The influence of soil suction on compressibility
and swelling

by

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A thesis submitted to the University of London
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Doctor of Philosophy in the Faculty of Engineering
One thing I have learned in a long life: that all our science, measured against reality, is primitive and childlike - and yet it is the most precious thing we have

Albert Einstein
Abstract

This thesis presents a study into the influence of suction on the compressibility and swelling characteristics of a broad range of reconstituted clay soils.

Soil water relationships are explored, discussing evaporation, shrinkage, swelling and the development of suction. A comprehensive review of suction measurement techniques is detailed, identifying operating principles, performance and limitations of each technique.

For the main experimental programme a new, osmotically suction controlled oedometer has been developed. The oedometer utilises the osmotic potential of a large molecular weight salt (polyethylene glycol) across a semi-permeable membrane to control matrix suctions in the sample during testing. Independent continuous measurement of the suction in the sample during testing is made using the Imperial College tensiometer.

Four materials have been assessed, Kaolin, London clay, and two expansive soils from Kenya.

A series of suction controlled stress path tests were carried out on the four materials.

In addition intrinsic and soil moisture characteristics were determined and discussed. Routine soil classification tests, mineralogy and SEM studies were also carried out.

The results are presented and discussed, and recommendations for future research made.
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The infectious enthusiasm of the researchers and academics alike was a joy. The thorough and comprehensive nature of the course, coupled with a willingness to discuss any problem or point of interest was central to the improvement in my understanding of soil mechanics.

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**Table of contents**

**Chapter 1**

**Introduction**

1.1 Introduction ......................................................... 1
1.2 Scope of research ..................................................... 2
1.3 Outline of thesis ..................................................... 3

**Chapter 2**

**Soil Water Relationships**

2.1 Introduction .......................................................... 5
2.2 Soil mineralogy ....................................................... 5
  2.2.1 Particle shape ................................................... 7
  2.2.2 Particle size .................................................... 7
2.3 Soil water systems .................................................. 8
  2.3.1 Granular soil .................................................... 8
  2.3.2 Clay soil ........................................................ 8
    2.3.2.1 Double layer theory ........................................ 8
    2.3.2.2 Osmotic effects ............................................ 9
    2.3.2.3 Clay water interaction ..................................... 10
2.4 Evaporation and the development of soil suction .................. 11
  2.4.1 Granular soils .................................................. 11
    2.4.1.1 Development of the air water interface .................. 12
    2.4.1.2 Evaporation from a granular soil ......................... 15
  2.4.2 Clay soils ..................................................... 16
    2.4.2.1 Initial pore structure ..................................... 16
    2.4.2.2 Change in pore structure .................................. 16
    2.4.2.3 Evaporation from a clay soil .............................. 17
2.5 Fabric .............................................................. 19
  2.5.1 Granular soils .................................................. 19
  2.5.2 Clay soils ..................................................... 19
  2.5.3 Study of soil fabric ........................................... 20
    2.5.3.1 Optical methods .......................................... 20
    2.5.3.2 Physical methods .......................................... 22
2.6 De-saturation ....................................................... 23
  2.6.1 Pore fluid ...................................................... 23
  2.6.2 Pore structure .................................................. 24
  2.6.3 Cavitation ...................................................... 24
  2.6.4 Adhesion ....................................................... 26
<table>
<thead>
<tr>
<th>2.6.5</th>
<th>Experimental data</th>
<th>27</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.6.6</td>
<td>Summary</td>
<td>29</td>
</tr>
<tr>
<td>2.7</td>
<td>Hysteresis</td>
<td>29</td>
</tr>
<tr>
<td>2.7.1</td>
<td>Pore fluid contact angle</td>
<td>30</td>
</tr>
<tr>
<td>2.7.2</td>
<td>Constricted pore structure</td>
<td>30</td>
</tr>
<tr>
<td>2.7.3</td>
<td>Entrapped air</td>
<td>30</td>
</tr>
<tr>
<td>2.7.4</td>
<td>Clay soils</td>
<td>31</td>
</tr>
<tr>
<td>2.7.5</td>
<td>Scanning curves</td>
<td>31</td>
</tr>
</tbody>
</table>

**Chapter 3 The influence of suction on compressibility and swelling**

<table>
<thead>
<tr>
<th>3.1</th>
<th>Introduction</th>
<th>47</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.2</td>
<td>Mechanistic behaviour</td>
<td>47</td>
</tr>
<tr>
<td>3.2.1</td>
<td>Validity of effective stress concept for partly saturated soils</td>
<td>48</td>
</tr>
<tr>
<td>3.2.2</td>
<td>Suction component influencing soil behaviour</td>
<td>50</td>
</tr>
<tr>
<td>3.3</td>
<td>Compression and swelling of soils</td>
<td>51</td>
</tr>
<tr>
<td>3.3.1</td>
<td>Saturated soils</td>
<td>51</td>
</tr>
<tr>
<td>3.3.2</td>
<td>Partly saturated soils</td>
<td>52</td>
</tr>
<tr>
<td>3.4</td>
<td>Summary</td>
<td>56</td>
</tr>
</tbody>
</table>

**Chapter 4 Soil suction and its measurement**

<table>
<thead>
<tr>
<th>4.1</th>
<th>Definitions of soil suction</th>
<th>67</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.1.1</td>
<td>Terminology</td>
<td>68</td>
</tr>
<tr>
<td>4.1.2</td>
<td>Matrix suction</td>
<td>69</td>
</tr>
<tr>
<td>4.1.3</td>
<td>Osmotic suction</td>
<td>70</td>
</tr>
<tr>
<td>4.1.4</td>
<td>Total suction</td>
<td>70</td>
</tr>
<tr>
<td>4.2</td>
<td>Influence matrix and osmotic suction have on each other</td>
<td>72</td>
</tr>
<tr>
<td>4.2.1</td>
<td>Matrix on osmotic</td>
<td>72</td>
</tr>
<tr>
<td>4.2.2</td>
<td>Osmotic on matrix</td>
<td>72</td>
</tr>
<tr>
<td>4.3</td>
<td>Suction measurement</td>
<td>72</td>
</tr>
<tr>
<td>4.3.1</td>
<td>Psychrometers</td>
<td>73</td>
</tr>
<tr>
<td>4.3.2</td>
<td>Filter paper method</td>
<td>77</td>
</tr>
<tr>
<td>4.3.3</td>
<td>Suction and pressure plate (axis translation technique)</td>
<td>81</td>
</tr>
<tr>
<td>4.3.4</td>
<td>Tensiometers</td>
<td>83</td>
</tr>
<tr>
<td>4.3.5</td>
<td>Electrical conductivity (porous blocks)</td>
<td>88</td>
</tr>
<tr>
<td>4.3.6</td>
<td>Thermal conductivity sensors</td>
<td>90</td>
</tr>
<tr>
<td>4.3.7</td>
<td>Osmotic suction measurement</td>
<td>91</td>
</tr>
</tbody>
</table>
Chapter 5  Suction controlled laboratory testing

5.1 An overview of previous work ........................................ 118
  5.1.1 Axis translation technique ........................................ 118
  5.1.2 Osmotic potential for control of soil suctions ............... 121

5.2 Osmosis ................................................................. 123
  5.2.1 Principles of osmosis ............................................. 123
  5.2.2 Mechanism of osmosis ........................................... 125
  5.2.3 Application of osmotic pressure in controlling pore water tension . 129

5.3 Experimental development .......................................... 131
  5.3.1 Current practice .................................................. 132
  5.3.2 Oedometer design number 1 ..................................... 133
  5.3.3 Oedometer design number 2 ..................................... 133
    5.3.3.1 Measurement of lateral stresses ............................ 133
    5.3.3.2 Measurement of moisture exchange ........................ 134
    5.3.3.3 Evaporation control ......................................... 139
    5.3.3.4 Circulation pumps .......................................... 144
    5.3.3.5 Semi-permeable membrane ................................ 145
    5.3.3.6 Polyethylene glycol (PEG) .................................. 147
    5.3.3.7 Experimental observations ................................. 147
  5.3.4 Additional apparatus development ................................ 151
    5.3.4.1 Underdrainage ............................................... 151
    5.3.4.2 Calibration of the osmotic potential of Polyethylene glycol (PEG) ........................................... 153

5.4 Oedometer design number 3 .......................................... 159
  5.4.1 Loading systems ................................................. 159
  5.4.2 Measurement and control systems .............................. 161

5.5 Summary ............................................................ 162

Chapter 6  Soils Properties

6.1 Introduction .......................................................... 193

6.2 Soil types, origins and mineralogy ................................ 193
  6.2.1 Kaolin ............................................................ 194
  6.2.2 London clay ...................................................... 194
  6.2.3 Bangali clay ..................................................... 194
  6.2.4 Black Cotton clay ............................................... 195

6.3 Particle size analysis ............................................... 195
### Chapter 7  
**Sample preparation and test procedures**

7.1 Sample preparation ............................................. 202
   7.1.1 Initial preparation ........................................ 202
   7.1.2 Mixing .................................................. 202
   7.1.3 One dimensional consolidation ........................... 203

7.2 Calibration .................................................... 204
   7.2.1 Instrument calibration .................................... 204
      7.2.1.1 Load cells ........................................... 205
      7.2.1.2 Pressure transducers ................................ 206
      7.2.1.3 Displacement transducers ........................... 206
   7.2.2 Compliance .............................................. 207
      7.2.2.1 Lever arm oedometer ................................. 207
      7.2.2.2 Suction controlled oedometer ....................... 207

7.3 Intrinsic compression tests ................................. 208
   7.3.1 Test procedure ........................................... 209

7.4 Soil moisture characteristic curves ........................ 209
   7.4.1 Samples .................................................. 210
   7.4.2 Test procedure ........................................... 210
      7.4.2.1 Drying curve ........................................ 210
      7.4.2.2 Wetting curve ........................................ 211

7.5 Air drying tests .............................................. 214
   7.5.1 Test procedure ........................................... 214

7.6 Suction controlled oedometer tests ........................ 215
   7.6.1 Sample preparation ....................................... 215
      7.6.1.1 Initial drying ....................................... 215
   7.6.2 Test procedure ........................................... 216
      7.6.2.1 Initial sample preparation .......................... 216
      7.6.2.2 Initial oedometer preparation ....................... 217
      7.6.2.3 Sample set up ........................................ 218
Chapter 8 Experimental results, analysis and discussion - intrinsic and soil moisture characteristics

8.1 Introduction .......................................................... 229
8.2 Intrinsic compression and swelling .................................. 230
  8.2.1 Terminology ..................................................... 230
  8.2.2 Intrinsic compression and swelling tests ....................... 231
  8.2.3 Discussion ...................................................... 233
8.3 Soil moisture characteristics ...................................... 233
  8.3.1 Filter paper calibration ........................................ 234
  8.3.2 Drying / wetting behaviour ................................... 234
    8.3.2.1 Kaolin ..................................................... 235
    8.3.2.2 London Clay ............................................... 239
    8.3.2.3 Bangali Clay .............................................. 242
    8.3.2.4 Black Cotton Clay ...................................... 245
    8.3.2.5 Discussion ................................................ 248
  8.3.3 Evaporation, shrinkage and crack propagation ............... 254
    8.3.3.1 Kaolin ..................................................... 255
    8.3.3.2 London Clay ............................................... 256
    8.3.3.3 Bangali Clay .............................................. 256
    8.3.3.4 Black Cotton Clay ...................................... 257
    8.3.3.5 Additional drying tests .................................. 258
    8.3.3.6 Discussion ................................................ 259

