero suction.

In Test TC26, the sample was wetted at a constant cell pressure of 100kPa. The plot of suction against time during wetting of the sample is shown in Figure 7.19. The suction probe near the base of the sample, further from the wetting surface, temporarily stopped functioning from hrs 67 until hrs 637. Only the top probe, located near the wetting surface, was in use during these hours. A period of about 48 hours was allowed between each wetting stage. From the experience with Test TC25, it was believed that this duration was sufficient for the equilibrium of suction and strains to be attained. From Figure 7.19, a drift in the suction measurement can be seen from hrs 265 to 288. This drift occurred as a result of a temperature control failure in the laboratory. After the control had been restored, the suction reading returned to the expected value.

The results of Tests TC25 and 26 are plotted in Figure 7.20 in terms of suction, net mean stress, specific volume, degree of saturation, and deviatoric strain. Figure 7.20a shows the stress paths followed by the two samples, on an $s-p$ plot. It can be seen that towards the end of wetting in Test TC26, the net mean stress reduced slightly from 100 to 95kPa. This was due to a failure of the pressure controller. The reduction nevertheless was considered to be only marginal and did not seriously affect the test. Figures 7.20b, c & d show the change in specific volume, degree of saturation and deviatoric strain plotted against the net mean stress, respectively. The reduction of the deviatoric strains during loading and wetting (Figure 7.20d) again indicated the anisotropy of the soil structure.

Figure 7.20e shows the plot between specific volume and suction. From this plot, the yield points were identified, as indicated by the arrows. Figure 7.20f shows the change in degree of saturation with suction during wetting. The curve for Sample TC25 lies slightly above that of Sample TC26. The influence of void ratio on the wetting curve therefore seems to be only marginal from this plot. More discussion and comparison with the SWRCs, as identified in Chapter 4 using the filter paper technique, will be
presented in Chapter 8. Figure 7.20g illustrates the change in deviatoric strain with suction upon wetting. For Test TC25, the suction at the yield point that is estimated from the deviatoric strain (Figure 7.20g) would be lower than the suction at the yield point estimated from the specific volume which is shown as the arrow in Figure 7.20e.

The onsets of yielding upon wetting are also compared with the LC curve from the loading tests of Group 1, as shown in Figure 7.20a. The yield points from the wetting tests were determined using the specific volume criteria in order to be consistent with the loading tests. A good agreement is observed between the yield points from the two types of tests.

### 7.4.4 Wetting tests- anisotropic stress condition

Two wetting tests, namely TC27 and TC28, were performed on samples in anisotropic stress states. Test TC27 involved wetting the sample at a constant stress ratio, $\eta = q/p = 0.5$. As shown in Figure 7.21a, Sample TC27 was compressed at an approximately constant water content from A to B. From B to C the sample was wetted incrementally at constant stresses. Figure 7.22 shows the plot of suction against time during the test. Sample TC28 was compressed under K₀ conditions from A to E, as shown in Figure 7.21b. The K₀ condition was maintained using the feedback control from the local radial strain measurement, as was used in Tests TC14, 15 & 30. Figure 7.23 shows the plot of suction against time during Test TC28. The dotted lines shown indicate the estimated results when the actual data were lost due to a data-scanning problem.

It might be argued from Figures 7.22 & 7.23 that the suction readings had not stabilised, especially before and during the first few stages of wetting. This apparent drift in suction (around 20kPa/day) could have been the result of two phenomena, excess pore pressure dissipation and the gradual drying of the sample. Experience from the other constant water content loading tests suggested that the latter was the more likely cause of the drift. Therefore, it would have been unnecessary to allow for a longer time between each stage, since the drift would still continue slowly due to the gradual drying of the sample. Nevertheless, the suction measurements during yielding, which are more critical in identifying the yield surface, are relatively stable (less than 10kPa/day) and considered to be more reliable.
In Test TC28, the membrane had been soaked in distilled water for two days before the test started. This was originally aimed at preventing the membrane from absorbing water from the sample. However it appeared to bring about an adverse effect instead. After soaking, the membrane became softer and expanded. When the sample was at a constant cell pressure of 20kPa (from point O to A in Figure 7.22), the rate of increase in the radial strain with time (not shown in the figures) was much more significant than previously observed in other tests. This was not believed to be the real behaviour of the sample, but the shrinkage of the membrane itself. After the sample had been consolidated under Ko conditions to a net mean stress, $p$, of 175kPa (point A’ in Figure 7.23), the suction appeared to increase significantly with time under a constant stress condition, as seen in Figure 7.23 from hrs 240 to 310 (from A’ to E). A possible explanation is that once soaked, the membrane expanded and its permeability increased. This increase in permeability might have made it easier for the glycerine cell fluid to draw water from the soil through osmosis. As in the other tests, the water content during Test TC28 was corrected by using linear interpolation to account for the difference in the measured and estimated final water contents.

Figure 7.24 shows the results of Tests TC27 & 28, plotted in terms of suction, net mean stress, specific volume, degree of saturation and deviatoric strain. The stress paths followed by the samples in the two tests are given in Figure 7.24a on a suction-net mean stress plot. From Figure 7.24b, it can be seen that Sample TC27 collapsed upon wetting from B to C. As for Sample TC28, after Ko consolidation (A-E), it was wetted to about 160kPa suction (E-F) and some collapse occurred. Afterwards, the sample was loaded at a constant $q$ (F-G) to the net mean stress of 891kPa (Figure 7.21b) and wetted again to the lowest attainable suction (G-H), causing the final collapse-on-wetting (Figure 7.24b). The yield points have been identified from the plots of specific volume and suction, indicated by the arrows in Figures 7.24c, d & e.

The changes in degree of saturation with net mean stress and suction during testing are shown in Figures 7.24f & g. The differences between the wetting curves of the two samples (Figure 7.24g) are caused by their differing structures, void ratios, and the stress paths followed. The plots between deviatoric strain, net mean stress and suction are shown in Figures 7.24h, i, j &k. Figures 7.24h & i indicate that the deviatoric strain
Chapter 7 - Triaxial tests

of Sample TC27 started to increase as the samples underwent gross yielding upon wetting. The trough in Figure 7.24i agrees well with the yield point estimated from Figure 7.24c.

A similar trend is observed for Sample TC28 in Figures 7.24 j & k. During the wetting path E-F, the deviatoric strain increased as the sample collapsed. Nevertheless, after the sample was loaded from F to G, the deviatoric strain decreased slightly during the wetting path G-H. This observation suggested a significant change in the anisotropy of the soil caused by the loading path F-G.

7.4.5 Final comments on the volumetric behaviour

The results from the loading and the wetting tests under isotropic stress conditions suggested that the onsets of collapse-upon-wetting approximately coincide with the Loading-Collapse curve identified by the loading tests. This observation was in line with that made from the oedometer test results. In Chapter 8, the anisotropic wetting and loading behaviour will be interpreted in a more quantitative manner and compared with the results of the oedometer tests in an elasto-plastic framework.

7.5 Shearing behaviour of unsaturated compacted Soil A

Four groups of shearing tests have been performed, namely Groups 5, 6, 7 and a special group. The descriptions of the tests from Groups 5, 6 & 7 are given in Table 7.3. The special group consists of Test TC29, 19, 20 & 25, which were the continuation of the tests from Groups 1, 3 & 4. Each sample of the special group had experienced yielding in different manners before being unloaded to a cell pressure of 200kPa, followed by shearing at a constant water content. In the following sections the test results of each group will be presented.

7.5.1 Group 5: shearing at a cell pressure of 50 kPa

Group 5 consists of six tests: TC7, 8, 9, 10, 21 & 23. The aim of this group was to investigate the shearing behaviour at a cell pressure of 50kPa of samples at different suctions. After compaction, each sample was wetted by directly spraying water onto it, before mounted in the triaxial apparatus, in order to modify its initial suction to a desired value, ranging from around 100 to 500kPa. In Tests TC21 & 23, the samples
were wetted by about 0.5% w/c before mounting in the triaxial cell. This initial wetting was carried out despite the fact that both samples would be dried again to a higher suction in the triaxial cell. As explained in Section 3.8.4, this initial wetting was necessary since it was very easy for the suction probes to experience tension breakdown (cavitation) during the test set up, and the initial wetting helped prevent this.

After mounting in the triaxial cell, Samples TC21 & 23 were compressed to a cell pressure of 50kPa and then dried to suctions of about 850 & 1000kPa respectively, at a constant cell pressure, using the air-circulation system (Figure 3.24). Afterwards, Sample TC21 was sheared at a constant water content, whereas the shearing of Sample TC23 was carried out at a constant suction of about 1000kPa, using the air-circulation system.

The results of tests from Group 5 tests during compression are shown in Figure 7.25. The contours of Normal Compression Lines as estimated from the Group 1 tests are also included in Figure 7.25a. Figure 7.25b shows the stress paths followed by the samples on an $s$-$p$ plot, together with the Loading-Collapse curve identified from Groups 1 & 4. It can be observed that after isotropic compression to a net stress of 50kPa, each sample was still well inside the yield surface. After compression, creep was allowed until the rate of change in the volumetric strain fell below 0.05%/day before shearing started.

The results during shearing are shown in Figure 7.26 in terms of deviatoric stress, pore water pressure, volumetric strain, equivalent shear modulus (tangential) and axial strain. The axial strains plotted are the average of the two local measurements. Regarding the volumetric strains, the correction method was applied, where necessary, for the barrelled shape of the sample, as described in Section 3.8.6g. The volumetric strains calculated using the right cylinder assumption are also included in the figures for comparison.

Table 7.4 summarises the failure modes observed in all samples on shearing. It can be seen that as the suction increases, the failure mode changes progressively from being barrelling without a shear plane, to barrelling with a shear plane, and finally non-barrelling with a shear plane. In Tests TC7, 8 & 9, the correction for barrelling was applied to the calculation of the radial and volumetric strains, after the sample shape
started to be barrelled, as shown in Figure 7.26c. The deviatoric stresses were also calculated using the radial strains corrected for the sample’s barrelled shape.

The sequence of the shearing rates used in these tests has been discussed in Section 3.8.4c. All shearing stages were carried out with the constant rate of strain pump. To start with, the rate of axial strain was 0.05%/hr, but this was doubled every 8 hours, towards a rate of 0.4%/hr. The purpose was to obtain a better resolution of the stiffness measurements in the small strain region, without having to perform excessively long tests.

From Figure 7.26a & b, it is evident that as the initial suction increases, the sample becomes stiffer, and reaches a higher peak deviatoric stress at a lower strain. For Tests TC21 & 23, the deviatoric stress reached a peak value, prior to a rapid drop on the formation of a failure plane. These observations correspond to the failure mode as shown in Table 7.4. Regarding the change in suction during constant water content shearing, Figure 7.26b suggests that the reduction in suction on loading is greater as the suction increases and as the degree of saturation decreases. This trend is similar to that observed in the constant water content loading tests (Groups 1 & 3).

Of particular interest is the result of Test TC23 in which the suction was maintained constant during shearing. The drying system was controlled based on the suction measurement near the base, closer to the drying surface. The top suction reading differed from the base reading within about 10%. When the rate of shearing was increased to 0.4%/hour (corresponding to when the deviatoric stress reached the peak), it appeared to be more difficult to maintain the suction at a constant value within the same tolerance (Figure 7.26b). This is probably since the shearing rate of 0.4% was too fast for the drying system to maintain the constant suction. After 4% axial strain, the sample had been sheared into two blocks and their relative movement caused the membrane to stretch, resulting in the top suction probe losing contact with the soil. This loss in contact brought about a steady increase of the top suction as shown in Figure 7.26b.