Chapter 9 Experimental results, analysis and discussion - suction controlled oedometer tests

9.1 Introduction .......................................................... 326
9.2 Development tests ................................................... 326
  9.2.1 Use of the osmotic potential of PEG to control matrix suction. .. 327
  9.2.2 Calibration of the osmotic potential of PEG ..................... 329
9.3 Introduction to stress paths and test details ..................... 331
  9.3.1 Stress states, stress paths and presentation .................... 331
  9.3.2 The application of Kc to suction controlled testing ............ 333
  9.3.3 Stress parameters used for presentation ........................ 335
9.4 Tests undertaken on kaolin ......................................... 335
  9.4.1 Test details .................................................... 335
  9.4.2 Comparison between suction controlled and lever arm oedometer tests 336
9.4.3 Effect of suction and applied stress on compression ............ 337
  9.4.3.1 Void ratio - suction - vertical stress relationship ......... 339
9.4.4 Representation using mean total stress ......................... 339
9.4.5 Effect of suction and applied stress on swelling ................. 341
  9.4.5.1 Discussion ........................................... 342
9.5 Tests on London, Bangali and Black Cotton Clays ............... 343
  9.5.1 Tests undertaken ...................................... 343
  9.5.2 London Clay tests .................................... 343
  9.5.3 Bangali Clay tests .................................... 344
  9.5.4 Black Cotton clay tests ................................. 345
9.6 Summary ................................................... 345

Chapter 10 Conclusions and recommendations for further research
10.1 Introduction ............................................... 387
10.2 Literature review ............................................ 387
10.3 Equipment and technique review .............................. 389
  10.3.1 Suction controlled oedometer ......................... 389
  10.3.2 Technique for obtaining wetting curve .................. 491
10.4 Experimental results ....................................... 391
  10.4.1 Intrinsic and soil moisture characteristics .............. 392
  10.4.2 Suction controlled oedometer tests ..................... 393
10.5 Future research recommendations ............................ 394

Appendix A Review of Lateral Stress Measurement in the Oedometer
A1 Introduction .................................................. 397
  A1.1 Passive - semi rigid systems ............................. 397
  A1.2 Active - compensating systems .......................... 397
A2 Passive - semi rigid systems ................................ 397
A3 Active - compensating systems ................................ 399
A4 Critique of existing systems .................................. 400
A5 Additional options ........................................... 403
A6 Systems for further consideration ............................. 404
  A6.1 A note on compensating systems ......................... 405

References ................................................... 413
Chapter 1

Introduction

1.1 Introduction

The development of soil mechanics has, historically, been based around saturated conditions where the soil is considered to be a two phase material, consisting of solid soil particles and water filled voids. In addition, with the pore water assumed to be under a positive pressure, the development of many classical soil mechanics concepts have concentrated on a special but important case.

In contrast, much of the earth's surface consists of soils where the pore water is under a negative pressure. This negative, tensile, force is known as soil moisture suction. In addition the soil is often a three phase material with the voids filled with both air and water.

Early research into partly saturated soil was often undertaken by soil scientists, studying soil water interaction, and agronomists studying the relationships between plant growth and the energy required to extract moisture from the soil. However, they were not concerned as to how negative pore pressures (suctions) would influence soil behaviour.

The Road Research Laboratory (now TRL) established a small team in 1949 to study the role of suction in soil mechanics, which marked the beginning of a long association with partly saturated soil mechanics. TRL's early involvement is not surprising as the majority of the structures they are involved with are founded on the ground some way above the water table.

Research at Imperial College into the behaviour of partly saturated soils began in the 1950's with work by Bishop on the principle of effective stress in partly saturated soils. Research into compacted clay fills, and soil water suction were also concurrent research themes during this period.

Two important conferences in the 1960's, Pore Pressure and Suction in Soils (1961), and Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas (1965) produced a number of seminal papers on the subject of soil suction and partly saturated soil behaviour.

The latter conference was promoted as an introduction to the 1st Expansive Soils Conference (1965). The Expansive Soils conferences continued and were supplemented by a number of "speciality" conferences on particular soil types, many of which had a central theme of partly saturated soil.
Historically the solution of geotechnical problems relating to partly saturated soils has centred around experimental studies of particular problems, such as heave or collapse. These studies have proved useful, but as they often relate to problem specific solutions, they have been limited in their contribution to the general understanding of partly saturated soil behaviour.

A number of "speciality" conferences have been held, each concentrating on a particular aspect of partly saturated soil behaviour. Whilst these conferences were very useful focus groups, there was a need for a general conference to bring together much common thinking.

The culmination of these soil specific conferences was the 1st International Conference on Unsaturated Soils (1995). It was intended that this conference would bring together the research from the specific conferences (regional soils, tropical soils, expansive soils, fills and embankments) under a more general theme of partly saturated soils. In their state of the art report at this conference Delage and Graham (1995) introduced partly saturated soils as the general case, with saturated soil behaviour as a sub-set.

The separation between saturated and partly saturated soils is often indistinct and transient. Fig 1.1 shows a simplified ground profile. The complexity is the pore pressure regime. Possible limiting conditions are indicated, and it can be seen that a considerable overlap between saturated and partly saturated soils could exist. Most soils will remain saturated to an extent, above the water table, through capillarity. The extent of this saturated zone, where the pore water pressure is negative, can be considerable in clay soils. Above this transitional zone may be a phase which can be considered partly saturated. The final phase, up to ground level, may experience the greatest change in pore pressures. It is furthest from the ground water table, and as such could have a low degree of saturation, and very high suctions. However this zone is also the upper boundary and is most influenced by precipitation and infiltration. Under certain conditions the pore pressures could become positive in the uppermost layers of the ground. Fig 1.1 demonstrates both the complexity of a pore pressure regime and the interaction between saturated and partly saturated soils.

In recent years there have been a number of advances in the study of partly saturated soils. Experimental techniques have improved dramatically. One of the major advances has been the development of a tensiometer capable of measuring high suctions simply, quickly and directly (Ridley & Burland, 1996). Theoretical understanding of the interaction between soil and water is improving, allowing conceptual and numerical models of partly saturated soils to be developed.

1.2 Scope of research

As noted above, the tensiometer developed by Ridley and Burland (1996) represents a major step in the study of the influence of soil suction. The ability to measure suctions directly under atmospheric conditions puts a greater emphasis on developing other experimental techniques such that they also operate under
atmospheric conditions.

A key aspect of this research project was to develop a suction controlled oedometer that could operate under atmospheric conditions. Suction is controlled at the base of the sample using an osmotic technique, and measured independently at the top of the sample using the Imperial College tensiometer. A new simple method has been devised for measuring moisture content changes in the sample during testing. Moisture content, applied stress, suction and sample compression are monitored continuously throughout a test.

A number of tests have been carried out to study the influence of suction on compressibility and swelling, concentrating on the intrinsic behaviour of a number of clay soils. Furthermore the emphasis is on soils possessing suctions irrespective of whether they are saturated or not, so as to explore the transition between saturated and partly saturated behaviour. This helps to develop an understanding of the influence of suction on saturated soil behaviour, and provide a link between saturated and partly saturated behaviour.

Further experimental work is presented on a wide range of clay soils, exploring the effects of soil type on soil moisture characteristics, evaporation and crack propagation during drying.

In addition, reviews of soil water interaction, the mechanistic behaviour of saturated and partly saturated soils, and suction measurement are given.

1.3 Outline of Thesis

Chapter 2
Reviews the interaction of both granular and clay soils with water. Clay mineralogy is discussed at the outset, before moving on to explore the interaction between soil and moisture. Evaporation from soils and mechanisms for desaturation are also described. Finally, the effects of hysteresis are discussed.

Chapter 3
Discusses the influence of suction on compressibility and swelling from a mechanistic point of view. The various suction components are detailed, and both saturated and partly saturated soil behaviour is reviewed.

Chapter 4
Provides a comprehensive review of suction measurement. Current techniques and some historical applications are described. The measurement techniques employed in this research are discussed in detail.

Chapter 5
Covers the main experimental development programme. In addition to the practical aspects of the development of an osmotically suction controlled oedometer, a theoretical review of osmotic principles, and the applicability of this technique to soil testing is
discussed.

Chapter 6 Describes the soil properties; their origins, mineralogy and particle size distribution.

Chapter 7 Gives details of the techniques for sample preparation, calibration of instrumentation and test procedures.

Chapter 8 Presents the intrinsic behaviour and soil moisture characteristics of the soils studied. These are discussed in detail.

Chapter 9 Presents the suction controlled oedometer test results. Analysis, discussion and a summary of the main findings are given.

Chapter 10 Summarises the main conclusions from the research project. In addition, recommendations for further relevant research are given.

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**Figure 1.1** Possible pore pressure profiles and fully and partly saturated zones.
Chapter 2

Soil water relationships

2.1 INTRODUCTION

Soils consist of three distinct phases, solid, liquid and gas. The extent to which each is present influences the current state of the soil, the properties it possesses and its likely engineering behaviour.

The solid phase may consist of inert granular material, clay minerals and organic matter in varying quantities. These constituents are the building blocks of the soil and will define the basic characteristics and limits of its engineering behaviour.

The liquid phase is commonly water containing dissolved salts. The extent to which moisture is present, whether it satisfies the demands of the soil, and the pressure which it exerts, have a profound effect on the engineering behaviour.

The gas phase can be considered to be air for the majority of situations. Its presence has a profound effect on soil response and affects the application of many fundamental soil mechanics theories.

Initially we will discuss soil mineralogy in isolation. Expanding on this, the relationship and interaction with water will be explored for both granular and clay soils. Drying, wetting and the process of evaporation will also be considered.

2.2 SOIL MINERALOGY

There are two distinct soil groups; granular and clay soils. Granular materials (sands and silts), display interesting engineering behaviour but can, for these purposes, be considered relatively inert. They have negligible electrical surface charge and, in isolation, little attraction for moisture (ignoring the capillarity exhibited within a granular mass). Although of irregular shape they are often considered as spherical for the basis of theoretical interpretation.

Clay particles are somewhat different. They are defined as particles whose mean effective size is less than 0.002 mm. In mineralogical terms, clay is defined by its:
i) Small particle size 
ii) Net negative electrical charge 
iii) Plasticity when mixed with water 
iv) High weathering resistance 

Clay minerals are silicates, with negatively charged atoms (ions) surrounding a positively charged silicon, aluminum or magnesium ion.

The two basic units composing the majority of clay mineral structures are the silica tetrahedron and the alumina octahedron, fig 2.1(a)

Sheets of these minerals are bonded into layers. The stacking of these layers, bonding between them, and substitution of other ions for the aluminium and silicon ions is what discriminates between the various mineral groups.

Substitution of ions within the clay lattice, and edge and surface imperfections leads to an overall negative charge being associated with clay particles. Positively charged ions (cations) from the pore water are attracted to the surface of the clay in order to maintain neutrality. These cations can be replaced by other available cations and as such are termed "exchangeable". The total number of exchangeable cations is generally expressed as the Cation Exchange Capacity, CEC, and is measured in milliequivalents per 100 g of clay.