The changes in volumetric strain during shearing are shown in Figure 7.26c. All samples, regardless of suction, initially reduced in volume upon shearing, up to an axial
strain of around 0.5 to 1.5%. The samples with lower suctions tended to compress more and did so up to a higher strain. Afterwards, the samples started to dilate. The general trend is that the higher the suction, the greater was the dilation. The arrows indicate the points at which the barrelling correction started to be applied in Tests TC7, 8 & 9. With the assumption of the barrellled shape, the dilation becomes less than that which was calculated based on the right cylinder assumption. The representative volumetric strains of the samples are likely to have values between those based on the two assumptions. From Figure 7.26c, none of the samples showed clearly that a constant volume condition was achieved. The volumetric strain in Test TC7 was probably reaching a constant value towards the end of the test. The degrees of saturation are plotted against the axial strain in Figure 7.26d. For all tests, the degree of saturation appeared to reduce slightly upon completion of shearing due to the dilation of the sample.

The influence of suction on the stiffness degradation curve is shown in the plots of the equivalent shear modulus in Figure 7.26e. As suction increases, the modulus increases. Apart from Tests TC7 & TC9, the modulus started to reduce significantly after an axial strain of about 0.01 to 0.02 %. The failure envelopes corresponding to the peak and ultimate deviatoric stress are shown on a $q$-$s$ plot in Figure 7.27. The points corresponding to zero suction were estimated based on the fully saturated failure envelope described in Section 7.2.6. The envelope is clearly non-linear and within the suction range investigated (up to 1000 kPa) it may be characterised as approximately bi-linear. The evidence of the critical state will be discussed later in Chapter 8.

In summary, the observations made from the results of Group 5 demonstrate most of the typical behaviour of partly saturated soils, namely the dependency of strength and stiffness on suction as well as the non-linearity of the failure envelope. In addition, the magnitude of change in suction during the constant water content shearing has also been shown to be dependent upon the initial suction value.

7.5.2 Group 6: shearing at a cell pressure of 200 kPa

Group 6 consists of three tests, namely TC13, 22 & 24, which involved shearing at a cell pressure of 200kPa at different suctions. The test procedure followed was similar to that of the Group 5 tests. After wetting outside the triaxial cell to modify its suction, each sample was isotropically consolidated at a constant water content to a cell pressure
of 200kPa. Sample TC22 was further dried using the drying system to a suction of around 900kPa. Afterwards, all samples were sheared at a constant content water to failure.

The results of the Group 6 tests during the compression stage are shown in Figure 7.28 in the same manner as for Group 5. It can be seen that both Samples TC13 and TC24 were at the onset of yielding, just before shearing started. Sample TC22 was still within the yield surface before shearing. The results during shearing of Group 6 are shown in Figure 7.29 in terms of deviatoric stress, pore water pressure, volumetric strain, equivalent shear modulus and axial strain.

As indicated in Table 7.4, all samples of Group 6 failed in a barrelling mode without any formation of failure planes. The plot between deviatoric stress and axial strain in Figure 7.29a shows a strain-hardening mode of failure for all three tests. Again, the sample with a higher suction is stiffer and reached a higher strength. In addition, the axial strain at which the deviatoric stress reached its ultimate value appeared to be inversely proportional to suction. Figure 7.29b shows the change in pore water pressure during shearing. Sample TC22, having a higher initial suction, underwent a decrease in suction during shearing. On the other hand, the suctions of Samples TC13 & 22 initially only slightly decreased on shearing and then increased gradually as shearing proceeded.

The volume change during shearing is shown in Figure 7.29c. It can be seen that none of the samples appeared to reach a constant volume state within the working range of the radial strain transducer, although from Figures 7.29a & b, the deviatoric stress and the pore water pressure appear to be reaching the constant values at larger strain. It is thus likely that at axial strains above 20% all samples would probably have reached a critical state. The changes in degree of saturation during shearing are shown in Figure 7.29d. Again the changes are relatively small in all tests. Figure 7.29e shows the stiffness degradation curves for the three samples. The same trend as for Group 5 can be seen and drier samples have higher shear moduli. The ultimate failure envelopes both for Groups 5 and 6 are shown in Figure 7.27. Group 6 results are shown as squares. Both envelopes again appear to be non-linear.
By comparison of Groups 5 and 6, the failure mode of the sample does not appear to depend only on the suction, but also on its relative distance from the yield surface in $p-q-s$ stress space. For example, Sample TC24, despite having a similar suction to Sample TC21, failed in a barrelling mode, as opposed to the slip-plane formation for TC21. This was due to the higher confining pressure used for Test TC24. The values of $q$, $p$ and suction at the ultimate state, deduced from Figures 7.29a & b, would represent the critical state. Chapter 8 will discuss in more detail about the critical states of these samples.

### 7.5.3 Group 7: air-dried shearing tests

This group of tests was carried out with the aim of identifying the boundary of the failure envelope on a $q-p$ plot at the maximum suction. As described in Table 7.3, three tests were performed, namely TC4, 12 & 3, in which the air-dried samples were sheared at a constant water content and cell pressures of 50, 100 & 200 kPa respectively. Each sample was air-dried by being exposed to air in the laboratory until the water content remained unchanged. It was then compressed to a desired cell pressure and creep was allowed until the rate of change of volumetric strain fell below 0.05%/day (normally for a period of around 1-2 days) before shearing started. Sample TC12 however underwent a longer period of creep (around 5 days) after compression, due to the author’s absence delaying the commencement of the shearing stage. Table 7.3 shows the final water contents measured at the end of each test. The water contents of Samples TC3 & 4 increased during the tests by 0.04 and 0.03% respectively, whereas the increase for Sample TC12 was 0.21%. Since water was used as the cell fluid in these tests, it was believed that the increase in water content was due to the adsorption of water through the rubber membrane.

The results of Tests TC4, 12 & 3 during compression are shown in Figure 7.30. The changes in volume during compression are only marginal due to the samples’ high suctions. The results during shearing are shown in Figure 7.31 in terms of deviatoric stress, equivalent shear modulus, volumetric, and axial strains. The failure mode of all samples was non-barrelling with a distinct shear plane. The stress-strain plots for the three samples show a very brittle behaviour as would be expected for samples with very high suctions. The peak deviatoric stresses are then plotted against the mean net stress in Figure 7.32, together with the failure envelope from the fully saturated tests. It might
be assumed that the peak failure envelope for air-dried samples is of the same gradient as the fully saturated one. Also included in Figure 7.32 are the stress states at the end of tests. The gradient of the failure envelope at the end of test, equal to 1.522, is slightly higher than that of the peak state. The suctions of the three samples were not directly measured but can be assumed from the SWRCs shown in Chapter 4 to be around 30 MPa and can also be assumed to be unaffected by shearing. The stiffness degradation curves shown in Figure 7.30c suggest that there is no significant influence of the confining pressure (within the range 50 to 200kPa) on the shear moduli of air-dried samples.

7.5.4 The special group
Sections 7.4.1 to 7.4.3 describe the stress paths followed prior to shearing for the tests of this group, TC19, 20, 25 & 29. All samples were sheared at a cell pressure of 200kPa under a constant water content condition. Samples TC19, 20 & 29 were well inside the Loading-Collapse surface before shearing. Only Sample TC25 had been collapsed on wetting and therefore its state was on the LC surface before shearing. Figure 7.33 shows the results during shearing, plotted in terms of deviatoric stress, pore water pressure, volumetric strain, degree of saturation, equivalent shear modulus and axial strain. The failure mode for each sample is also presented in Table 7.4.

The ductility of Sample TC25 during shearing can be seen in Figure 7.33, which corresponds to the barrelling failure mode observed. The strain hardening and contractant behaviour can be observed in the plots of deviatoric stress and volumetric strain against axial strain in Figures 7.33a &c. However, the critical state of the TC25 sample had not been reached in the test due to the limitation of the travel distance of the axial ram. Nevertheless, the deviatoric stress at the critical state may be estimated to be between 500 and 550 kPa from Figure 7.33a. This estimated stress state at the critical state falls close to the same trend as those from the tests of Group 6 as shown in Figure 7.27. The tests of Group 6 involved shearing at the same cell pressure as this special group. Sample TC29 had been slightly overconsolidated before shearing. The sample had a barrelled shape with a slip plane upon failure. The stress state at the peak deviatoric stress is also plotted in Figure 7.27 and is found to lie close to the same trend as the failure envelope of the Group 6 tests.
In Tests TC19 & 20, however, the samples were heavily over-consolidated. From Figure 7.33, it might be assumed that both samples had reached the critical state upon shearing, as suggested by the constant deviatoric stress and pore water pressure and the trend of the volumetric strain towards a constant value. The stress states at failure of the two tests, when plotted in Figure 7.27, however, lie significantly above the failure envelope of Group 6 and Tests 25 & 29, due to the overconsolidation. Similarly, as shown in Figure 7.33e, the small strain stiffnesses for Samples TC19 & 20 are much greater than those of the other two samples.

7.5.5 Final comments on the unsaturated shearing behaviour

The shearing behaviour of compacted Soil A in unsaturated states has been shown to be typical of the behaviours of many unsaturated soils reported in the literature. These include the non-linearity of the relationship between shear strength and suction, and the increase in stiffness of the soils with increasing suction. More discussions will be made in Chapter 8 regarding the critical state and its characterisation as well as the stress-dilatancy relationships during shearing.
### Table 7.1 Fully saturated triaxial tests on reconstituted and compacted Soil A (U = undrained, D = drained)

<table>
<thead>
<tr>
<th>Test</th>
<th>Initial properties</th>
<th>Compression</th>
<th>p’ before shearing (kPa)</th>
<th>e before shearing</th>
<th>Shearing</th>
<th>Sample diameter, mm</th>
<th>Note</th>
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<td>w/c, %</td>
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<td>Sr, %</td>
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<td>TR1</td>
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<td>0.526</td>
<td>97.5</td>
<td>Isotropic</td>
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<td>0.472</td>
<td>U</td>
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<td>19.9</td>
<td>0.549</td>
<td>99.2</td>
<td>Isotropic</td>
<td>100</td>
<td>0.526</td>
<td>D</td>
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<td>0.553</td>
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<td>U</td>
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Table 7.2 Parameters for the fully saturated reconstituted Soil A from the triaxial tests

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<td>$M$</td>
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### Table 7.3 Suction-controlled triaxial tests on compacted Soil A

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
<th>As-compacted</th>
<th>Prior to testing</th>
<th>Pre-shearing stages</th>
<th>Shearing</th>
<th>Final w/c, %</th>
<th>Test Duration (days)</th>
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<td>Sr, %</td>
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<td>Group</td>
<td>Test</td>
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<td>Prior to testing</td>
<td>Pre-shearing stages</td>
<td>Shearing</td>
<td>Final w/c, %</td>
<td>Test Duration (days)</td>
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Table 7.3 Suction-controlled triaxial tests on compacted Soil A (continued)
### Table 7.3 Suction-controlled triaxial tests on compacted Soil A (continued)