The most common mineral groups are

1) Kaolin group (1:1 lattice) 
2) Mica group (2:1 lattice) 
3) Smectite group (expanding 2:1 lattice) 

1) Kaolinite group 

A simple, repeating layer arrangement of one silica and one alumina sheet, fig 2.1(b). The individual layers are held together through hydrogen bonding. Hydrogen atoms bond the oxygen from one face of a silica sheet to the oxygen from one face of an alumina sheet. The repeating layers are held tightly together in this way.

The repetition of this process results in the stacking of clay mineral layers into the clay crystals with up to 100 layers bound into a single clay particle
There is little opportunity for ion or water penetration between the layers and hence the kaolinite particles are relatively inert, with their outer surfaces determining their colloidal properties. The CEC is less than 10 me/100 g.

2) Mica group (Illite)

A 2:1 lattice mineral consisting of a single alumina sheet and two silica sheets for each layer, fig 2.1(c). Individual layers are bonded together by potassium ions. More active than the kaolinite these clay minerals have a CEC of 20 - 40 me/100 g.

3) Smectite group (Montmorillonite)

Although similar to the Mica group in that they are of the 2:1 lattice structure, the montmorillonite clay minerals have one fundamental difference. There is no potassium bonding within the lattice, hence water enters readily between the individual layers causing the lattice structure itself to expand, fig 2.1(d). The colloidal activity is therefore dictated by the inter layer attraction for water as well as the outer surface. As such the ability to swell is marked, and the CEC is between 80 and 150 me/100 g.

2.2.1 Particle shape

Although for simplicity, many concepts have been developed by modelling clay particles as spherical in shape, they are in fact plate or disc shaped. This is borne out through both consideration of the sheet lattice structure and micro-graphical investigation.

2.2.2 Particle size

We can gain an understanding of the relative differences in particle size by considering the specific surface area of each of the clay minerals, table 2.1

From this table we can see two important points. Firstly that there are significant differences in the size of the various clay minerals. When we consider that the ability of the mineral to absorb water and hence swell is directly related to the available surface area we can identify the most active minerals. Secondly, this is further exacerbated by the ability of water to be absorbed between the layers of a montmorillonite mineral allowing the mineral structure itself to expand and contract.
2.3 SOIL WATER SYSTEMS

Clay-water interaction is a complex field and has been the subject of much research. It is not proposed to present here a comprehensive review of the published material, but a concise summary of the options and opinions in this field.

2.3.1 Granular soil

Granular soil / water interaction is limited to the surface wetting and pore water tension within the soil mass. The net attraction for water is a direct result of capillarity within the pore structure. This concept, in the form of matrix suction is discussed more fully in section 4.1.2.

2.3.2 Clay soil

We have seen that the clay particles have a net negative charge which is balanced by the positive ions in the pore water. There are a number of theories relating to clay-water interaction, none of which describe fully the behaviour of the material. However there is a general acceptance of the double layer theory as proposed by Gouy (1910) and revised by Bolt (1956).

2.3.2.1 Double layer theory

It can be said that the water at, or close to the clay surface differs from that at some distance. There are three layers of water associated with the clay surface; adsorbed water, diffuse layer (or absorbed) water and free water. These are indicated diagrammatically in fig 2.2.

'Adsorbed Water

This water is closest to, and in intimate contact with the clay surface. The attractive forces acting on the adsorbed water come from two sources:

i) The negative surface charge of the clay surface. This attracts positive ions in the pore fluid to achieve neutrality.

ii) Hydrogen bonding. In this instance the bond is between the oxygen on the clay surface and the hydrogen in the pore water.

This water is often termed structural water, (i.e. it is an integral, structural part of the clay particle). It is not removed by oven drying. The hydrogen bond diminishes rapidly with distance from the surface, and although it is not clear, the adsorbed layer is thought to be no more than a few molecules thick.
Diffuse (absorbed) Water

Beyond the strongly bound adsorbed water, the effects of the cation exchange principle continue. Just as it is not clear where the distinction between the adsorbed and absorbed water layers lies, the point at which the absorbed and free water coincide is not precisely known. It is known however that the density of cations decreases with distance from the clay surface and that the greater the CEC value the greater the depth or extent of this diffuse layer. This is accentuated when a high CEC value coincides with a small clay particle. Furthermore, when the ion valency is low the diffuse layer will tend be less extensive, because the number of ions required to maintain neutrality is greater.

The exchangeable cations are as their name suggests, exchangeable. They cannot be removed, only replaced, and can best be considered as restricted in a two dimensional plane, free to move parallel to the clay surface but not away from it. Replacement can occur with ions of equal valence, or by two ions of half the valency of the original.

The properties of the double layer water have been investigated and are, as one would expect, different from those of the free water. Densities of up to 1.4 g/cm³ have been measured at the clay mineral surface, reducing to 0.97 in the vicinity of the adsorbed / absorbed boundary rising to 1 g/cm³ in the free water. The viscosity can increase dramatically close to the surface, (by up to two orders of magnitude, Yong and Warkentin, 1975). These significant changes in properties of the water will have an effect not only on the permeability and compressibility of the material, but may also influence the development of suction within the clay mass.

Free water

This is any additional water which is beyond the influences of the adsorbed and absorbed water.

2.3.2.2 Osmotic effects

Despite the indistinct horizons between the three "zones" of water related to clay soils, it is likely that an osmotic effect is generated, due to the ion distribution. Considering that the boundaries are unclear the effect probably occurs as an osmotic gradient between the outer free water and the bonded, adsorbed water. The concept presented by Marinho (1994) of definite osmotic potentials occurring across the various boundaries is unlikely in view of the dispersed ion distribution.
2.3.2.3 Clay-water interaction

Initially we will consider the interaction of clay with water on an idealised basis. Under these conditions shrinkage and swelling will be similar and wholly reversible processes. Once we have outlined a simplified theoretical base we can introduce additional influences such as soil structure, particle interaction and hysteresis, allowing us to develop a more realistic model.

Clay particles can interact through either intimate contact, or through the clay-water. There are both repulsive and attractive forces associated with clay particle interaction, the relative strengths of which vary depending on the proximity of the particles themselves.

If we commence with the clay soil containing a minimum amount of water such that the particles are in intimate contact, we can consider the forces which cause and restrict the repulsion of particles from each other (swelling).

Firstly, there is elastic unloading and rebound of the particles themselves. Clay particles can be considered incompressible for most purposes, hence this is likely to be a secondary effect, although flexing of the particles may take place.

We noted earlier that water molecules adsorbed directly onto the surface of clay particles can generally be considered as part of the clay structure. If this is the only water available, the soil is, to all intents and purposes, dry. Whilst the forces operating at particle level under these conditions have not been quantified, there must be an overall attraction for moisture or a dry soil would be unable to wet up. This affinity for moisture results from a combination of a moisture deficiency in the absorbed layer, osmotic attraction of the existing structural water, and capillarity within the pore structure.

From the initially dry condition, the addition of water will allow the absorbed (diffuse) layers to form around each particle. In montmorillonite clays it must also be remembered that water is adsorbed on an inter-layer basis, causing expansion of the lattice structure.

At distances of less than 15 Å (1.5 nm) a net attraction will exist between two adjacent particles. This (excluding cementation) will be a result of two phenomenon:

i) London - Van der Waals forces. Short range forces which rapidly diminish as the inter-particle distance increases.

ii) The existence of a single absorbed layer around both particles. The exchangeable cations will be uniformly distributed across this common zone.
Whilst clay particle forces at this stage are attractive, an overall expansion of the clay mass will continue, due to the affinity of the soil for moisture through capillarity and osmotic effects. The theoretical stress generated in a fluid meniscus at the interface of two surfaces 15Å apart is of the order of 200 MPa!

With the addition of more water, individual diffuse layers will form. They will now repel each other as both attempt to attract further water to satisfy their respective ion deficiencies.

The extent of swelling will be determined by a number of factors relating to both the clay particle and the pore fluid. Table 2.2 highlights some of the key components of clay-water interaction. The higher the relative value the more active the mineral. We can see that the higher values occur in the same minerals causing clear separation in behaviour between the mineral groups.

2.4 EVAPORATION AND THE DEVELOPMENT OF SOIL SUCTION

The definition of soil suction and its measurement is discussed in detail in chapter 4. Here we will explore the development of suction in both granular and clay soils.

Tension within the pore fluid will develop and increase as moisture is lost from the soil. The principle mechanism for loss of moisture is evaporation, although pore water tension could develop as a result of a particular flow regime within the soil mass.

If the equivalent suction generated by the atmosphere (measured in terms of relative humidity) is greater than that prevailing in the soil, water molecules will be drawn from the air-water interface, changing state from liquid to vapour in the process. This will continue until either all the moisture - structural water excepted - evaporates from the soil, or the total suction in the soil balances that generated by the atmosphere.

The rate of evaporation will be controlled by the suction gradient between soil and atmosphere. During drying the suction in the soil could, under extreme conditions, change by several MPa. However the equivalent suction generated in the laboratory is likely to be significantly higher (70% RH ≈ 50 MPa). Therefore it is likely that the atmospheric conditions will be the dominant, and hence controlling parameter.

We will consider separately the development of suction for both granular and clay soils.

2.4.1 Granular soils

The particles can be considered incompressible due to the effects of the suction alone, and can be
considered inert, with negligible attraction for water.

Some shrinkage may occur during drying, due to slight re-orientation of the particles, however in general moisture loss will not be associated with volume change.

As moisture is lost, through evaporation, the air-water interface of the pore water retreats into the soil pores, becoming curved in the process. The surface tension at the interface increases as the radius of curvature reduces (see section 4.1.2). The curved surface is indicative of the existence of a pressure differential across the interface. The convex side of the interface registering a negative gauge pressure (suction).

2.4.1.1 Development of the air-water interface

A widely held conceptual model of the interaction between soil and water is that depicted by the capillary model (Buckingham, 1907), fig 2.3.

It is known that the water will rise up the glass tube due to tensile forces at the air-water interface. Consider the forces acting on the meniscus which forms the air-water interface. There is a net downward force comprising that due to atmospheric pressure on the meniscus and that due to the height of the meniscus above the pool of water:

\[ (U_a \pi r^2) + (\pi r^2 \rho_w g) \]

where:
- \( r \) = radius of capillary tube
- \( h \) = height of water column
- \( \rho_w \) = density of water
- \( g \) = acceleration due to gravity
- \( U_a \) = ambient air pressure

This downward force is resisted under equilibrium conditions by a net upward force related to the tension \( \tau \) at the air-water interface:

\[ 2\pi r \tau \cos \alpha \]

where
- \( \tau \) = surface tension
- \( \alpha \) = angle of contact between the water and the glass surface
hence under equilibrium conditions:

\[
(u_n \pi r^2) + (\pi r^2 h \rho_w g) = 2 \pi r \tau \cos \alpha
\]

\[
\pi r^2 [(u_n) + (h \rho_w g)] = \pi r (2 \tau \cos \alpha)
\]

\[
r (u_n - u_w) = 2 \tau \cos \alpha \quad 2.3
\]

Assuming perfect wetting, \( \alpha = 0 \), hence equation 3.3 above can be reduced to:

\[
(u_n - u_w) = \frac{2 \tau}{r} \quad 2.4
\]

(noting that \( u_w \) will be a negative pressure)

Extending this concept, let us now consider the air-water interface between two discrete particles. Fig 2.4 shows the air-water interface between two discrete particles. There are two distinct curvatures to this interface, in planes perpendicular to each other. Firstly there is the curvature depicted by radius \( r_1 \). The concave side is exposed to the atmosphere, suggesting a higher pressure outside the interface. On a plane perpendicular to this is a second curvature, with radius \( r_2 \), whose convex surface is exposed suggesting a lower pressure on the outside of the interface. Clearly however, the state of tension in the body of pore fluid is described by a single parameter only. The sum of the effects of both radii must be taken into account when defining the equilibrium state of pressure in the pore fluid. In this instance there are two radii resulting in a "saddle" shaped interface.