<table>
<thead>
<tr>
<th>Group</th>
<th>Test</th>
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<th>Prior to testing</th>
<th>Pre-shearing stages</th>
<th>Shearing</th>
<th>Final w/c, %</th>
<th>Test Duration (days)</th>
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<td>Constant water content/ 50kPa σ₃</td>
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<td>10.0 0.720 36.7</td>
<td>10.2 0.720 37.3</td>
<td>Constant w/c Iso-comp to 50kPa</td>
<td>Constant water content/ 50kPa σ₃</td>
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<tr>
<td></td>
<td>TC21</td>
<td>10.0 0.711 37.0</td>
<td>10.6 0.711 39.2</td>
<td>Constant w/c Iso-comp to 50kPa; Drying to 850kPa suction</td>
<td>Constant water content/ 50kPa σ₃</td>
<td>9.3</td>
<td>8</td>
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<tr>
<td></td>
<td>TC22</td>
<td>10.4 0.711 38.4</td>
<td>10.8 0.705 40.6</td>
<td>Constant w/c Iso-comp to 50kPa; Drying to 1000kPa suction</td>
<td>Constant suction (drying)/ 50kPa σ₃</td>
<td>9.2</td>
<td>10</td>
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<td>TC13</td>
<td>9.9 0.693 37.7</td>
<td>12.9 0.702 48.4</td>
<td>Constant w/c Iso-comp to 200kPa</td>
<td>Constant water content/ 200kPa σ₃</td>
<td>12.6</td>
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<td>TC22</td>
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<td>10.8 0.702 40.7</td>
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<td></td>
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<td>11.5 0.724 42.03</td>
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<td>Constant water content/ 200kPa σ₃</td>
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### Table 7.3 Suction-controlled triaxial tests on compacted Soil A (continued)

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<th>Group</th>
<th>Test</th>
<th>As-compacted</th>
<th>Prior to testing</th>
<th>Pre-shearing stages</th>
<th>Shearing</th>
<th>Final w/c, %</th>
<th>Test Duration (days)</th>
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<td>Sr, %</td>
<td>w/c, %</td>
<td>e</td>
<td>Sr, %</td>
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<td>0.699</td>
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<td>TC4</td>
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<td>39.3</td>
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<td>0.676</td>
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CHAPTER 8

DATA INTERPRETATION

8.1 Introduction

In this chapter the behaviour of compacted Soil A is interpreted in a more quantitative manner, comparing and contrasting that observed in the unconfined wetting/drying tests, in the suction-monitored oedometer, and in the suction-controlled triaxial tests. The two main aspects of the discussion are the volumetric and the shear behaviours. Two types of elasto-plastic models are considered in these discussions: those using the two conventional stress variables and those using two modified stress variables. The shear behaviour is investigated with a particular emphasis on the critical state framework. In addition, the soil-water retention behaviour, as observed in all experiments, is interpreted. The dependency of the retention curves on the void ratio and different components of strain is also discussed. Finally, a summary is given with remarks on the advantages and shortcomings of the two main types of elasto-plastic model.

8.2 Volumetric behaviour in oedometer tests: conventional stress variables

In this section, the Loading-Collapse behaviour as observed in the suction-monitored oedometer tests will be discussed in relation to an elasto-plastic model that uses the two conventional stress variables: suction, $s$, and net stress, $p$. Section 8.3 will describe the same behaviour within the framework using the modified stress approach.

8.2.1 Mathematical expression of the Loading-Collapse yield surface

In Chapter 5, the Loading-Collapse surface in an $s-v$-$\sigma_v$ space was identified for compacted Soil A from Series 7-10, 5-10 and 7-13. As shown in Figures 5.32, 5.33 & 5.34, the surfaces are presented as contours of Normal Compression Lines at different constant suctions. Various mathematical expressions have been proposed in the literature in order to describe these surfaces (e.g. Alonso et al., 1990, Wheeler & Sivakumar, 1995), as shown schematically in Figure 2.23. The trends observed in Figures 5.32-5.34 suggest that models predicting the convergence of the compression
lines at high stresses would be more accurate. Such models are, for example, those by Josa et al. (1992), and Wheeler & Sivakumar (1995).

Based on its simplicity, the Wheeler & Sivakumar type model was considered more suitable to fit the experimental data from the oedometer tests. Figure 8.1 shows the compression path during compaction of a Series 7-10 sample on a $\nu$-log $\sigma_v$ plot together with the contours of Normal Compression Lines from the oedometer tests (Figure 5.32). It may be assumed that the compaction path represents approximately a ‘virgin’ compression line at a constant suction for the compacted samples. The slope of this ‘virgin’ compression line is steeper than that of the NCL at zero suction and thus the Wheeler & Sivakumar type model represents this behaviour more accurately than the Josa et al. type model.

In order to take into account the non-linearity of the compression lines on a $\nu$-log $\sigma_v$ plot, a new mathematical formulation of the surface is suggested in the current studies and will be presented in the following. For convenience, the notation, $p$, will be used to represent the net vertical stress from the oedometer tests.

At zero suction the Normal Compression Line can be represented using the Power law relationship (e.g., Butterfield, 1979, Sridharan et al., 1991);

\[
\frac{\nu_s}{\nu_1} = p^{-b}
\]

where $\nu_s$ is the specific volume at zero suction and $\nu_1$ and $b$ are fitting parameters. For compacted samples with suctions greater than zero, the specific volume, $\nu$, on the Normal Compression Lines can be expressed by the following equation:

\[
\nu = \nu_s + \nu_c
\]

where $\nu_s$ is the specific volume at zero suction with the same net stress (Equation 8.1). The potential collapse, $\nu_c$, is a function of net stress, suction and the fabric of the soil.
Equation 8.2 has the form similar to the models proposed by Liu & Carter (2000) and Georgiadis (2003). However, the difference lies in the formulation of the potential collapse, $\nu_c$, which is proposed as follows:

\[
\frac{\nu_c - \nu_{c\infty}}{\nu_{c1} - \nu_{c\infty}} = p^{-\beta}
\]

\hspace{1cm} (8.3)

where,

- $\nu_{c\infty}$ is the potential collapse at $p \to \infty$
- $\nu_{c1}$ is the potential collapse at $p = 1$ kPa
- $\beta$ is the parameter which indicates the stiffness and destructuration process

Figure 8.2 shows a schematic plot of the variations of the potential collapse with the three parameters. This formulation is similar to that proposed by Den Haan (1992) for describing the NCLs of fully saturated soils. The parameters $\nu_{c\infty}$, $\nu_{c1}$, and $\beta$ are soil characteristics which are functions of suction and fabric. Their variations with suction are specified empirically, based on the experimental results. It is readily seen from Equation 8.2 that $\nu_c$ is zero at zero suction. It follows that $\nu_{c\infty} = \nu_{c1} = 0$ when suction is zero. In addition, since the compression lines appear to converge at large stresses, $\nu_{c\infty}$ is specified as zero for all suctions. In the case of swelling soils, $\nu_{c\infty}$ might be negative in order that swelling is predicted for wetting at large stresses.

In order to obtain the parameters $\nu_{c1}$ and $\beta$, each contour line shown in Figures 5.32, 5.33 was fitted using a transformed regression model in an Excel spreadsheet. The values of $\nu_{c1}$ and $\beta$ calculated were then plotted against suction and their values were adjusted slightly by trial-and-error until reasonably smooth variations with suction were obtained as shown in Figure 8.3. Only the data from Series 7-10 and 7-13 (Figure 5.32 & 5.33) were analysed. The data from Series 5-10 (Figure 5.34) were not analysed in this manner since the range of vertical stresses for their NCLs was more limited. Also shown in Figure 8.3 are the tentative multi-linear fitting lines for the variations. Other fitting functions such as hyperbolas with incline asymptotes could also be used, but for the purpose of checking the performance of Equation 8.3, these multi-linear lines were considered adequate. The value of 1.0 in the expression $\ln (s+1)$ was inserted in order
to avoid infinity at zero suction. Equations 8.1, 8.2 & 8.3 and the fitting lines were then used to predict the specific volume after yielding for the oedometer tests. The comparisons between the experimental results and those from the fitting expressions are shown in Figure 8.4 for Series 7-10. The proposed expressions give a satisfactory fitting to both results from the wetting and loading tests. Certainly, the good accuracy of fit is a result of the circular nature of the interpretation process, but it also shows the good performance of Equations 8.1, 8.2 & 8.3 for reproducing the Normal Compression (or Loading-Collapse) Surface. It also demonstrates that both loading and wetting paths after yielding can be represented by the same surface.

As in other elasto-plastic models (e.g., Alonso et al., 1990, Wheeler & Sivakumar, 1995) ‘elastic’ behaviour was assumed for states inside the Loading-Collapse surface. In the present studies, the following power laws were assumed to govern the elastic behaviour:

\[ d\nu = -\kappa \frac{dp}{p} \]  
(8.4)

for unloading-reloading, and

\[ d\nu = -\kappa_s \frac{ds}{s + 1} \]  
(8.5)

for wetting-drying.

The changes of specific volume during the oedometer tests of Series 7-10 samples were then reproduced using the aforementioned formulations (Equations 8.1 to 8.5). The changes of suction were taken directly from the measurements made in the tests. The values of \( \nu_1, b, \kappa \) and \( \kappa_s \) used are summarised in Table 8.1. Figures 8.5 and 8.6 show typical reproductions of test results from the formulations for a loading and a wetting test respectively.

8.2.2 Influence of initial compacted properties

The input parameters for Series 7-10 were then used to predict the post-yield results of Series 5-10. Figure 8.7 shows comparisons between the prediction and the experimental results in terms of specific volume. Slightly more scatter can be observed in Figure 8.7 than there is in Figure 8.4, because the expressions are now being used to extrapolate...
from the test results of Series 7-10 to that of Series 5-10. It is nevertheless considered that the Loading-Collapse surface for Series 5-10 can be approximately represented using the same parameters as for Series 7-10.

As already discussed in Section 5.6, the contours of NCLs for Series 7-13 (Figure 5.33) are significantly different to those of Series 7-10 (Figure 5.32). This is also reflected as the different input parameters for Series 7-10 and 7-13, shown in Figure 8.3. Figure 8.8 shows the comparison between the fitting and the experimental results. The quality of fit of the expression is reasonably good, similar to that observed for Series 7-10. The difference in the position of LC surface in $s$-$\nu$-$\sigma_v$ space for these two series can be attributed to the difference in the initial as-compacted water content, and thus in the fabric of the material. Whether this difference can be taken into account using the modified stress approach will be explored in Section 8.3.

8.2.3 Comparisons between results from oedometer tests and triaxial tests

The input parameters for Series 7-10 from the oedometer tests were also used to predict the results of $K_0$-compression tests in the triaxial apparatus (Tests TC14, 15 & 30). The comparisons are shown in Figure 8.9. Apart from the input parameters from oedometer tests, only the vertical stress and suction change, and the samples’ initial specific volume from the triaxial tests were used in creating these plots. It can be seen that the LC surface as identified from the oedometer and the triaxial tests (only in $K_0$-condition) can be approximately represented by the same set of parameters.

8.3 Volumetric behaviour in oedometer tests: modified stress approach

In Section 2.4.5, two recently proposed models were reviewed, i.e. Gallipoli et al. (2003) and Wheeler et al. (2003), which use two independent modified stresses as the variables. The main assumptions made in these two models are similar, involving analysis based on Fisher (1926) used to evaluate the stabilising force at the particle contacts, and the relationship between the number of meniscal water lenses and the degree of saturation. In the following, the Gallipoli et al. (2003) modelling assumptions will be validated against the oedometer results. Again, for convenience, the vertical
stress will be assumed to be equivalent to the mean stress. The notation, $p''$, as was used by Gallipoli et al., will refer to the vertical average skeleton stress in this section.

The bonding variable, $\xi$, as defined by Gallipoli et al., is the product of two factors: the degree of saturation of the air, $(1-S_r)$ and the function of suction, $f(s)$:

$$\xi = f(s)(1-S_r)$$

The function $f(s)$ is based on an analysis of the stabilising force at the contact between two spherical grains (Figure 2.38). The term $(1-S_r)$ is assumed to be directly related to the number of menisci per unit volume of the solid fraction. The fundamental modelling assumption specifies that, during virgin compression of an unsaturated soil, the ratio $e/e_s$ is a unique function of the bonding variable $\xi$, where $e_s$ is the void ratio in saturated conditions at the same average skeleton stress, $p''$. Figure 2.39 shows such a function, based on test results of a compacted bentonite-kaolin mixture.

The data on the Normal Compression Lines for Soil A have then been replotted in the same manner as in Figure 2.39. Only results from loading tests are used in preparing these plots since information about the degree of saturation is not available for the wetting tests. Figure 8.10 shows the relationships between $\xi$ and $e/e_s$ for Series 7-10 and 7-13. The variation of $e_s$ with $p''$ is specified using the power law as described in the previous section (Equation 8.1), where $p$ represents $p''$, and $e_s$ equals $n-1$. Also included in Figure 8.10 are the fitting curves, using Equation 2.15. It is evident that for the tests with $\xi > 10$ in which the suctions were higher than 3000kPa, and the filter paper technique was used, the trends do not conform to Equation 2.15. The increase in the ratio $e/e_s$ appears to level off and the scatter in the data is more significant than those at lower values of $\xi$. This is not surprising, as Gallipoli et al. also pointed out that the assumptions regarding the relationship between the number of menisci with $(1-S_r)$ would not hold true for soils in an extremely dry state, where menisci start to disappear from the particle contacts. Nevertheless, the agreement is good between the curve fitting and the experimental results for the values of $\xi$ up to 0.8.

Different sets of parameters are also needed for fitting the data of Series 7-10 and 7-13. Therefore, the influence of fabric, caused by different compaction water contents, is not
taken into account by the modified stress approach. The values of void ratio for Series 7-13 samples on the NCLs (or LC surface) are greater than those of Series 7-10 at the same suction and at the same bonding factor, $\xi$. This is certainly due to the differing fabrics of the two series, as discussed in Chapter 6. Figure 8.11 shows the results from Series 7-10 and 5-10. For values of the bonding factor, $\xi$, between 0.3 and 0.9, the relationship appears to be similar for both Series. However, the $e/e_s$ ratios of Series 5-10 for $\xi$ less than 0.3 lie slightly above the trend for Series 7-10. This deviation corresponds to a difference in the specific volume of about 0.04 and it comes only from Test 5-10B, in which the data during equilibration time was lost (Table 5.2). This data is thus subject to some uncertainty. In general, the behaviour of samples from Series 5-10 and 7-10 with bonding factors less than about 0.9 may be represented by the same parameters. For samples with the bonding factors greater than 0.9, the trends for the change in $e/e_s$ with $\xi$ for the two series are rather different, but it is difficult to draw any conclusions due to the scatter of the data.

8.4 Volumetric behaviour in triaxial tests

In this section, $p$ represents the mean net stress, instead of the net vertical stress as was used in the previous section. Figure 8.12 shows the yield points identified earlier from Groups 1, 2, 3 & 4 triaxial tests on Series 7-10 samples, plotted together with the estimated contours of yield loci at constant suctions. The yield loci are skewed, reflecting the anisotropic structure of the compacted soil. The Normal Compression Lines at constant suctions have also been estimated on a $\nu$-$\ln p$ plot, using the same contour technique as was employed for the oedometer test results. Figures 8.13a, b & c show the NCLs at different stress ratios for suctions of 150, 250 and 600 kPa respectively. The NCL for isotropic compression lies below those for anisotropic states at a suction of 150kPa. As the suction increases, the NCLs for all stress ratios become closer. It is worth mentioning that within the more limited range of net stresses investigated in the triaxial apparatus, the NCLs appear to be linear. If the stress range were larger, the NCL would be likely to be curved as shown in the oedometer test results. The NCLs have then been characterised using the conventional formulation:

$$\nu = N(s) - \lambda(s) \cdot \ln p$$ (8.6)
Figures 8.14 & 8.15 shows the variations of parameters $\lambda(s)$ and $N(s)$ respectively with suction as obtained from the NCLs in Figure 8.13.

8.5 Shear behaviour

The failure envelopes for the Series 7-10 samples have been identified in Chapter 7 as shown in Figures 7.27 & 7.32, both for the peak and the estimated ultimate states. The results of tests on Samples TC19 and 20 (Figure 7.27) demonstrates that, due to their highly overconsolidated stress histories, their ultimate states lie significantly above the envelope identified from the other samples sheared at the same cell pressure of 200kPa. This suggests that the ultimate deviatoric stresses are not a function of the suction and the net stress alone. As discussed in Chapter 2, the degree of saturation contributes significantly to the shear strength of unsaturated soils. Various relationships have been proposed in order to describe the variation of shear strength. In this section, two approaches will be used to fit the data at the ultimate state from the current studies: the conventional stress and the modified stress approaches. The evidence of the critical state will also be discussed.

8.5.1 Critical state framework

Toll (1990) and Toll & Ong (2003) proposed a relationship describing the critical state of unsaturated soils, as explained in Sections 2.4.2 and 2.4.4e. This approach has been used to fit the data for the Series 7-10. Figure 8.16 shows the variations of parameters $M_a$ and $M_b$ with the degree of saturation at the critical state. The values of $M_a$ at the degree of saturation of 100% and at the air-dried state ($S_r \sim 3\%$) are taken directly from the test results shown in Figure 7.32. The end-of-test states were assumed to be the critical states for the air-dried samples. A linear variation of $M_a$ with $S_r$ was initially assumed and the values of $M_b$ were calculated from the test results. The variations of $M_a$ and $M_b$ at different values of $S_r$ were then adjusted iteratively until the smooth variations were obtained with compatible values of the two parameters. The fitting curve for $M_a$ in the figure was derived from Equations (2.7a, b & d). However, the equations specifying the variation of $M_b$ are different to those proposed by Toll & Ong (2003), i.e. Equations (2.7c & d). The minimum value of $M_b$ is specified directly from
the test results of air-dried samples, as opposed to the value of zero used by Toll (1990). The following equations were then used in formulating the variation.

\[
M_a = M_b = M_s \quad ; \text{for } S_r > S_{r1b} \quad (8.7a)
\]

\[
\frac{M_b}{M_s} = \left( \frac{M_b}{M_{s_{\min}}} \right) - \left[ \left( \frac{M_b}{M_{s_{\min}}} \right) - 1 \right] \left( \frac{S_r - S_{r2b}}{S_{r1b} - S_{r2b}} \right)^{b_b} \quad ; \text{for } S_{r2b} < S_r < S_{r1b} \quad (8.7b)
\]

\[
M_b = \left( \frac{M_b}{M_{s_{\min}}} \right) \cdot M_s \quad ; \text{for } S_r < S_{r2b} \quad (8.7c)
\]

The reference degree of saturation, \( S_{r1} \) \& \( S_{r2} \), for the parameters \( M_a \) and \( M_b \) are specified independently and not necessarily the same values, as indicated by an additional subscript \( b \). Nevertheless, values of \( S_{r1a} \) and \( S_{r1b} \) for Soil A are the same and equal to 100%. This trend is similar to the suggestion by Vanapalli et al. (1996) that \( S_{r1} \) represents full saturation (100%), and \( S_{r2} \) represents the degree of saturation at the residual state. Nevertheless, the values of \( S_{r1} \) \& \( S_{r2} \) are determined based on the data at degrees of saturation between 30% to 60% and on some results at air-dried and fully saturated state. More test results at degrees of saturation greater than 60% are needed before any relationships can be established for the strength in that range. The data points included on Figure 8.16 are only from the tests in which the samples appeared more likely to have reached critical states (Tests TC7, 8, 9, 10, 22, 19 \& 20). Table 8.2 summarises the parameters for the two fitting curves.

For fully saturated soils, the critical state framework specifies that at the critical state the soil sample is sheared continuously at a constant volume and a constant shear stress ratio. For drained tests, it is often convenient to express this statement in terms of a stress-dilatancy plot between \( q/p' \) and \( d\varepsilon_v/d\varepsilon_q \) during shearing, where \( q/p' \) reaches the value of \( M_s \) at \( d\varepsilon_v/d\varepsilon_q \) equal to zero. In order to extend this concept to partly saturated soils, the ratio \( (q - s \cdot M_b)/p \), or the ‘mobilised’ \( M_b \) is plotted instead of \( q/p' \). The product \( s \cdot M_b \) can be thought of the additional shear strength due to the suction, which also takes into account the effect of degree of saturation via the term \( M_b \). The term \( M_b \)
during shearing is calculated using the fitting relationships in Figure 8.16 with the current degree of saturation at any stage in the test. Figures 8.17, 8.18 & 8.19 show stress-dilatancy plots for Tests TC7, 8, 19 respectively. The estimated values of \( M_a \) at critical states in these figures were taken directly from the relationship in Figure 8.16. The average values of volumetric strain from the barrelling and right assumptions have been used. These samples initially contracted upon shearing before dilating to the critical state. After the ratio \( \frac{d \varepsilon_v}{d \varepsilon_q} \) reached the minimum value (maximum rate of dilation), the mobilised \( M_a \) still continued to increase. At the phase transformation point, where \( \frac{d \varepsilon_v}{d \varepsilon_q} \) was temporarily zero, the mobilised \( M_a \) was noticeably lower than the value at the critical state. In Test TC8 (Figure 8.18) where a constant volume was not clearly reached, the extrapolated line indicates that the sample would have gone to a critical state eventually. It is worth mentioning again that the fitting relationship in Figure 8.16 was calculated based on the ultimate states of only some tests, which appeared to have reached critical states, and not based on these stress-dilatancy plots. The stress-dilatancy plots were produced in order that the evidence of the critical state could be assessed. All three samples may be considered as being lightly- to moderately-over consolidated. They all failed in a barrelling mode (Table 7.4). This trend was also followed by Samples TC9 and TC20.

Figures 8.20 & 8.21 show the stress-dilatancy plots for Tests TC10 & TC21 respectively. The samples in both tests failed on a distinct plane. Consequently, due to the stress localisation, the critical state was not clearly reached in these tests. The mobilised stress ratios at the phase transformation point \( (\frac{d \varepsilon_v}{d \varepsilon_q} = 0) \) are significantly less than the critical state values estimated from relationship in Figure 8.16. Both Samples TC10 & TC21 were sheared at a relatively high suctions and could be considered as being heavily over-consolidated. Samples TC22 & 29 also followed similar trends to these two samples.

The stress-dilatancy plots for Samples TC13 & 25 are shown in Figures 8.22 & 8.23 respectively. Both samples were sheared while their states were on the yield surface and thus can be considered as being normally consolidated. Even though the shearing was not complete for both tests, they appeared to contract towards the same critical state stress ratio \( M_a \) as estimated from the relationship which was obtained from other tests in
Figure 8.14. Sample TC24 also appeared to follow the same trend, but the data was unclear and therefore is not shown here.