The form of the equation describing this overall tensile force is similar to that in equation 2.4 above, except that it is expanded to account for the two radii acting at the air-water interface:

\[
(u_n - u_w) = \tau \left( \frac{1}{r_1} - \frac{1}{r_2} \right) \quad 2.5
\]

where

\[
(u_n - u_w) = \text{pore fluid pressure}
\]

\[
\tau = \text{Surface tension of the fluid}
\]

\[
r_1 = \text{Radius of curvature in one plane}
\]

\[
r_2 = \text{Radius of curvature in plane perpendicular to } r_1
\]

The actual soil particle array will be far more complex than these idealisations, hence the shape of the air - water interface will also be of a complex form.

When the soil is partly saturated the pore fluid will exist as both discrete interfaces between particles and
as more complex interfaces scribing a conglomerate of particles. The extent of each type of interface will depend on the tension in the fluid and the pore structure.

We can see that the application of a simple hypothesis, based on uniform capillary-like pores is a major departure from the real situation.

Childs (1969) however, draws the readers' attention to the fact that a similar simplifying assumption is made in the application of Stokes law for determining the particle size of a clay sample. Stokes law is applied assuming the particles are spherical, although it is clearly known that this is not the case. The determined particle size is classified as the effective diameter and it is suggested that the calculated radius equating to a particular soil model be termed the effective radius. This will allow us to explore the relationship between particle size and suction in a purely qualitative way.

Let us consider further the simplified two dimensional case with discrete air-water interfaces as shown in fig 2.4. We have seen that for any given condition there will be two distinct radii acting \( r_1 \) & \( r_2 \). The volume occupied by these can be calculated and the equivalent volumetric moisture content determined. Using granular soils with particle diameters of 2, 6, 20 and 60 \( \mu \)m (equivalent to a range from fine silt to sand) calculations were made of the relationship between volumetric moisture content and suction. The results are shown in fig 2.5. We can see a trend of increasing sensitivity to changes in moisture content with decrease in particle size.

There are a number of simplifying assumptions made in this model:

1) The particles are spherical and inert
2) The model is two dimensional only
3) The packing is uniform and close
4) There is no volume change of the structure during moisture loss

Comparing the trend with experimental results in granular materials is difficult as there are very little data available. The lack of cohesion in such materials makes the determination of a moisture characteristic curve difficult. Krynine (1948) published data in which capillary rise was plotted against water content for materials from silts to coarse sands, fig 2.6. These data confirm the trend of a smaller effective pore sized material being more sensitive, in terms of suction, to moisture loss.

Whilst containing a number of significant simplifying assumptions, the hypothesis that the particle size has a significant influence on the sensitivity of a material to moisture content changes, is reinforced by the limited experimental data available.
2.4.1.2 Evaporation from granular soil

Moisture is lost from the surface of a soil through evaporation from the air-water interface. The moisture changes from liquid to vapour during this process. (see section 4.1)

As moisture is lost from the soil two phenomena are observed, firstly the soil volume reduces and secondly the air-water interface retreats into the soil structure. For a granular soil there will be little if any volume reduction as moisture is lost. The particles do not absorb water, and hence do not swell or shrink with change in water content.

Evaporation will, however, cause the air-water interface to retreat into the soil structure. Fig 2.7 depicts the retreating and subsequent advancing of the interface.

As the air-water interface retreats below the surface of the soil, tension in the pore fluid will develop as menisci form between the soil particles. With continued moisture loss the radius of curvature of the menisci will reduce causing the interface to retreat into smaller pore openings (a corresponding increase in pore fluid tension will be apparent). This will continue until a minimum pore size is reached, point 2 on fig 2.7. Any further moisture loss will cause the menisci to move beyond this constriction. This will result in a reduction in suction, as the radius of curvature increases to accommodate the wider pore structure. In practice a period of instability will occur as a more significant amount of moisture is lost until a pore capable of supporting the required menisci is found. This hypothesis has been attributed to the observed stepwise change in the moisture content at a given suction, Haines (1923).

When the menisci move into the soil from the surface (or form within the soil mass), fundamental changes in the soil behaviour occur. It signals the onset of desaturation of the soil mass, where the volume of moisture lost is no longer equal to the volume change of the soil. The soil becomes a three phase material with the voids filled with both air and water. The pore fluid is now compressible and the principle of effective stress becomes invalid. This is discussed further in chapter 3.

As the moisture continues to be lost, a number of discrete menisci form at grain contacts throughout the soil mass. The loss of moisture will continue until the total suction of the pore water equals the total suction generated by the atmosphere, or all moisture is lost.

If, prior to this state, the suction were reduced, say from point 6 on fig 2.7 moisture would begin to return to the sample, menisci curvature would reduce and air would gradually be replaced with moisture.

This could continue, with the radius of curvature of the menisci increasing, denoting a reduction in suction, until point 5 was reached. From this point the radius would need to reduce once again as the pore space is reducing, equating to an increase in suction! In practice this would not occur, rather a reversal of the
instability experienced earlier would be seen with a more significant intake of moisture occurring at that particular suction, until a suitable pore structure could be bridged.

2.4.2 Clay soils

We have discussed the composition of the clay minerals, their relationship with water and the constitution of that water in close proximity to the particles themselves. Now let us consider the drying and wetting of a clay soil mass.

The nature of clay particles means that the loss of moisture is not accompanied by a simultaneous retraction of the air-water interface into the body of the soil in the same fashion as for a granular material. The key difference is the ability of clay to attract and retain moisture by processes other than capillarity. Considerable volume change and consequential pore water tension can result from loss of moisture without desaturation occurring.

With evaporation occurring from the exposed surfaces, volumetric shrinkage will be equal to the volume of moisture lost. This coupled shrinkage will continue until the menisci radius can no longer be supported by the pore structure. From this point desaturation will occur. In order to define a model in the same fashion as we have for granular material we must first determine the effective pore size.

There are a number of factors influencing the effective pore size at which desaturation will occur. These act in addition to those outlined for granular soils.

2.4.2.1 Initial pore structure

Fig 2.8 shows a scanning electron micrograph of a lightly consolidated kaolin slurry. We can see clearly the plate-like shape and complex structural arrangement of clay particles. This is far removed from the idealised soil used for theoretical studies, and numerical or visual definition of effective pore sizes is a complex proposition.

2.4.2.2 Change in pore structure

As drying takes place, the pore structure of a clay soil will change. We have seen how clay particles react with water in isolation. Swelling and shrinkage will occur readily with a change in moisture content causing the pore size to vary. Factors which affect the extent of the change in pore structure, and influence the pore water can be summarised as follows:
i) Mineralogy.

ii) Extent of the double layer water. This will not only influence the attraction and repulsion of the individual clay particles but can also affect the properties of the pore fluid and its ability to sustain a tension i.e. tensile strength and adhesion of water in the double layer.

iii) Pore fluid chemistry. This will directly affect the extent of the double layer water.

iv) Fabric of the soil mass. This effect could range from insignificant to dominant.

Summary

We see that the initial pore structure is both varied and complex. We also note that the structure is not fixed but varies with moisture content.

The definition of an effective pore size may be possible experimentally through mercury porosimetry studies. However the results from such an investigation relate to a particular moisture content condition only. The effective pore size will vary and would need to be defined across the expected range of moisture contents through both wetting and drying cycles if necessary, to provide useful data.

2.4.2.3 Evaporation from a clay soil

Whilst saturated, moisture will be lost from the surface of a clay soil in the same way as for a granular soil, through evaporation from the exposed surface. The nature of clay minerals will however result in changes in both the soil structure and pore fluid characteristics as this moisture is lost.

The attractive and repulsive forces between clay particles and their relative orientation to each other vary throughout the sample. The extent and influence of these factors will change with loss of moisture, as particles move closer together and undergo reorientation.

During the initial stages of moisture loss, shrinkage of the sample will ensure it remains saturated. A point will be reached, where the stress required to cause further volume reduction is in excess of that which can be achieved through the tension of the air-water interface for a given pore opening. Desaturation will occur and the menisci will begin to retreat into the soil mass. Volume change will not cease, but the rate of volume change will reduce and become uncoupled from the volume of moisture lost.

In a granular soil, moisture is lost through the air-water interface only. In clay soils we also need to consider the moisture retained within the diffuse layers. Some moisture will remain after the menisci have retreated into the soil mass. We noted earlier that this moisture can be considered free to move in two dimensions, with a degree of fixity in planes perpendicular to the clay surface. Whilst in principle evaporation from these surfaces may be restricted, this water layer could operate as a transport mechanism.
Moisture could be fed to the absorbed layer from within the soil mass allowing an equivalent evaporation from the outer surface. This would allow moisture to move in a film around the clay surface with evaporation occurring from the outermost exposed surfaces. As such, evaporation could continue to occur from surfaces other than the menisci between particles, and at, or close to the exposed surface until quite low (residual) moisture contents were reached.

In granular soils the smaller the particle size, the greater the increase in suction for a unit reduction in moisture, fig 2.5. In clay soils the opposite effect is seen, with the smaller clay particles being less responsive to a change in moisture content.

Fig 2.9 shows the relationship between void ratio and applied pressure for a range of soil types. Whilst saturated Blight (1965), fig 2.10, showed that there is a reasonable degree of comparability between mechanical compression and that induced by suction, thus the applied mechanical pressure can be taken as approximately equivalent to suction. It can be seen that the smaller clay particles (montmorillonite) are the least sensitive, (in terms of a change of suction) to a unit reduction in volume (moisture). The trend observed for granular soils is not applicable to clay soils. The activity of the clay minerals and their interaction with water is a dominant force in the soil moisture behaviour.

The soil moisture characteristic curve of a soil is a term used to describe the relationship between moisture content and suction of a soil. More generally the soil moisture characteristics of a soil describes the relationship between any soil parameter and either the moisture content or suction. Marinho (1994) determined soil moisture characteristic curves for a wide range of silt-clay mixtures. The results are shown in fig 2.11. Although the trend is slight, it can be observed that increasing the silt content decreases sensitivity to moisture content changes. This is contrary to the hypothesis for granular material as the effective pore size would be increasing. However it is more a reflection of a move from clay to silt as the controlling influence in behaviour.

Summary

We have identified fundamentally differing soil moisture characteristics for granular and clay soils. For granular soils a reduction in particle size results in an increase in suction for a unit change in moisture content. This is supported by simple theory and limited experimental data.

We note the opposite effect in clay soils. Here, for a unit change in moisture content, a greater change in suction is observed when the particle size is increased. The clay mineralogy would seem to be a dominant factor in this relationship.

Natural soils may consist of more than one type of clay mineral together with varying quantities of granular material. Prediction of the soil moisture characteristic is likely to prove a complex and in many
instances onerous exercise.