A typical stress-dilatancy plot for an air-dried sample is shown in Figure 8.24 for Sample TC12. The sample failed on a distinct plane. It is interesting to note that the mobilised $M_a$ increased rapidly at an almost constant value of $d\varepsilon_v/d\varepsilon_q$ of 1.5 before starting to move towards the phase transformation point and dilation. The mobilised $M_a$ at the phase transformation point is also higher than the end-of-test point. This trend in the behaviour is similar to that of bonded materials such as cemented sandstones (Coop & Willson, 2003, and Alvarado-Gutierrez, 2005). The difference in the stress-dilatancy relationship between the air-dried sample and other over-consolidated samples at lower suctions (such as TC10 and TC21) can be attributed to the greater degree of cementation in the air-dried samples.

**8.5.2 Volumetric behaviour at the critical state**

Only the tests in which the samples appeared to move towards the constant volume state (TC7, 8, 9, 10, 22, 19 & 20) at the end of test are included in the interpretation of the specific volume at the critical state. Figure 8.25 shows estimated Critical State Lines together with the Isotropic Normal Compression Lines at constant suctions on a $\nu$-log $p$ plot, again estimated using the contour technique. The error bars shown indicate the range of uncertainties in specific volume due to uncertainties in determining volume changes from the radial strain belt measurement. The upper limit shows the value calculated using the right cylinder assumption, while the lower limit shows that of the barrelling assumption. It is also worth mentioning that only relatively few samples can truly be said to have approach the critical state, especially those that exhibit contraction over much of the stress path. There is thus uncertainty about the relevance of critical state to samples that are strongly dilatant. Nevertheless, the estimated CSLs shown in Figure 8.25 do not appear to be parallel to the constant suction NCLs on the $\nu$-log $p$ plot, as would normally be the case for fully saturated soils at zero suction.

As discussed in Section 2.4.4e, Toll (1990) and Toll & Ong (2003) suggested that the parameters for the CSLs are a unique function of the degree of saturation (Equations 2.10a & b). Nevertheless, the data points at the critical state are limited and cannot be
used for the multiple-regression technique as was used by them. However, a fundamental assumption of the constitutive models within the critical state framework is that the CSL on a $\nu - \ln p'$ plot is parallel to the NCL and thus have the same value of gradient, $\lambda$. In this section, the NCLs identified from triaxial tests of Groups 1, 2, 3, & 4 thus could be used to identify the change of the gradients, $\lambda_a$ & $\lambda_b$, with the degree of saturation.

Figure 7.12, however, suggests that the NCL at zero suction might be curved on a $\nu - \ln p'$ plot and converged to a line parallel to the CSL at stresses larger than 200kPa. The power law assumption (Butterfield, 1979) was used to fit this trend and the parameters are summarised in Table 8.3. The equation originally proposed by Toll is therefore modified in order to be consistent with the power law formulation as follows.

$$\ln \nu = \ln \nu_{1s} - b_s \cdot \ln p \quad ; \text{for NCL (or CSL) at zero suction} \quad (8.8a)$$
$$\ln \nu = \ln \nu_{1ab} - b_a \cdot \ln p - b_b \cdot \ln s \quad ; \text{for NCL (or CSL) at suction > 0} \quad (8.8b)$$
$$\nu_{1ab} = 1 + \frac{(\nu_{1s} - 1)}{S_r} \quad (8.8c)$$

where $\nu_{1s}$ and $b_s$ are the fitting parameters of the NCL (or CSL) at zero suction, $S_r$ is the degree of saturation expressed as a ratio, and the values of $b_a$ and $b_b$ equal $b_s$ at the fully saturated state. The value of $b_s$ is the same for both the CSL and NCL, whereas the values of $\nu_{1s}$ are different as shown in Table 8.3.

Using the same technique as suggested in Toll (1990), the variation of the parameters $b_a$ and $b_b$ (which are similar to $\lambda_a$ and $\lambda_b$ respectively) with the degree of saturation is shown in Figure 8.26. In preparing this plot, the variation of $b_b$ with $S_r$ was assumed to be linear, estimated from the multiple linear regression analysis, and then the values of $b_a$ at different $S_r$ values were calculated from the results directly. All data at the critical state and the compression and wetting tests, under isotropic stresses, constant $q/p$, and $K_o$ conditions are included in the plot. The values of $b_a$ from the wetting tests tend to be greater than the general trend at low values of $S_r$ and lower than the general trend at
high values of \( S_r \). Nevertheless, it can be seen that the relationships are approximately unique for all testing conditions. The assumption that the gradient of the CSL and the NCL are of the same value may be considered valid only when the degree of saturation is included in the framework. Another interesting point observed in Figure 8.26 is that the value of \( b_b \) (based on multiple regression analysis) is negative at \( S_r \) values less than 75%, whereas the results reported by Toll (1990) and Toll & Ng (2003) show only positive values. This negative value reflects the collapsible nature of the compacted Soil A.

8.5.3 Modified stress approach

Early researchers have long been using the single modified stress approach to describe the shear strength of unsaturated soils (e.g. Bishop et al., 1960). Recently, Tarantino & Tombolato (2005) proposed a simple version of this approach for compacted kaolin by visualising the clay as a granular material, with the grains replaced by the saturated clay packets (aggregates). They argued that the degree of saturation that is effective in controlling the mechanical behaviour of the aggregate fabric is the degree of saturation of the macropores, \( S_{r, r} = \frac{e - e_{um}}{1 - e_{um}} \), where \( e_w \) is termed the water ratio, which is the ratio of water volume over solid volume, \( e_{w} = S_r \cdot e \). The term \( e_{um} \) is the microstructural water ratio, originally defined by Romero & Vaunat (2000) as the value of \( e_w \) at which the retention curves of the same material with different densities converge. For the Soil A compacted at 10% moisture content, Figure 4.4b suggested that the convergence occurred at a gravimetric water content of about 3%, corresponding to a value of \( e_{um} \) of 0.0792. The shear strength can then be expressed in the same way as for fully saturated soils as:

\[
q = M_s \cdot (p + s \cdot S_{r, r}) 
\]  

(8.9)

where \( (p + s \cdot S_{r, r}) \) is similar to the effective stress, \( s \) is the suction and \( M_s \) is the critical state stress ratio for the fully saturated state. The values of \( q \) at the critical state are then predicted using this approach and are shown in Figure 8.27, compared with the experimental values. A good agreement is evident between the predicted and the
The results from the tests on air-dried samples are excluded from this figure, since these samples were at the moisture content less than the microstructural value. The ultimate shear strength of the air-dried samples however is far greater than the value calculated from Equation 8.9 with the value of $S_{sm}$ set to zero. It can then be suggested that for the samples drier than those at the “microstructural state”, the contribution from the water menisci, or the bonding factor, to the ultimate shear strength becomes significant, and obviously a change to the modified stress approach is required to predict accurately the behaviour of samples in this condition.

The volumetric behaviour at the critical state has then been interpreted with the modified stress approach, as was carried out for the oedometer test results in Section 8.3. The NCLs identified from the compression and wetting tests are again included in this interpretation. The power law is used to describe the NCL and CSL at zero suction using Equation 8.1 and the parameters in Table 8.3. Due to the lack of data for anisotropic compression at zero suction, the isotropic NCL was used instead to calculate the value of $e_s$. Figure 8.28 shows the relationship between $e/e_s$ and $\xi$ for samples on the Critical State Surface and Normal Compression Surfaces at different stress ratios from loading and wetting tests. The relationship for the critical state differs from that for the normal compression states. Subsequently, the interpretation was modified and the values of $e_s$ for the critical state were calculated based on the parameters from the fully saturated isotropic NCL instead (i.e. $b_{is} = 0.0480$ and $\nu_{is} = 1.924$). Figure 8.29 shows the variation of $e/e_s$ with $\xi$ based on this modified interpretation. The variation appears more unique for samples in all states. Nevertheless, the scatter in the variation suggests that this approach would only give approximate predictions.

### 8.5.4 Relationship between water content and suction at the critical state

Figure 8.30 shows the relationship between the water content and suction at the ultimate state. Even though only few samples appeared to have constant volume state at the end of test, most samples appeared to have a relatively constant suction towards the end. The constant water content condition was also maintained during shearing. It thus follows that the data shown in Figure 8.30 would be more representative of the critical state than the specific volume plot in Figure 8.25. The data from all the samples appear
to lie on a unique line. This line has been identified as the continuously disturbed line (CDL) by Croney & Coleman (1954) and discussed by Ridley (1995). Tarantino & Tambolato (2005), based on their experimental results on compacted kaolin, showed that this line approximates to the main wetting retention curves. They suggested that the main wetting surface also acted as boundary surface even during shearing. A similar comparison will be made in the next section for compacted Soil A.

8.6 Soil-Water Retention Surface

As discussed in Section 2.4.4h, Gallipoli et al. (2003) suggest a hypothesis stating that in absence of the hydraulic hysteresis, there is a unique relationship between degree of saturation, suction, and specific volume for a given soil (Equation 2.11). Several simplifying assumptions are involved in this hypothesis. The influence of shear strain on the degree of saturation is ignored, and no distinction is made between elastic and plastic components of the strains. In this section, the results from unconfined wetting/drying, oedometer and triaxial tests will be interpreted based on this hypothesis.

8.6.1 Comparison between measurements using filter paper and the suction probe

Prior to the presentation of the SWRS for Soil A, a comparison is made between the suction measurements using the filter paper technique and the suction probe in the oedometer tests. The unconfined sample underwent less than 0.25% radial strain and thus is considered to be similar to the Ko-wetted samples. Figure 8.31 shows the results on the first wetting after compaction from the unconfined wetting test (7-10W) and the initial state of the oedometer tests (7-10D, G, H & K). The void ratios for these samples are of similar value (Figure 8.31b).

It can be seen that for suctions greater than about 100 kPa, the filter paper technique consistently yielded a higher suction than the values from the suction probe. As the suction reduced to values less than 100 kPa, both techniques tend to give similar measurements. The contact conditions between the filter papers and the soil sample were observed during the unconfined wetting/drying test. The stickyness of the filter papers to the soil sample appeared to increase as the suction decreased. It might be expected that since the contact between the samples and the filter paper was not intimately made at higher suctions, the measured value might not be truly representative.
of the matrix or the total suction. In addition, the transfer of moisture between the filter paper and the soil sample would be partially through the vapour phase and partially through the liquid phase. The vapour transfer would certainly require a longer time than the liquid transfer. The measurements using the suction probe involved a more intimate contact between the porous stone and the sample; otherwise the probe would have cavitated (or experienced tension breakdown). Thus, the filter paper technique seems to indicate a higher suction than actually was within the sample for suctions greater than 100kPa. Nevertheless, for suctions above 2000kPa, the filter paper was the only technique available in the laboratory for measuring suctions, and considering that the suction measured would be the total suction, whose measuring condition required no contact between samples and the papers, their measurements in this range was considered reliable. Accordingly, in the interpretation of the SWRS, the suction probe measurements were used for suctions less than 1500kPa, the suction plate measurements were used for suctions between 1 and 10kPa, and only for the suctions greater than 2000kPa were the data from the filter paper technique used.