However having developed an understanding of the relative behaviour of the various clay minerals and granular soils in isolation we can make some tentative judgements of the type of response and perhaps the limits of likely behaviour.

2.5 SOIL FABRIC

The terms structure and fabric are used to describe soil particle arrangements and interactions. The terms are often interchanged, which can lead to confusion.

Mitchell (1992) suggests the FABRIC is used to describe the arrangement of particles, particle groups and pore spaces in a soil, and STRUCTURE is used to describe the combined effects fabric, composition and bonding or cementation. We will consider fabric and structure in general terms only. As such, it is not proposed to label the individual aspects under these headings.

2.5.1 Granular soils

The fabric of an inert, granular material is centred on the packing of the soil grains. This is heavily influenced by the gradation and shape. Oda (1985) identified three components of the fabric of a granular soil:

i) Grain shape and orientation
ii) Void shape and orientation
iii) Orientation and distribution of grain contact normals.

The loss of moisture from a granular soil will have a stabilising effect as menisci form at the grain contacts. There is a limiting condition where the increase in tension is offset by the decrease in contact area. With little volume change associated with desaturation the overall effects on the fabric of changes in moisture content are minimal. The definition of pore structure for a granular soil will largely be unaffected by the state of the pore fluid. Therefore a single determination could describe the fabric adequately.

2.5.2 Clay soils

Apart from the clay-water interaction, bonding between the clay particles will occur through cementation
via any organic and inorganic material present with each "bond" linking a number of clay particles together. Oxides and carbonates form the most common inorganic material. Organic material can additionally be held through hydrogen bonding, in a similar way to the adsorbed water layer.

Clay soil is influenced by a number of clay-clay and clay-water interactions (see 2.3.2). The extent of these attractive and repulsive forces is dependent on the mineralogy of the clay particles, their proximity and orientation relative to one another, and the availability and composition of the pore fluid.

When considering soil moisture characteristics, the macro fabric, i.e. void structure, is one of the controlling influences. We have seen how the void structure is dominated by, and dependent on, the moisture condition. Thus the pore structure must be determined across the range of moisture conditions under consideration. This may also require such determinations to be carried out on both wetting and drying cycles.

2.5.3 Study of soil fabric

The gradation and extent of pore sizes in a soil will be one of the fundamental factors in defining its moisture characteristics. We will concern ourselves with methods for the determination of the pore size distribution only, at this stage.

There are two basic approaches to determination of the pore size distribution:

i) Optical methods
ii) Physical methods

2.5.3.1 Optical methods

Optical and electron microscopy are established methods for soil fabric studies. Modern software driven techniques can define grain size and spacing automatically, although this can be a lengthy process. The results are in two-dimensional, planar form only. Their extension to a volumetric definition is not straightforward, and may be a qualitative determination.

Smart and Tovey (1982) present a comprehensive review of electron microscopy, and the reader is referred to this work.
Sample preparation - optical

Thin sections for optical microscopy are prepared by carbowax impregnation, Morgenstern and Tchalenko (1967). In this process the sample is submerged in a bath of carbowax (polyethylene glycol) in a warm oven (65° C) for at least one week. During this time the carbowax replaces the pore water. When cooled the carbowax solidifies, retaining the original fabric. Thin sections can be prepared from this "fixed" structure.

Sample preparation - SEM

Samples for electron microscopy need all pore fluid to be removed, not replaced.

In granular soils they may lose all integrity and collapse when dry. Juang and Holtz (1986) overcame this by adding a small amount of resin during sample preparation. Obviously this is only applicable for micrographic studies.

In clay soils, removal of water will cause shrinkage, and therefore a change in pore size. Additionally samples with high water content may suffer cracking during the drying process.

In order to avoid these problems, critical point and freeze drying are often employed. Both techniques remove the pore fluid whilst retaining the original structure.

Critical point drying involves increasing both the pressure and temperature of the sample until it passes the critical point (374° C, 22.5 MPa) - where the liquid and gas phases become indistinguishable. From here the pressure can be reduced allowing the pore fluid to move from the liquid to gas phase directly. The temperature can then be reduced and the pore fluid drawn off in the vapour state by vacuum, fig 2.12

Freeze drying involves rapidly freezing the soil sample to a very low temperature. The rapid freezing prevents ice crystal formation, and hence expansion of the fluid which might cause cracking. Following freezing, the pressure is reduced until the fluid is in a vapour state and again it is drawn off under vacuum, fig 2.12

These techniques leave a dry clay soil sample which contains the same structure as the soil did in its original moisture condition. A fresh face is then exposed by fracturing the sample and a conductive coating is placed (normally gold) on the new surface. This allows the scanning electron microscope to be utilised.
2.5.3.2 Physical methods

There are two principle methods for direct determination of pore size distribution, capillary condensation and mercury intrusion.

Capillary condensation

Capillary condensation is based on the interpretation of adsorption and desorption isotherms of a vapour, commonly nitrogen. These principles are discussed by Wall (1981), and their application, by Al-Mukhtar (1995). It has a practical drawback in that the maximum pore size that can be determined is 0.1 μm and is therefore of limited value in many instances.

Mercury porosimetry

Mercury porosimetry has an operating range from 0.001 μm to >10 μm. Its principles are based on the fact that a non-wetting liquid (one whose wetting angle, θ, is greater than 90 °) will not enter a pore unless a pressure is applied to the fluid. Mercury is one such fluid. Diamond (1970) discussed mercury porosimetry in detail. He covered both the theoretical and practical considerations of the technique with respect to the study of clay soils. More recently Ahmed, (1974) and Delage (1984) have employed the technique in pore size studies.

Measurements are made of the applied pressure and intruded volume of mercury. This will give an indication of the volume of pores of a particular size. The capillary pressure equation can be used to define the relevant pore size:

\[ d = \frac{-4 \tau \cos \theta}{P} \]  

where:

- \( \tau \) = surface tension (4.84 x 10^{-4} N/mm) (values for mercury)
- \( \theta \) = surface wetting angle (130-140°)
- \( P \) = applied pressure
- \( d \) = equivalent pore diameter

There are a number of points to note with the application and interpretation of this data:

i) Sample preparation. (see section 2.5.3.1)

ii) The equipment must be capable of penetrating the smallest pores.
The pressures required to intrude mercury into clay soil pores can be very high. Delage et al (1984) used equipment with a working capacity of 150 MPa.

iii) Isolated pores will not be measured

iv) Constricted pores can influence the data

Constricted pores can be defined as a complex of larger pores with a smaller entrance pore. The intruded volume will be attributed to the entrance pore and not the various pore sizes contained within the constricted mass. This phenomenon is again similar to that experienced during wetting and drying cycles of a soil (see 2.4.1.2). Thus the initial intrusion will define the boundary "wetting curve" only.

The use of both an intrusion and extrusion gives both an indication of the overall constricted pore mass (and its entrance pore size), as well as a clearer indication of the pore shapes within the constricted mass, (Pellirin, 1979)

2.6 DESATURATION

We have discussed soil moisture relationships. In more detail we have looked at a number of factors which influence the soil moisture characteristics and also explored mechanisms for evaporation. We will now consider the process of desaturation

This can be defined as the point at which the sample-volume-change and moisture-volume-change become uncoupled. It is more often quoted as the point of desaturation, or air entry value.

This definition leads us to one of the fundamental points for discussion, is the air drawn into the sample or does it form within the soil mass?

There are several points which need to be addressed in defining the process in operation. These relate to the pore fluid, pore structure and cavitation of liquids. We will address them in this order.

2.6.1 Pore fluid

The air-water interface forms the boundary between liquid and vapour states. It has been termed the contractile skin by Fredlund and Rahardjo (1993). This surface possesses properties which allow it to
sustain tension under certain conditions, maintaining equilibrium between the liquid and vapour states.

There has been some debate that this surface can be viewed as a separate, additional phase, raising the question of whether tension exists in the body of the pore fluid, or is contained within the contractile skin. If the pore fluid is not under tension, then breakdown cannot occur within the fluid, and the mechanism of desaturation can only be one of air entry from the exposed surface.

However whilst the contractile skin possess some interesting properties it is merely a boundary between liquid and vapour states. The curved shape of the menisci is due to a tensile force acting on the convex side of this surface.

Consider the menisci in fig 2.4. Let us assume that the tension in the pore fluid and the suction generated by the atmosphere surrounding the soil are in equilibrium. There are two menisci between these two soil particles, equal in magnitude but acting in opposite directions. For equilibrium conditions to exist there must be a link between these two menisci, and the pore fluid is the only mechanism available. The tension described by the radius of curvature of the menisci is equivalent to the tension in the pore fluid.

2.6.2 Pore structure

We have already discussed pore structure with respect to soil water interaction (section 2.4). Here we will focus on the effects of desaturation on the various pore structures.

If we assume that air can only enter the sample from the exposed surface, then the effective pore size will control desaturation in all circumstances. Air will enter the soil as depicted in fig 2.7, with the entrance pore size determining the point of desaturation, and the pore structure, determining the shape of the desaturation curve.

If however we consider that air could form internally through cavitation, then the pore structure as a whole may determine the point of desaturation.

At this stage it is probably useful to consider cavitation in more detail.

2.6.3 Cavitation

This describes the instantaneous formation of bubbles within a fluid. There are two instances which would allow cavitation to occur:

i) Rupture of the fluid
ii) Expansion or growth of existing air bubbles

Rupture of pore fluid

The tensile strength of water has been calculated to be a minimum of 500 MPa, by Harvey (1944). It is therefore improbable that pure rupture of the pore fluid is a mechanism of desaturation.

Expansion or growth of existing air bubbles

Cavitation is experienced at far lower stresses in practice. It is readily seen as the cause of damage to pumps, turbines and ships propellers.

In theory, as a bubble reduces in size, the pore fluid tension required to sustain equilibrium increases (it is proportional to the radius of curvature of the air-water interface, eqn. 2.5). If the fluid tension is greater than that required for equilibrium, the bubble should grow, and conversely if the tension is lower than that required for equilibrium, the bubble should collapse.

Therefore tiny air bubbles should only exist where the pore fluid tension is very high. In practice microscopic air bubbles exist readily in a stable state under conditions of low fluid tension, Knapp (1970), Trevena (1987).

There are two theories explaining the ability of bubbles to exist in a stable state in a fluid whose tension is theoretically insufficient to support them:

i) Stable nuclei theory

This concept is based on the precept that bubbles reach a finite minimum size and do not dissolve completely. As the bubble reduces in size, due to the excess tension in the contractile skin, the air will diffuse through the menisci maintaining atmospheric pressure internally. It is hypothesised, Fox and Hertzfeld (1954), that the contractile skin contains impurities present in the fluid. As the bubble shrinks the impurities aggregate and ultimately present an impermeable barrier, precluding the escape of the remaining air within the bubble. The air pressure within the bubble now counteracts the tension generated by the contractile skin and prevents further shrinkage. Should the tension in the pore fluid become significant, it could cause the bubble to grow once more. This concept was explored by Yount (1979).

ii) Crevice model

Proposed by Harvey et al (1944), this theory is based on the stabilisation of gas bubbles in crevices in the solid surface. We can accept that there will be minute imperfections on the surfaces in which the fluid will
come in to contact. It is also likely that air bubbles will fill some of these crevices. Under normal conditions the air bubbles in these crevices have been subjected to atmospheric pressure and can easily be drawn out should tension develop in the fluid. The crevice model and the ability to stabilise this retained air is discussed in relation to the performance of tensiometers in section 4.3.4. For these purposes it is sufficient to note that the presence of such air allows cavitation to be instigated at low tensile stresses.