8.6.2 Main boundary wetting surface for $K_o$ conditions

The results from the oedometer tests, the suction plate, and the filter paper were used to prepare the main boundary wetting surface at different void ratios using the contour technique with the Surfer© software. Appendix 3 gives the details of procedure followed in the data processing. Figure 8.32 shows the resulting contours for samples compacted at a water content of 10% with different constant void ratios and the fitting curves. Figure 8.32b is plotted without the grid data points for clarity of the contours. It is noteworthy that in preparing the data for processing, the results from tests that involved initial drying were excluded, since the sample would have followed the scanning surface as opposed to the main wetting surface. The degree of saturation at the transition between macro- and micro- structural levels increases with decreasing void ratio. The curve fitting for these contours was carried out using the functions proposed by Gitirana & Fredlund (2004). These equations were chosen as opposed to other models (e.g. Van Genutchen, 1980) since their input parameters are more representative of the various features of the retention curves and also since they are capable of reproducing the bi-modality of the retention curves. A bi-modal retention curve can be represented by four hyperbolas with eight parameters. The input parameters for the
fitting curves, summarised in Table 8.4, were derived by trial & error, based on visual observation of the accuracy of fit of the curves. The sharpness of the transitions at the bending points on the retention curve is defined by parameter $a$, while $\psi_b$ represents the blow-through suction, $\psi_{res}$ the residual suction, $S_{res}$ the degree of saturation at the residual point, and $S_b$ the degree of saturation at the second blow-through value. The subscripts 1 and 2 represent the macro- and micro-structural levels respectively. Appendix 4 gives the details of the equations. Figure 8.33d also suggests that the value of $\psi_{b2}$ increases as the void ratio increases. This is somewhat puzzling and contradicting to the trend observed in Figure 8.33a for $\psi_{b1}$, which suggests that the air-occlusion suction (the name for the blow-through suction on the wetting path) increases as the sample becomes denser. No clear explanation is arrived at of this ‘enigmatic’ form of the curve at the moment.

Figure 8.33 shows the variations of these parameters with void ratio, together with their fitting curves. These mathematical expressions for the Soil-Water Retention Surface were then used to fit the experimental data as shown in Figure 8.31. The accuracy of fit of the expression is within 10% of the measured degree of saturation.

### 8.6.3 Influence of different strain components

The contours shown in Figure 8.32 are based on the experiments carried out under a $K_o$ condition. The $K_o$ condition can be specified by the constant ratio of $\varepsilon_q/\varepsilon_v$ of $2/3$. The strains used in the calculation of the ratio are the accumulated values from the as-compacted state, which was also arrived at under a $K_o$ condition. The surface in Figure 8.32 thus represents the pore size distribution of the sample complying with the condition of constant $\varepsilon_q/\varepsilon_v$ of $2/3$. Under triaxial test conditions, the values of $\varepsilon_q/\varepsilon_v$ can deviate considerably from the $K_o$-condition. The influence of changes in $\varepsilon_q/\varepsilon_v$ on the main wetting surface will be presented in the following paragraph.

The fitting expressions for the contours in Figure 8.32 are used to predict the degree of saturation from triaxial tests following a wide range of stress paths, including shearing, compression, and wetting. Again, tests that involved initial drying after compaction
were excluded from the analysis since the samples would have followed the scanning surface and not the main wetting surface. Figure 8.35 shows the comparison between the predictions and the experimental results for $\varepsilon_q / \varepsilon_v > 2/3$. The measured values of degree of saturation appear to comply with the surface identified earlier for the Ko-condition. The data for shearing at the critical state is also included in this plot, and thus support the comment made by Tarantino & Tombalo (2005) as described in Section 8.5.4. The data with $\varepsilon_q / \varepsilon_v < 2/3$ shows some deviation from the general trend and they were sorted so that only data with suction more than 190 kPa was included. As shown in Figure 8.35, the results are in a good agreement with the Ko-wetting surface.

Figure 8.36 shows the data for suctions less than 190 kPa and $\varepsilon_q / \varepsilon_v$ less than 2/3. It can be seen that only when these conditions are met, did the degree of saturation start to deviate from the trend observed for samples in Ko condition. This situation is common in isotropic compression and wetting tests. Based on this observation, Equation 2.11 might be tentatively modified to:

$$S_r = S^{Ko}(\varepsilon, s) + S^d_r\left(\frac{\varepsilon_q}{\varepsilon_v}, s\right)$$

(8.10)

The function $S^{Ko}_r$ is specified as the expression shown as the surface in Figure 8.32. The value of $S^d_r$ is then the deviation of the measured values from the prediction of $S^{Ko}_r$ and thus could be identified from the experimental data. The relationship between $S^d_r$, $\varepsilon_q / \varepsilon_v$ and suction, s, can then be estimated using the contour technique as described in Appendix 3. Figure 8.37 shows the data from the contour technique and the fitting lines for variation of $S^d_r$ with natural logarithm of suction at constant values of $(2/3 - \varepsilon_q / \varepsilon_v)$. Considering that $S^d_r$ is equal zero when $(2/3 - \varepsilon_q / \varepsilon_v)$ is zero, the function $S^d_r\left(\frac{\varepsilon_q}{\varepsilon_v}, s\right)$ can then be expressed as follows,

$$S^d_r = \left(\frac{2}{3} - \frac{\varepsilon_q}{\varepsilon_v}\right) \cdot (28 - 5.344 \cdot \ln s)$$

(8.11)
Equations 8.10 and 8.11 can then be used to predict the experimental results. As shown in Figure 8.38, the prediction becomes more accurate with the influence of different strain components taken into account.

### 8.7 Final Comments

A variety of models explaining different aspects of unsaturated behaviour have been validated against the experimental results of compacted Soil A. Both types of models using conventional and modified stress approaches can provide reasonably good fits to the experimental results. The limitations of the conventional stress approach, as highlighted in Chapter 2, include the inaccurate prediction of paths involving wetting-drying cycles and of transient behaviour from an unsaturated to a fully saturated state. The power law relationships, suggested in Section 8.2.1 for describing the Loading-Collapse surface can reproduce this transient nature by providing the convergence at high net stresses between NCLs at constant suction and that at zero suction. Nevertheless, the conventional stress model cannot reproduce the paths involving wetting-drying cycles. This type of path was not investigated in the main experiments.

The basic assumption for the Gallipoli et al. (2003) model has been tested against the compacted Soil A. In general the assumption is valid at suctions lower than 1500kPa. For higher suctions, the number of meniscus water lenses might not be directly related to the degree of saturation as assumed in the model. A modification is therefore needed. Both types of models, those employing conventional stress variables and those employing modified variables, were used to fit the behaviours of Soil A compacted at two different moisture contents. It has been shown that both types of models require different parameters to characterise these materials due to the differences in their fabrics.

The concept of a retention surface (e.g. Gallipoli et al., 2003b) has also been tested with the results on compacted Soil A. A fairly unique surface has been identified. In addition, the influence of distortion of the pore structure on the non-uniqueness of this surface was identified and taken into account in terms of the $\varepsilon_q / \varepsilon_v$ ratio. A suggestion has been made of how to include this influence in the formulation of the surface.
Table 8.1 Parameters for the Loading-Collapse surface from the oedometer tests

<table>
<thead>
<tr>
<th>$v_1$</th>
<th>$b$</th>
<th>$\kappa$</th>
<th>$\kappa_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.075</td>
<td>0.0533</td>
<td>0.004</td>
<td>0.001</td>
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</table>

Table 8.2 Parameter for the variations of $M_a$ and $M_b$ for compacted Soil A

<table>
<thead>
<tr>
<th>$M_s$</th>
<th>1.32</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{r1a}$</td>
<td>100</td>
</tr>
<tr>
<td>$S_{r2a}$</td>
<td>0</td>
</tr>
<tr>
<td>$k_a$</td>
<td>0.5</td>
</tr>
<tr>
<td>$(M_a/M_s)_{max}$</td>
<td>1.153</td>
</tr>
<tr>
<td>$S_{r1b}$</td>
<td>100</td>
</tr>
<tr>
<td>$S_{r2b}$</td>
<td>6</td>
</tr>
<tr>
<td>$k_b$</td>
<td>1.5</td>
</tr>
<tr>
<td>$(M_a/M_s)_{min}$</td>
<td>0.00452</td>
</tr>
</tbody>
</table>

Table 8.3 Parameters for Power law expressions of NCL and CSL at zero suction from Triaxial tests on Series 7-10 samples

<table>
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<tr>
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<th>NCL</th>
<th>CSL</th>
</tr>
</thead>
<tbody>
<tr>
<td>$b_s$</td>
<td>0.0480</td>
<td>0.0480</td>
</tr>
<tr>
<td>$v_{1s}$</td>
<td>1.924</td>
<td>1.830</td>
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</table>
Table 8.4 Summary of curve fitting parameters for the main wetting surface of Soil A, compacted at w/c of 10%

<table>
<thead>
<tr>
<th>e</th>
<th>a</th>
<th>$\Psi_{bl}$ (kPa)</th>
<th>$\Psi_{res1}$ (kPa)</th>
<th>$S_{res1}$</th>
<th>$\Psi_{b2}$ (kPa)</th>
<th>$S_b$</th>
<th>$\Psi_{res2}$ (kPa)</th>
<th>$S_{res2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.77</td>
<td>0.05</td>
<td>3.5</td>
<td>40</td>
<td>0.50</td>
<td>1300</td>
<td>0.33</td>
<td>4500</td>
<td>0.06</td>
</tr>
<tr>
<td>0.70</td>
<td>0.05</td>
<td>4.5</td>
<td>35</td>
<td>0.56</td>
<td>1200</td>
<td>0.35</td>
<td>4500</td>
<td>0.06</td>
</tr>
<tr>
<td>0.60</td>
<td>0.05</td>
<td>5.5</td>
<td>25</td>
<td>0.68</td>
<td>800</td>
<td>0.43</td>
<td>4500</td>
<td>0.06</td>
</tr>
<tr>
<td>0.50</td>
<td>0.05</td>
<td>6.5</td>
<td>22.5</td>
<td>0.78</td>
<td>700</td>
<td>0.52</td>
<td>4500</td>
<td>0.06</td>
</tr>
<tr>
<td>0.40</td>
<td>0.05</td>
<td>6.5</td>
<td>15</td>
<td>0.85</td>
<td>150</td>
<td>0.86</td>
<td>4500</td>
<td>0.06</td>
</tr>
<tr>
<td>0.35</td>
<td>0.05</td>
<td>6.5</td>
<td>12</td>
<td>0.86</td>
<td>150</td>
<td>0.9</td>
<td>4500</td>
<td>0.06</td>
</tr>
</tbody>
</table>
Figure 8.1 Compaction path and the contours of NCLs for Series 7-10 samples (NCLs data from Figure 5.32)
Figure 8.2 Schematic representation of Equation 8.3
Figure 8.3 Variations of the parameters $\nu$ and $\beta$ with suction for Series 7-10 & 7-13
Figure 8.4 Comparison between the experimental results and predictions from the fitting expressions for Series 7-10

Figure 8.5 Comparison between the experimental results and predictions from the fitting expressions for Test 7-10H
Figure 8.6 Comparison between the experimental results and predictions from the fitting expressions for Test 7-10Q
Figure 8.7 Comparison between the experimental results (post yield) of Series 5-10 and their predictions using the fitting parameters from Series 7-10.