We have identified two concepts that would allow cavitation to occur at stresses significantly below that required for rupture of liquid itself. Both use existing, microscopic bubbles as points of instigation. The crevice model would allow cavitation to occur at much lower states of tensile stress than the stable nuclei theory.

It must be noted that as cavitation is defined as the formation of bubbles in the body of a fluid it is questionable as to whether either of the above models can be classified as cavitation in the purest sense. Perhaps cavitation should be reserved for the true rupture of the fluid and the formation of air bubbles without a point of instigation.

2.6.4 Adhesion

In the context of this study adhesion is defined as the bond between fluid and soil surface. It is very different in granular and clay soils. Firstly we will consider granular soils.

Granular soils

Fig 2.13 shows, in planar form, a menisci between three ideal spherical particles. Let us consider the effects of allowing the moisture to evaporate.

We can see a curved menisci in contact between two particles. Also we can see another "effective" menisci at the third particle surface. As the soil is allowed to dry, the air-water menisci retreats into the soil mass, with a corresponding reduction in radius of curvature. Whilst the radius of this menisci is greater than the radius of the soil particle the tensile effects at the soil / water interface ensure that the soil remains saturated. Once the radius of curvature of the menisci is less than the radius of the soil particle, adhesion between water and soil is required to maintain saturation.

Is such adhesion likely in practice?

Consider the surface of a granular soil on a microscopic scale. It is likely to have many imperfections, surface deposits and perhaps air bubbles attached. Any one of these could represent a point of instigation allowing loss of adhesion. These imperfections should not be viewed in the same way as was discussed
for cavitation above. In this instance we are considering a point which allows the water to peel away from the surface. Hence, once the radius of the menisci reduces to that of the particle desaturation could occur.

Clay soils

Surface adhesion for a clay soil is a far more complex process than for a granular soil. Clay soils have significant surface attraction for water (see section 2.3.2). The water closest to the surface is bound very tightly to the clay, thus making loss of adhesion at the clay surface less likely. This bonding process reduces with distance from the clay surface, however if loss of tension occurred here between the various water layers it would be more correctly termed cavitation of the fluid (which we have seen is unlikely to occur). The mechanism controlling desaturation is unlikely to be due to loss of adhesion, or cavitation of the pore fluid. Clays have the ability to retain moisture far beyond that predicted by the mechanism postulated for granular soils. We need to consider whether air entry due to the tensile stresses exceeding the capillary potential of the pore structure is the dominant influence.

2.6.5 Experimental data

We will examine the two proposed mechanisms, loss of adhesion (related to particle size) and capillary potential of the pore structure, for both granular and clay soils.

Loss of adhesion

Capillary potential is defined as the maximum suction that can be sustained for a given capillary radius, it is defined by equation 2.4. Fig 2.14 shows the relationship between capillary potential and pore radius. Superimposed on this diagram are the effective radii of sands, silts and clays. These can be used to estimate the range of suctions that could be sustained by the various particle size gradations.

Granular soils

From fig 2.14 we see that sands sustain suctions up to about 5 kPa and silts, between 5 - 140 kPa. The data from Brooks and Corey (1966), fig 2.15, shows two sands, both desaturating at less than 5 kPa. Also a silt is shown, this desaturates at about 7 - 8 kPa. Data from Burland (1961), fig 2.16, for a silt indicates a point of desaturation of about 30 kPa. All of these results fit with the mechanism proposed for granular soils.

Clay soils

Fig 2.14 indicates that clay soils should desaturate from 140 kPa upwards, depending on particle size,
if loss of adhesion was the controlling factor, and this was defined by the particle size. It is well known that clay soils can sustain tensions of several MPa in many instances without desaturating. Therefore loss of adhesion as defined above is not the mechanism of desaturation for clays.

**Capillary potential of the pore structure**

**Granular soils**

Let us consider the possibility of pore size controlling air entry for granular soils. Fig 2.17 shows three ideal particles. The effective pore size can be shown by simple trigonometry to be 0.15 \( r \). If we apply this factor to the range of particle sizes associated with sands and silts we can calculate the range of pore water tensions which could be sustained were this the controlling factor. The revised range of sustainable tension would be:

- **Sands** = 1 - 32 kPa
- **Silts** = 32 - 973 kPa

These values are far in excess of the experimental evidence for granular materials. This is unlikely to be the controlling mechanism of desaturation for granular soils.

**Clays**

We will consider the data of Al-Mukhtar (1995) and Ridley (1995). Both authors performed experimental work on Kaolin. Al-Mukhtar undertook porosimetry measurements on compacted Kaolin, Ridley defined the soil moisture characteristics for both compacted and reconstituted Kaolin.

Fig 2.18 shows data from Al-Mukhtar of pore size distribution for kaolin compacted at 1 MPa. We can see that the largest pore size identified is about 4000 Å (0.4 \( \mu \)m). If we compare a pore of this diameter (0.2 \( \mu \)m radius) on fig 2.14 we see a maximum capillary tension of about 700 kPa.

Fig 2.19 shows data from Ridley (1995) on the relationship between degree of saturation and suction for compacted and reconstituted Kaolin. The compacted sample was statically compacted to 1 MPa and should be similar to that studied by Al-Mukhtar. The air entry value is given as 650 kPa which compares very well with that inferred from the pore size analysis. The reconstituted sample de-saturates at 1000 kPa and although we do not have any data for the pore size distribution we can infer a possible pore size from the particle size. Kaolin is the largest of the common clay minerals and its particle size is likely to be at the upper end of the scale i.e. 2 \( \mu \)m diameter. If we assume that the particles are spherical this would give a pore radius of 0.15 \( r \), = 0.15 \( \mu \)m. From fig 2.14 the maximum sustainable pore tension would be 973
2.6.6 Summary

We have explored a number of options in developing an understanding of the process of desaturation. We have examined the influence of the properties of the pore fluid, its adhesion to solid surfaces and the possibility of cavitation. We have also looked at pore structure in simple mechanistic terms.

We have suggested two very different mechanisms are operating for granular and clay soils.

In granular soils desaturation is heavily influenced by the dominant particle size. Loss of adhesion may occur at the soil/water interface once the radius of curvature of the air-water interface becomes less than the radius of the soil particle.

In clay soils, this phenomenon is unlikely. The pore water is tightly bound to the clay surface, effectively precluding loss of adhesion as a possible mechanism. Cavitation of the pore fluid is also unlikely at the relatively low levels of tension at which desaturation occurs. Air entry from the surface is postulated as the most likely mechanism of desaturation in clay soils, occurring when the tensile stresses of the pore fluid exceeds the capillary potential of a particular pore structure.

The mechanisms of desaturation postulated for granular and clay soils are supported by simple theory and limited experimental data.

2.7 HYSTERESIS

The hysteresis observed between wetting and drying moisture characteristics is a phenomenon experienced in all porous media. The moisture content suction relationship has a common pattern, with the drying portion forming an upper boundary curve and the wetting portion scribing a lower boundary curve, fig 2.20.

The separation of the two boundary curves is influenced by several factors including pore fluid, pore structure and the movement of wetting and drying fronts. In addition clay soils may experience irreversible fabric and hence volume changes.
2.7.1 Pore fluid contact angle

Under ideal conditions, perfect wetting, a single contact angle, $\theta$, would describe the relationship between the fluid and surface. However fluid composition, surface irregularities and adhesive properties all contribute to a variance in the contact angle under real conditions.

The most significant influence on the value of $\theta$, is whether the fluid is advancing or retreating. The angle will increase with an advancing fluid and decrease with a retreating fluid interface, fig 2.21(a).

The effect of this change in contact angle is that the angle of contact will be smaller during loss of moisture (drying). Thus for a given pore size the radius of curvature of the air-water menisci will be smaller and hence suction greater during drying than wetting, fig 2.21(b).

2.7.2 Constricted pore structure

The effects of a constricted pore structure have been discussed with respect to evaporation (section 2.4.1.2) and pore size determination (section 2.5.3.2). These principles can be applied equally to the drying and wetting cycles of a soil.

During drying, moisture will be retained within a pore structure until a suction sufficient to overcome the entrance pore size is developed. At this point moisture will be lost from all pores equal to or larger than the entrance pore. The entrance pore will control the point of air entry.

This concept was demonstrated by Ridley (1997) using the newly developed IC tensiometer (discussed in detail in chapter 4). During an investigation into the cause of tension breakdown of the instrument, he fitted a 100 kPa air entry value (AEV) porous stone beneath a 1500 kPa AEV porous stone in the tensiometer. He saturated the instrument and allowed the moisture to evaporated from the exposed (1500 kPa AEV) surface. The point at which tension breakdown occurred would indicate the controlling pore size. A tension of in excess of 1000 kPa was reached clearly indicating that the entrance pore size controls the point of air entry.

It is not practical to undertake a wetting up exercise, however one could expect that the pores in the 1500 kPa AEV stone would fill first, and those in the 100 kPa AEV at a much lower suction during wetting.

2.7.3 Entrapped air

If a soil, in an initially saturated state, is gradually dried to a residual moisture content and subsequently
allowed to wet up, the moisture content - suction relationship for drying and wetting will not necessarily
form a closed loop.

During the first wetting up cycle the advancing moisture front will often trap pockets of air within the soil
mass. This is particularly likely where constricted pore structures exist, as the smaller entrance pore will
fill first.

However, subsequent drying and wetting cycles will form a predominantly closed loop.

2.7.4 Clay soil

Permanent structural changes will occur in a clay soil during its first drying cycle. If we assume that the
soil mass is fully saturated, (with nominal suction), then we can accept that the clay particles' attraction
for free moisture is satisfied. As such the soil mass may be composed of a random array of soil particles,
reflecting perhaps depositional and stress history.

As moisture is lost, the particles will move closer together and their respective double layers will begin
to interact, causing repulsion between particles (see section 2.3.2). The particles cannot move further apart
under these conditions, but may undergo re-orientation, with the particles adopting a more efficient
alignment which will reduce the extent of double layer interaction. A more parallel alignment is common.

Subsequent wetting up will not cause a reversal in alignment as the particles now exist in an efficient
arrangement. Thus whilst they may move apart and absorb water they tend to retain the new preferred,
alignment.

The effect of this suction induced compression will be that the initial drying curve will lie above any
subsequent drying, or wetting, curves.

2.7.5 Scanning curves

We have noted three distinct curves associated with the wetting and drying behaviour of soil, fig 2.22. The
initial drying curve represents the greatest moisture content change of the soil. This curve is not closed
by the wetting curve, and as such the difference in behaviour cannot be termed truly hysteretic. The
wetting curve defines the lower boundary of the moisture content suction behaviour. The second drying
curve describes the upper boundary curve for the soil.

Theoretically these should form a closed loop. In making this statement, it should be noted that:
i) The drying curve must be established from the fully wetted state (zero suction) to the air dry condition.

ii) The wetting curve must be established from a residual moisture content to a fully wetted state.