Figure 8.8 Comparison between the experimental results and predictions from the fitting expressions for Series 7-13.
Figure 8.9 Comparison between the experimental results and predictions from the fitting expressions of Series 7-10 oedometer tests, for triaxial tests; a) TC14, b) TC15, and c) TC30
Figure 8.9 Comparison between the experimental results and predictions from the fitting expressions of Series 7-10 oedometer tests, for triaxial tests; a) TC14, b) TC15, and c) TC30 (continued)
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Figure 8.11 Relationships between ratio $e / e_s$ and bonding ratio $\xi$ during oedometric virgin loading for Soil A from Series 7-10 and 5-10

Figure 8.12 Contours of yield loci at different suctions from Groups 1, 2, 3 & 4 triaxial tests
Figure 8.13 NCLs for isotropic, constant $q/p$, and $K_o$ compression tests at a constant suction of: a) 150 kPa, b) 250 kPa & c) 600 kPa (see Figures 7.15, 7.16 & 7.17 for raw data)
Figure 8.13 NCLs for isotropic, constant q/p, and Ko compression tests at a constant suction of: a) 150 kPa, b) 250 kPa & c) 600 kPa (continued)

Figure 8.14 Variation of $N(s)$ with suction for isotropic, constant q/p, and Ko compression tests
Figure 8.15 Variation of $\lambda(s)$ with suction for isotropic, constant $q/p$, and $K_o$ compression tests

Figure 8.16 Variations of $M_a$ and $M_b$ with the degree of saturation for compacted Soil A of Series 7-10
Figure 8.17 Stress-dilatancy relationship for Test TC7

Figure 8.18 Stress-dilatancy relationship for Test TC8
Figure 8.19 Stress-dilatancy relationship for Test TC19

Figure 8.20 Stress-dilatancy relationship for Test TC10
Figure 8.21 Stress-dilatancy relationship for Test TC21

Figure 8.22 Stress-dilatancy relationship for Test TC13
Figure 8.23 Stress-dilatancy relationship for Test TC25

Figure 8.24 Stress-dilatancy relationship for Test TC12
Figure 8.25 Critical State Lines and Isotropic Normal Compression Lines at constant suctions

Figure 8.26 Relationships between $b_a$ and $b_b$ with the degree of saturation for CSLs and NCLs
Figure 8.27 Comparison between the predicted and experimental deviatoric stresses at the critical state using Tarantino & Tombolato (2005) approach

Figure 8.28 Relationship between $e/e_s$ and $\xi$ for Series 7-10 from triaxial tests
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Figure 8.29 Relationship between $e/e_s$ and $\xi$ for Series 7-10 from triaxial tests, based on the modified interpretation

Figure 8.30 Relationships between the water content and suction at the critical state for Series 7-10
Figure 8.31 Comparison between the suction measurements using the filter paper technique and the suction probe
Figure 8.32 Contours of the main wetting surface for Soil A compacted at 10% water content; a) with raw grid data, b) only with fitting expressions
Figure 8.33 Variations of the fitting parameters with suction for the main wetting surface of Soil A compacted at 10% water content
Figure 8.33 Variations of the fitting parameters with suction for the main wetting surface of Soil A compacted at 10% water content (continued)
Figure 8.33 Variations of the fitting parameters with suction for the main wetting surface of Soil A compacted at 10% water content (continued)

Figure 8.34 Comparison between the predicted and measured degrees of saturation for Soil A, compacted at 10% w/c from oedometer, filter paper and suction plate tests
Figure 8.35 Comparison between the predicted and measured degree of saturation for Soil A, compacted at 10% w/c for triaxial tests with $\frac{\varepsilon_v}{\varepsilon_q} \geq \frac{2}{3}$ and with $\frac{\varepsilon_v}{\varepsilon_q} < \frac{2}{3}$ and suction $> 190$ kPa

Figure 8.36 Comparison between the predicted and measured degree of saturation for Soil A, compacted at 10% w/c for triaxial tests with suctions $< 190$ kPa and $\frac{\varepsilon_v}{\varepsilon_q} < \frac{2}{3}$
Figure 8.37 Estimated contours of variation of $S_r^d$ with suction at constant $\frac{\varepsilon_q}{\varepsilon_v}$

Figure 8.38 Comparison between the predicted and the measured degree of saturation with suctions < 250 kPa and $\frac{\varepsilon_q}{\varepsilon_v} < 2/3$, based on Equations 8.10 & 8.11
CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

The conclusions drawn from the current research project are presented in this chapter, according to different subject areas; namely, experimental technique, collapse behaviour, water retention behaviour, shear behaviour and modelling of unsaturated soils.

9.1 Experimental technique

9.1.1 Summary

- The suction-monitored oedometer test provides a quick and easy means of characterizing the collapse behaviour of unsaturated soils. A number of different paths can be followed with only minor modifications to a standard oedometer apparatus.

- The use of suction probe under cell pressure inside the triaxial apparatus created problems of electrical drift in the measurement from the probe due to water leaking into its body. This problem was eventually avoided by replacing the water inside the cell with glycerol. Due to the glycerol’s relatively high viscosity and low electrical conductivity, the leaking problem no longer existed. However, the use of glycerol created another problem of osmosis through the membrane, thus drying the sample slowly during long-term tests. Nevertheless, for tests with relatively short periods (less than 1 week), this effect was only minimal, and for longer tests, a linear correction of water content provided a satisfactory solution to the problem.

- The air-regulated drying system originally developed by Cunningham (2000) was improved by means of incorporating a relative humidity sensor block. With this technique, the change of water content of soil sample during the test can be estimated.

- A wetting system for the triaxial apparatus was developed, capable of wetting the sample gradually in a controlled manner. The system was still operated manually, although there seems to be no obvious need to automate it.
Chapter 9 - Conclusions and recommendations for future work

- The filter paper technique appeared to consistently indicate higher suctions than those measured using the suction probe for suctions greater than 100kPa. This is believed to be due to the low degree of saturation of compacted Soil A, especially of Series 7-10, giving rise to poor contact between the filter paper and the soil sample.

9.1.2 Recommendations for future work

- The uniformity has been confirmed of the suction distribution within the sample during triaxial test based on the suction measurements. However, the evidence for the uniformity of the water content has not been given in the present tests. In order to check its vertical water content distribution, the triaxial sample should be chopped up into horizontal disks after testing, and the water content is determined for each disk.

- Further work is needed in the interpretation of the radial strain belt in order to calculate the most realistic volume changes. A comparison between different interpretation methods with the other means of global volume measurement, such as the volume gauge, might be required.

- Recent constitutive models (e.g. Wheeler et al., 2003, and Gallipoli et al., 2003a) employ modified stress variables that incorporate the degree of saturation. This trend in data interpretation might make the need to perform tests at a constant suction condition less necessary, as the data from constant water content tests can also be used with the same level of confidence. In addition, constant suction tests normally involve drying the sample during tests. This condition is fundamentally different to the critical situations more likely to happen in the field involving wetting of the ground or loading under constant water content condition. The influence of hydraulic hysteresis might make the results of constant suction tests unrepresentative of the actual field behaviour.

- The wetting system originally developed for the triaxial apparatus could be incorporated into the oedometer apparatus. This would enable a precise control of wetting in the oedometer with a system that is simpler than the osmotic system developed by Dineen (1997), although more limited stress paths could be followed by the proposed system.
9.2 Collapse behaviour

9.2.1 Summary

- The uniqueness of the Loading-Collapse (or Normal Compression) surface has been demonstrated for tests involving monotonic loading and wetting paths, both from oedometer and triaxial tests.
- The influence on the collapse behaviour of the compaction water content has been demonstrated by the difference in location of the Loading-Collapse surface for the Series 7-10 and Series 7-13 samples. The samples that were compacted at a higher water content (Series 7-13) yielded at a higher net stress at the same values of suction and void ratio, than samples at a lower compaction water content (Series 7-10). This difference might be explained by the larger average aggregate sizes of the Series 7-13 soil as observed in the fabric study. Two different sets of parameters are needed to characterize the behaviour of these two series, for both types of model: those employing conventional variables and those employing modified variables.
- Samples compacted at the same water content but different dry densities (i.e. Series 7-10 and 5-10) can be modelled using the same parameters.
- Due to the $K_o$ condition imposed on the samples during compaction, the behaviour observed in the triaxial tests shows strong anisotropy both in the skewed yield surface and plastic potentials.

9.2.2 Recommendations for future work

- The influence of fabrics induced by different compaction water contents should be investigated further with samples compacted at a wider range of different water contents.
- The testing technique used in this current research could be extended for studies of natural collapsible soils such as loess, which are of more practical significance.
- A further set of tests for evaluating the uniqueness of the LC surface would be ones wetted up at various initial suctions while keeping the sample at an approximately constant volume and measuring the changes in net stress and
suction. These tests might be carried out in the oedometer or triaxial apparatus, under one-dimensional or isotropic conditions.

9.3 Soil-Water Retention behaviour

9.3.1 Summary

- The concept of the Soil-Water Retention Surface by Gallipoli et al. (2003), which includes the influence of void ratio on the retention curves, provides a satisfactory fitting to the experimental results for the $K_o$-condition.
- For triaxial test results, for which the $K_o$-condition no longer holds, some deviation from the surface is observed in certain circumstances. A tentative approach has been proposed to take into account the influence on the retention surface of the distortion of pore structure from the original $K_o$-condition.

9.3.2 Recommendations for future work

- Only the main wetting surface has been identified together with its mathematical expression. A similar approach should be pursued for the main drying surface and the scanning surfaces in between. More experimental evidence would be required for this purpose.

9.4 Shear behaviour

9.4.1 Summary

- The fully saturated behaviour of Soil A has been investigated both in its reconstituted and the compacted states and found to be typical of low plasticity clays.
- For unsaturated compacted Soil A, a unique relationship of the shear strength with suction and net stress is only obtained when the degree of saturation is also taken into account.
- Stress-dilatancy relationships were plotted, which provided an indication of the critical state at the end of tests. The critical state appeared to be reached in the cases where the samples were at relatively low suctions and show contractant behaviour over much of the stress paths. Nevertheless, for tests in which the
suctions were relatively high at low cell pressures, the samples failed along a distinct failure plane. Due to dilation and strain localization along the shear band, the critical state for these samples did not appear to be reached at the end of test. The identification of critical states for these samples with relatively high suctions at low cell pressure is less certain.

- The estimated critical state parameters in terms of volumetric behaviour were identified, based on Toll (1990) model, with some modification. The relationship between these parameters and the degree of saturation is consistent with those of the Normal Compression Surfaces identified from a variety of stress paths, although the same uncertainties still exist regarding the tests in which the critical states were not clearly reached.
- Shearing tests on air-dried samples are relatively easy to perform and yet their results provide a valuable boundary condition for the strength relationships of unsaturated soils.

9.4.2 Recommendations for future work

- Shearing tests at higher cell pressure (>200kPa) would provide more understanding into the behaviour of the material. This would require some modification of the apparatus.

9.5 Modelling of unsaturated soils

9.5.1 Summary

- Due to the monotonic nature of the paths followed in most tests, a framework using conventional stress variables appears to be valid based on the test results from the current research. A new formulation of the Loading Collapse surface within this type of framework has been given, to provide a better fit to the results of oedometer tests.
- The assumption of the Gallipoli et al. (2003) model, which employs modified stress variables, has been validated. The assumption involves the analysis of stabilizing force provided by the meniscal water lenses and the relationship between the number of these lenses with the degree of saturation. It appears to be valid for suctions less than 1500kPa and for degrees of saturation greater than
30%. For samples drier than these conditions, some modification is required to the model. This observation also applies to the application of the modified stress approach for shear behaviour.

9.5.2 Recommendations for future work

- The modelling of the Soil Water Retention Surface, taking into account the influence of hydraulic hysteresis as well as the bi-modality of the surface, is an area requiring further study.
REFERENCE


APPENDIX 1
ESTIMATION OF MOISTURE CHANGE DURING DRYING

Section 3.8.3 explains the experimental set up of the air-regulated system, originally developed by Cunningham (2000), and further refined in the current research project. As shown in Figure 3.24, the system included a source of dry air from a mains compressor, a valve controlling the flow of air-in, a pressure transducer which indicated when the air-flow was on or off, and finally, a sensor measuring the relative humidity of the moist air coming out of the sample. This appendix describes the method used to estimate the amount of moisture evaporating from the sample during drying.

The assumptions made in the calculation are listed as follows.

- The relative humidity of dry air from the mains compressor, $h_{in}$, was always constant (e.g. for Test TC29, $h_{in}$ was 5%). Its value was measured only once before drying took place.
- The flow of dry air mass, $q_d$, which was controlled by a manostat, was constant throughout the drying stage.
- Since the temperature of the laboratory was controlled within ± 1°C, the saturated vapour pressure, $e_a$, (i.e. at 100% relative humidity), which is a function of temperature, was assumed constant.

From Penman (1955), the relative humidity, $h$, is defined as:

$$h = 100 \cdot \frac{e}{e_a}$$  \hspace{1cm} (A1.1)

where $e$ is the vapour pressure.

The mixing ratio, $x$ (g/kg), is defined as the mass of water vapour per unit mass of dry air and can be related to the pressure by:

$$x = \frac{622 \cdot e}{p}$$  \hspace{1cm} (A1.2)
where \( p \) is the total atmospheric pressure (also assumed to be constant), and where \( e \) is small compared to \( p \).

From Equation A1.1 and A1.2:

\[
x = h \cdot x_a / 100
\]  

\( (A1.3) \)

where \( x_a \) is the saturated mixing ratio, which is again assumed to be constant.

From the mass continuity equation, the mass of vapour coming into the sample plus the evaporation from it equals the mass of vapour coming out of the sample, thus giving:

\[
Evaporation(t) = \int_{0}^{t} (q_d \cdot \frac{h_{\text{out}}}{100} \cdot x_a) dt - \int_{0}^{t} (q_d \cdot \frac{h_{\text{in}}}{100} \cdot x_a) dt
\]  

\( (A1.4) \)

where \( t \) is the time (sec) during a drying stage, \( q_d \) the flow rate of dry air mass (kg/sec) and \( Evaporation(t) \) is the amount of evaporation from the sample since the start of drying until time \( t \). As \( q_d \) and \( x_a \) are assumed constant, Equation A1.4 becomes

\[
Evaporation(t) = Q \cdot \int_{0}^{t} (h_{\text{out}} - h_{\text{in}}) dt
\]  

\( (A1.5) \)

where \( Q = q_d \cdot \frac{x_a}{100}, \) (g/sec), which is a constant and can be calculated using the total amount of evaporation, which is the difference between the initial and final water mass of the sample:

\[
Evaporation(t_f) = \frac{(w_i - w_f)}{100} \cdot m_d
\]  

\( (A1.6) \)

where \( t_f \) is the elapsed time (sec) for the whole drying stage, \( w_i \) the water content (%) of the sample before drying, \( w_f \) the water content (%) after drying, and \( m_d \) the dry mass of the sample (g). From Equations A1.5 and A1.6, \( Q \) can be estimated and then the amount of evaporation at any time \( t \) can be calculated by direct integration.
In the present studies, the integration of Equation A1.5 was carried out using the trapezoidal approximation, based on the scanning time interval \((dt)\) employed during the test.

Referring back to Figure 3.24, when the control valve switched off the air-in flow and the pressure transducer indicated no-flow, the value of \(h_{\text{out}}\) was set equal to \(h_{\text{in}}\) in the calculation to indicate that there was no evaporation (since there was no flow) during these times.
APPENDIX 2
ESTIMATION OF VOLUMETRIC STRAIN BASED ON A BARRELLING ASSUMPTION

As explained in Section 3.8.4g, the barreling assumption was made in order to estimate the global volumetric strain, based on the measurements from the local axial and radial strain devices. This assumption was made only during shearing and only when the sample shape appeared to be noticeably barreled.

Figures A2.1 & A2.2 shows the deformations of samples during compressive shearing, based on the right cylinder and barreling assumptions respectively.

$$D = \text{Diameter}$$

![Figure A2.1 Right cylinder](image1)

![Figure A2.2 Barrelling](image2)

The volumetric strain, $\varepsilon_v$ was calculated using the expression $\varepsilon_v = \varepsilon_a + 2 \cdot \varepsilon_r$, where $\varepsilon_r$ was the measured local radial strain, $\varepsilon_r^m$, if the right cylinder assumption was used and $\varepsilon_r$ was the corrected radial strain, $\varepsilon_r^c$, if the barreling assumption was used.

The final diameters of the barreled sample were measured at $n$ different elevations ($n = 9$ elevations in the present study), $D_1, D_2, \ldots, D_n$. The number of elevations $n$ had to be odd number and thus the measured local radial strain represented the measurement at the elevation $(n + 1)/2$ with the diameter of $D_{(n+1)/2}$. The lowest value of these final diameters was then considered to be the threshold diameter, $\bar{D}$. It was normally either $D_1$ or $D_n$. 

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In the calculation of strains during shearing, the right cylinder assumption was used from the start of shearing up to the point where the diameter of the sample corresponded to the value of $D_j$. Beyond this point, the barreling assumption was used and the value of the corrected radial strain, $\varepsilon^c_r$, was calculated as follows.

An assumption was made that the ratio, $x_i = \frac{(D_j - D_i)}{(D_{(n+1)/2} - D_j)}$ was constant throughout barreling shearing, where $D_i$ was the diameter at any elevation at any particular time during shearing. In other words, at any time during barreling shearing, the difference between the diameter at any elevation and the threshold diameter was always proportional to this difference at the mid-height elevation, $(n + 1)/2$.

The value of corrected radial strain $\varepsilon^c_r$ was then expressed as:

$$\varepsilon^c_r = \varepsilon^r + AX \cdot (\varepsilon^m_r - \varepsilon^r)$$

where $\varepsilon^r$ was the radial strain at the threshold point, $\varepsilon^m_r$ the measured radial strain (at mid-height) and $AX$ the weighted average value of ratio $x_i$, which was defined as,

$$AX = \frac{1}{(2n-2)} \left( x_1 + x_n + 2 \cdot \sum_{i=2}^{n-1} x_i \right)$$

This formulation was based on a linear estimation for volume of an irregular prism shape.

The value of cross-sectional area used for the calculation of the deviatoric stress was the average value from $\varepsilon^m_r$ and $\varepsilon^c_r$. When the working range of the radial strain device had been reached, the volumetric strain was assumed to remain constant so that the cross-sectional area was calculated solely from the axial strain measurement.
APPENDIX 3
CONTOURING TECHNIQUE USED FOR DATA INTERPRETATION

Some data interpretation presented in Chapters 5, 7 and 8, involved identifying the surfaces of different functions that describe certain volumetric behaviour of compacted Soil A. These functions are the Loading-Collapse surface and the main wetting surface. Experimental results obtained directly from the tests normally did not consist of data at a constant value of one variable. Accordingly, a contouring technique was resorted to in the interpretation of these data.

A3.1 Contouring technique for the Loading-Collapse surface
For the oedometer test results, the data were available in terms of vertical stress, $\sigma_v$, potential collapse, $\nu_c$, and suction, $s$. The objective was to identify, based on interpolation from these data, the variation of $\sigma_v$ and $\nu_c$ at constant values of $s$. The gridding function in Surfer© software was thus employed for this interpolation purpose. Nevertheless, since the software was originally designed for geographically contouring purposes, the coordinates $x$, $y$ and $z$ needed to be within a similar order of magnitude. Consequently, the grid function was performed on the data set of $\log \sigma_v$ (for $x$), $10 \cdot \nu_c$ (for $y$), and $\log s$ (for $z$). The Kriging method and other default settings were used in this function. The resulting grid data file was imported into an Excel spreadsheet and converted into $\sigma_v$, $\nu_c$ and $s$. Then the data at constant values of suction were sorted. These data sets were then used for further curve fitting and for the identification of the mathematical expression of the Loading-Collapse surface. This technique was also used for the identification of isotropic, anisotropic and $K_o$- compression lines from the triaxial tests where the mean stress was used instead of the vertical stress.

A3.2 Contouring technique for the main wetting surface
A similar technique was used to identify the main wetting surface (Figure 8.29). However, in this case, the grid function was performed on the data set of $S_r/10$ (for $x$), $\log s$ (for $y$), and $10 \cdot e$ (for $z$). For the identification of data points in Figure 8.34, the data set used was $\ln s$ (for $x$), $S_r^d$ (for $y$), and $(2/3 - \varepsilon_q / \varepsilon_y)$ (for $z$).
APPENDIX 4

This appendix gives the description of the mathematical expression for the Soil-Water Retention Curve proposed by Gitirana Jr. & Fredlund (2004), as described in Section 8.6.2.

The equation for the SWRC is that of the bimodal curve. Four hyperbolae are needed to model this curve, which is delineated by the five asymptotes that are defined by (suction, \( \psi \), degree of saturation, \( S \) (in ratio)), \((0, 1), (\psi_{b1}, 1), (\psi_{res1}, S_{res1}), (\psi_{b2}, S_b), (\psi_{res2}, S_{res2})\), and \((10^6, 0)\).

\[
S = \frac{S_1 - S_2}{1 + (\psi / \sqrt{\psi_{b1} \cdot \psi_{res1}})^2} + \frac{S_2 - S_3}{1 + (\psi / \sqrt{\psi_{res1} \cdot \psi_{b2}})^2} + \frac{S_3 - S_4}{1 + (\psi / \sqrt{\psi_{b2} \cdot \psi_{res2}})^2} + S_4
\]

where

for \( i = 1, 2, 3, 4 \)

\[
S_i = \frac{\tan \theta_i \cdot (1 + r_i^2) \cdot \ln(\psi / \psi_i^a)}{(1 - r_i^2 \cdot \tan^2 \theta_i)} + (-1)^i \times
\]

\[
\times \frac{(1 + \tan^2 \theta_i)}{(1 - r_i^2 \cdot \tan^2 \theta_i)} \cdot \left[\frac{r_i^2 \cdot \ln^2(\psi / \psi_i^a) + \frac{a^2}{2} \cdot (1 - r_i^2 \cdot \tan^2 \theta_i)}{(1 + \tan^2 \theta_i)}\right] + S_i^a
\]

\( \theta_i = -(\lambda_{i-1} + \lambda_i) / 2 \) = rotation angles of hyperbolae

\( r_i = \tan[(\lambda_{i-1} - \lambda_i) / 2] \) = tangents of the aperture angles

\( \lambda_0 = 0 \)

\( \lambda_i = \arctan[(S_i^a - S_{i+1}^a) / \ln(\psi_i^a / \psi_{i+1}^a)] \) = desaturation slopes

\( S_1^a = 1; S_2^a = S_{res1}; S_3^a = S_b; S_4^a = S_{res2}; S_5^a = 0 \)

\( \psi_{b1}^a = \psi_{b1}; \psi_{b2}^a = \psi_{res1}; \psi_{b2}^a = \psi_{b2}; \psi_{b3}^a = \psi_{res2}; \psi_{b4}^a = 10^6 \)

\( d_j = 2 \cdot \exp[1 / \ln(\psi_{j+1}^a / \psi_j^a)] \) = weight factors, \( j = 1, 2, 3. \)