The term residual moisture content is often used when discussing soil moisture characteristics. It is taken as that moisture remaining at the conclusion of the drying process. However the amount of moisture remaining will be directly influenced by the atmospheric conditions under which the sample is allowed to dry and can be considerable in more plastic soils. Fredlund & Xing (1994) define the residual moisture content as the intersection of a tangent drawn from the point of inflection of the moisture content - suction curve and one from the linear part of the relationship at very high suctions. This approach is useful in establishing comparable parameters for the purposes of modelling, however soils wetted from this moisture content may not necessarily define the lower boundary curve.

It is proposed that the residual moisture content be defined as the moisture content at which a sample having undergone drying, will define the lower soil moisture characteristic boundary upon subsequent wetting.

In addition one should consider the effects of creep, and perhaps the effects of a number of wetting and drying cycles on the bonding and particle arrangement of the soil. Thus it is possible that there will be a small but progressive, non-closure of the boundary curves.

The actual wetting or drying path will only follow these boundary curves if it commences from one of the limiting states of suction, i.e. zero (fully wetted) for a drying curve determination or an air dry condition.

If wetting or drying starts from an intermediate point, and does not represent a monotonic continuation, a different moisture content - suction relationship will be defined, fig 2.23.

The path will be contained within, and influenced by the boundary curves. It may even touch one of them (although probably only at the intersection of the boundary curves), however the actual relationship will be unique. Childs (1969) called these intermediate curves "scanning curves", a term used by many authors.
<table>
<thead>
<tr>
<th>Clay mineral</th>
<th>specific surface area (m²/g)</th>
<th>cation exchange capacity (me/100g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>5 - 20</td>
<td>3 - 15</td>
</tr>
<tr>
<td>Illite</td>
<td>100 - 200</td>
<td>10 - 40</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>700 - 800</td>
<td>80 - 150</td>
</tr>
</tbody>
</table>

*Table 2.1 clay mineral properties*

<table>
<thead>
<tr>
<th>Property</th>
<th>Kaolinite</th>
<th>Illite</th>
<th>Montmorillonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface area</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>Cation exchange capacity</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>Expansion of the clay lattice</td>
<td>X</td>
<td>X</td>
<td>✓</td>
</tr>
<tr>
<td>Hydration of the adsorbed layer</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Extent of diffuse (absorbed) layer</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
<tr>
<td>Hydration of diffuse layer</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Capillarity effects of particle interaction</td>
<td>Low</td>
<td>Medium</td>
<td>High</td>
</tr>
</tbody>
</table>

*Table 2.2 comparison of activity of clay minerals*
Figure 2.1 Clay mineral structures

(a) Silica tetrahedron

(b) Alumina octahedron

(c) Illite structure

(d) Montmorillonite structure
Chapter 2 page 35

Clay particle adsorbed water

Absorbed water

Free water

Section X - X

Figure 2.2 Double layer water (after Bolt 1956)

Capillary tube

Air / water interface

Figure 2.3 Capillary rise and air water interface
figure 2.4 Meniscic curvature between two discrete particles

figure 2.5 Theoretical soil moisture characteristic for uniform granular material
figure 2.6 experimental data on effect of particle size on soil moisture characteristic (after Krynine, 1948)

1,2,3... = retreating air water interface

figure 2.7 loss of moisture from soil through evaporation (after Childs, 1969)
Figure 2.8 SEM of lightly consolidated kaolin (after Smart and Tovey 1981)

Figure 2.9 Intrinsic compression data
Figure 2.10: Comparison of compression curves for mechanical, suction, and gravitation compression of remoulded clay (after Blight 1965)

Figure 2.11: Effect on soil moisture characteristic of clay silt ratio (after Marinho 1994)
Figure 2.12: Critical point and freeze drying
pore water tension at limiting condition
radius of particle, \( R = \text{radius of menisci, } r \)

*figure 2.13 limit of pore water adhesion in granular soils*
figure 2.14 theoretical capillary potential

figure 2.15 matrix suction against degree of saturation (after Brooks and Corey, 1966)
Figure 2.16 Drying of a silt from a slurry (after Burland, 1961)

Figure 2.17 Entrance pore size for desaturation through air entry
Figure 2.18 Pore size distribution for compacted Kaolin
(after Al-Muhhtar, 1995)

Figure 2.19 Soil moisture characteristics for kaolin
(after Ridley, 1995)
Chapter 2 page 45

Figure 2.20 typical hysteretic behaviour for porous media

1. **Contact Angles for Advancing and Retreating Fluid**
   - Fluid gain: \( \theta_a > \theta_b \)
   - Fluid loss: \( \theta_b > \theta_a \)

2. **Suction Developed from Advancing and Retreating Fluid**
   - Suction a < suction b

Figure 2.21 effect of pore fluid contact angle
Figure 2.22 Initial drying curve and subsequent hysteretic behaviour

Figure 2.23 Scanning curves
Chapter 3

The influence of suction on compressibility and swelling

3.1 \( \text{INTRODUCTION} \)

The main focus of soil mechanics has, historically, been on fully saturated soils, where all the voids are assumed to be filled with water. Early seminal work in this field was often undertaken in temperate climates, and the assumption that the saturated case was the worst possible soil state, coupled with the development of the effective stress principles, has guided much research in this direction, Alonso et al (1987).

However the majority of the populated landmass (60%, Fredlund 1995) is in a condition where the above idealisations are not applicable. In arid environments the water table is often situated at some depth below ground level. The soil some metres above the water table may remain saturated through capillarity, but the pore water will be in a state of suction. Above this capillary zone, the soil will be in a condition of partial saturation, with voids filled with both air and water. This final point is of fundamental significance, as it renders the principles of effective stress invalid.

Whilst partly saturated soils are a special case, (Burland and Ridley, 1996), they should not be viewed as one of limited occurrence, confined only to arid environments. Many temperate climates have soils which, close to the surface are in a state of pore water tension, and may also be partly saturated. Man made soil structures, slopes, embankments etc, constructed from compacted material, will generally be in a state of moisture deficiency. Also many exposed soil surfaces exist in a partly saturated state.

In chapter 2 we looked at soil water relationships on a microstructural scale, exploring the interaction of water with both granular and clay soils, and the development of soil suction.

In this chapter it is intended to consider a simplified mechanistic approach to the influence of suction on compression and swelling behaviour.

3.2 \( \text{MECHANISTIC BEHAVIOUR} \)

Burland and Ridley (1996) made two propositions in their discussion on the importance of soil suction in soil mechanics:
i) Soils are particulate and essentially frictional.

ii) Resistance to compression, swell and shearing is enhanced by inter-particle bonding.

They added a corollary to this second proposition:

Removal of such bonds can give rise to compression or swell, and a reduction in shearing resistance.

It was noted that these "bonds" are as likely to result from pore water tensions within menisci at the grain contact points, as from cementation.

The second proposition leads us to examine the applicability of Terzaghi's effective stress principles in partly saturated soils.

### 3.2.1 Validity of the effective stress concept for partly saturated soils

Let us first consider the fully saturated case. In fig 3.1 we see two discrete soil particles. There are two stresses acting on the soil grains. Firstly that due to the external applied stress, $P$, which generates both normal and shear forces between the two particles. Secondly, there is the stress generated by the pore fluid pressure, $u_w$. The pore fluid pressure acts equally on the soil and the fluid, in all directions.

Terzaghi, (1936) defined a single stress term, the effective stress, which could be used to describe the behaviour of the soil. He stated that "all measurable effects of a change in stress, such as compression, distortion and a change in shearing resistance are exclusively due to changes in the effective stress".

Burland and Ridley (1996) reiterated two important points implicit in Terzaghi's statement:

i) The statement does not describe inter-granular stresses, merely the overall effect of a stress change, which has its "seat in the solid phase of the soil"

ii) It gives no indication of pore pressure distribution, the fluid pressure is defined as the stress measured through a porous tip in equilibrium with the pore fluid.

These two points serve to emphasise that the effective stress principle relates to a mechanistic and not microstructural level.

The success of the development, and application, of the effective stress principles for the special case of saturated soils gave impetus to the belief that a similar concept could be derived that would account for the behaviour of partly saturated soils. A number of relationships were proposed in the 1950's, table 1. The
equation proposed by Bishop (1959) was generally accepted at a conference on "Pore pressure and suction in soils", Skempton (1961).

Bishop related the relative areas of soil under the influence of the pore air and pore water, using a factor $\chi$. Consider the partly saturated soil grains in fig 3.2. We can consider that the pore water will act over an area equal to $\chi A$, and the pore air will act over an area equal to $(1-\chi) A$. This leads to a revised pore pressure term accounting for the contributions of both the pore air and pore water, eqn 3.1

$$pore\,\text{pressure} = \chi u_a + (1-\chi) (u_a - u_w) \quad 3.1$$

where

$u_a = $ pore air pressure
$u_w = $ pore water pressure

This can be incorporated into Terzaghi's equation for effective stress, eqn 3.2

$$\sigma' = \sigma - u_a + \chi (u_a - u_w) \quad 3.2$$

Where $\sigma' = $ effective stress in the soil
$\sigma = $ total applied stress

Bishop's equation is statically correct, however to satisfy the effective stress principle, changes in soil behaviour, either shear or volume changes must only occur as a result of a change in the effective stress.

Jennings and Burland (1962) were among the first authors to challenge the validity of this equation. They demonstrated that, although perhaps statically correct, Bishop's equation did not satisfy Terzaghi's definition of an effective stress. Soil behaviour differed markedly depending on whether a change in effective stress was brought about by a change in the pore water pressure or the total stress component. They also demonstrated the variation of Bishop's $\chi$ factor with respect to both degree of saturation and soil type, fig 3.3.

Let us consider once again the total stress and pore fluid pressures, and the forces they generate in a partly saturated soil, fig 3.4. The total stress produces both normal and shear force components, ($\sigma$ & $\tau$). The pore fluid tension produces a normal force only, ($\phi$), at the grain contact points. Extending this to a column of soil grains allows us to investigate the effects of changing each component in isolation, fig 3.5.

Increasing the effective stress by increasing the total stress, P, fig 3.5(a), reduces stability of the column as the ratio of shear to normal forces increases. Producing the same change in effective stress by increasing the pore water tension however, fig 3.5(b), results in a more stable structure as the pore tension contributes a normal force only. Wheeler and Karube (1995) stated that the increase in pore tension represented an increase in effective stress in one sense, as there was an increase in normal forces, but could equally be viewed as a reduction in effective stress, as the net result was one of an increased overall
stability.


3.2.2 Suction component influencing soil behaviour

Before considering further the influence of suction on soil behaviour we will examine the individual components and their effects on the soil behaviour.

Shear strength

It is generally accepted that the matrix suction is the controlling suction component for shear strength behaviour of a partly saturated soil. There has been much debate however, over the effect of osmotic suction on shear strength. Skempton & Northey (1952) and Kenny et al (1967) both presented data from triaxial shear strength tests on marine clays. The salt content, and hence osmotic suction was determined and varied. Shear strengths were determined at each stage. Despite variations of osmotic suction of up to 2 MPa the shear strength measurements varied by less than 15 kPa.

Although it is difficult to separate the effects of the osmotic suction from the physio-chemical effects that the soluble salts may have on the clay fraction, from these data it can be said that osmotic suction has little effect on the shear strength.

Volume change

The influence of osmotic suction on volume change is not clear. A number of workers have investigated the effect of various pore fluid compositions on compression and swelling. A common test is to allow a partly saturated sample free access to water containing different solute and solute concentrations (Blight 1983, Richards et al 1984). Alonso et al (1987) commented that whilst differences in behaviour were noted when the solute in the pore fluid was varied, it had yet to be confirmed that this was due to the osmotic suction or a physio-chemical reaction between the solute and the active clay minerals present.

Fig 3.6, (Richards, 1984), shows the effect of reducing the osmotic suction generated by a number of different salts. The sample containing NaCl salts swells noticeably when the effective osmotic suction is reduced, whereas those containing different salt types do not. The effect seen here appears to be due to the salt type, although the osmotic and physio-chemical effects are not separated.

Changes in compression and swelling behaviour are also noted in saturated soils when the pore fluid
composition is varied. This suggests that physio-chemical reactions between soil and solute do occur. These could mask any osmotic suction effects and make it difficult to be conclusive as to the osmotic contribution, if any, to changes in soil behaviour.

3.3 COMPRESSION AND SWELLING OF SOILS

3.3.1 Saturated soils

Fig 3.5 (a) depicts a column of soil grains subjected to both external total stress and pore fluid pressure. As the total stress is increased the shear and normal forces increase. The angle of shearing resistance for the particles is given by equation 3.3

$$\frac{\tau}{\sigma} = \mu$$

Where

$\tau$ = shear stress at the grain contact
$\sigma$ = normal stress at the grain contact
$\mu$ = angle of shearing resistance

Providing the ratio of $\tau/\sigma < \mu$ grain slippage will not occur. Once $\tau/\sigma \geq \mu$ slippage at the grain contact points will commence. This may occur locally at first, with a larger stress required to cause generalised slippage. This will result in irrecoverable straining of the sample or plastic deformation. Significant straining is generally identified as a yielding of the soil.

It should be remembered that because the sample is saturated the principle of effective stress will still apply. Hence this yielding could be brought about by either an increase in total stress or a reduction in the pore fluid pressure.

Fig 3.7 shows the isotropic compression curve for kaolin. Following initial compression, a small cycle of unloading-reloading is performed, (test T10). Unloading, B - C, does not recover the initial compression between A - B, indicating that grain slippage and plastic deformation has occurred. The extent of plastic deformation is given by $e_A - e_C$. Upon reloading, C - D, the path followed is approximately the same as that for B - C with significant yield occurring at the point B, indicating that the volumetric changes that occurred upon unloading, were largely elastic and recoverable. The yield at point D is the boundary between quasi-elastic and plastic behaviour. Test T14 follows the same initial stress path. The exception is that the sample is unloaded to a much lower stress, C', before reloading. We see that the reloading path does not follow the unloading path closely, and that the yielding occurs over an increased stress range.
This shows that there is some plastic deformation occurring during the unloading phase. Elasto-plastic models and the behaviour of soils at small strains can account for this behaviour, Burland (1989) & Jardine (1992).

To summarise, we can say that for small stress changes, unload-reload cycles may move largely within elastic boundaries. Larger stress cycles involve plastic, unrecoverable strains. The governing equation from a simple, mechanistic viewpoint is the relationship between normal forces, shear forces and the angle of shearing resistance, $\mu$.

### 3.3.2 Partly saturated soils

Before exploring the behaviour of partly saturated soils it is perhaps useful to define the most appropriate general terminology to use in this field, un-saturated or partly saturated.

Delage and Graham (1995) discussed this aspect in their state of the art report, suggesting that a single term should be used, and that it should be un-saturated. However they also conceded that there were many instances where partially saturated would be equally if not more applicable!

Their arguments are that un-saturated should be applied where the degree of saturation is both low and uniformly distributed, for example in granular soils. The term partly saturated would be more applicable where the degree of saturation is both higher and varied throughout the soil mass. Compacted soils are a good example of this, containing clay packets with a high degree of saturation in a matrix with a lower overall degree of saturation.

The adoption of a single, generic term is required for clarity, and partly saturated is suggested. This term can be applied equally to any soil where the degree of saturation is less than 1, irrespective of its actual value. It is not biased towards a uniform, or distributed degree of saturation, level of suction, or soil type. Furthermore it is an easily understood term without ambiguity.

#### Granular soils

Jennings and Burland (1962) performed a series of oedometer tests to examine the effects of suction on granular soils.

Five samples of silty sand were dried to various degrees of saturation from a slurry. They were then compressed, one dimensionally in an oedometer. Three were then wetted up, under a maximum load (1500 kPa), one was wetted at 800 kPa and then loaded to 1500 kPa, and one wetted prior to loading. In addition the intrinsic compression line (ICL) was determined. The results are given in fig 3.8.
Burland and Ridley (1996) made the following observations on this data:

i) The compression curves for the partly saturated samples all cross the intrinsic compression line.

ii) As the applied stress increases the partly saturated samples "yield".

iii) The "yield stress" increases with decrease in degree of saturation.

iv) When soaked under constant applied stress the partly saturated samples "collapse" onto the saturated intrinsic compression line.

v) The sample soaked at zero suction converges on the saturated intrinsic compression line.

These tests clearly show the stabilising effect of suction in a partly saturated granular soil. Once discrete menisci form at the grain contacts, the additional normal force creates a stiffer material.

It should be noted that these tests were all carried out at constant water content, and not constant suction (excluding the sample soaked prior to loading). During compression, the void ratio will reduce resulting in an increase in degree of saturation, and hence a decrease in suction.

Remembering the stabilising effect that suction has at the grain contact points in a partly saturated material, any reduction in suction would allow increased grain slippage to occur.

This effect can be seen in the post yield compression behaviour. The compression lines all tend towards the ICL, behaving as a more compressible material than that compressed from a slurry. This is due to the combined effects of increasing applied stress and reducing suction.

The sample at an initial degree of saturation of 69.5% allows us to draw some interesting conclusions when compared to the ICL. Initially it behaves as a much stiffer material (than the saturated sample), crossing the ICL. A gradual yielding of the sample occurs between 50 - 200 kPa following which the sample behaves as a softer material and its compression path tends towards that of the ICL. At 1500 kPa the two are coincident for all practical purposes. It could be concluded that at this stage the degree of saturation was high, and the suction (if present) had reduced to a nominal value.

As discussed in chapter 2, granular soils have little attraction for moisture beyond capillary effects. As such, they do not exhibit any discernable swelling upon wetting. The reduction in suction associated with wetting can lead to collapse of the soil structure. This is due to the loss of the inter particle "suction bond". Collapse upon wetting is seen in fig 3.8 when the samples at initial degrees of saturation of 49.4%, 40.7% and 36.3% are wetted up under load. They all subsequently collapse onto the ICL following loss of suction.
Clay soils

Clay soils behave in a fundamentally different way to granular material. They have a significant attraction for moisture, both at a particle level and on an aggregate basis. We have discussed clay water interaction on a particle level in section 2.3.2, we will now consider clay soil behaviour from a generalised mechanistic approach.

From a saturated state, significant shrinkage occurs during drying of a clay soil. The nature of both a small particle size and a strong interaction with water, allows clay soils to remain fully saturated to very high levels of suction (upwards of 10 MPa for very plastic clays).

Whilst saturated, the suctions generated will act as an all round, isotropic stress. This can be compared to the application of an external, total, isotropic stress in many instances.

Isotropic compression will cause moisture to be forced from the soil. Some particle reorientation will occur during the primary consolidation phase. Secondary consolidation, or creep, is evidence of further particle reorientation due to strong interparticle forces. In expansive soils this may represent a noticeable amount of compression, and can continue for some time after the initial consolidation is complete.

During drying one could expect to see the same effects. However, when a sample is allowed to dry freely, cracking and desiccation of the clay mass may occur. This is particularly likely if the soil is very plastic, the rate of drying high, or duration prolonged. The confinement placed on a sample during mechanical compression prevents cracking occurring.

One result of cracking occurring is that a more open fabric results, with tightly bound clods of clay in a more open matrix. On a mechanistic level these two fabrics give significantly different responses.

The fabric will not only influence the soil water retention characteristics but also the effects of suction on the behaviour. Once the sample has desaturated, the clay soil is likely to consist of some fully saturated phases in a partly saturated matrix. Furthermore the areas consisting predominantly of clay will tend to remain saturated, whereas the granular material will have lost moisture more readily.

During compression the fabric is altered, and often destroyed in a saturated soil. One effect of suction is to preserve the initial fabric to higher states of stress.

We can visualise a clay soil as consisting of both pure clay minerals and granular material. Fig 3.9 shows a number of possible fabric arrangements, with clay packets existing between, and bonded to the granular material.
*Increase in suction*

The clay infill or "packets" will shrink and become stiffer as the suction increases. The clay portion will provide the strength of the soil mass. If the level of suction becomes high enough cracking and desiccation may occur. The overall structure could now be in a loose metastable state.

*Reduction in suction*

The clay packets will absorb water and swell. The volume change associated with this is not clearly defined, but very much dependent on the magnitude of the change in suction, applied stress condition, clay mineralogy and fabric. The packets could distort during swelling, filling cracks and larger void spaces with little overall volume change observed. They may swell uniformly, maintaining their overall shape and generating measurable volume change. In addition to swelling, the shear strength of the clay will reduce and, depending on the fabric, a reduction in volume through collapse could occur. Fig 3.10 shows the possible volumetric changes that could occur following a reduction in suction at various states of applied stress.

Gehling et al (1995) presented two aspects of swell and swell - collapse behaviour. Fig 3.11 shows the swelling behaviour of a compacted sample which is held at constant water content. A number of samples of an expansive grey clay were compacted to a range of dry densities. They were then placed in an oedometer and, at constant water content, monitored for swelling strains. The swelling continued for up to one week. It is postulated the clay fraction absorbed water from the granular material allowing it to swell. This redistribution of moisture allowed swelling to occur without any overall change of moisture content. Initially the sample would have been at a uniform moisture content but not at suction equilibrium. At the conclusion of swelling the sample would be at suction equilibrium but have a distributed moisture content profile.

Gehling et al (1995) also presented results of a test on a swell-collapse mechanism. Two samples of the same expansive clay were compacted and placed in the oedometer. One was allowed free access to moisture, the other held at its "as compacted" moisture content. Both were prevented from swelling and the pressure required to maintain constant volume conditions monitored. The results are shown in fig 3.12. The pressure required to prevent swelling in the sample allowed free access to moisture increased to a peak and gradually fell back to a stable value. This phenomenon can be visualised as one where the clay packets initially swell, creating a high swelling pressure. With time there is some distortion of the clay packets as their strength reduces with increasing moisture content. This reorientation leads to a reduction in measured pressure. The same trend of behaviour is observed in the second sample, which is held at a constant moisture content. Here, once more, the clay packets draw moisture from the surrounding granular material as the sample moves towards suction equilibrium. The lower peak swelling pressures probably indicate that a suction exists in the soil as a result of the restricted availability of moisture.
The influence of stress path on swelling behaviour was demonstrated by Schreiner et al. (1994) and reviewed by Burland and Ridley (1996). Schreiner compared three "standard" swell tests. The swell test procedures are outlined in fig 3.13.

**Swell test 1** is known as a double oedometer test and was introduced by Jennings and Knight (1957). It involves comparing the compression at natural moisture content with that in a similar sample which has been allowed to swell under a low applied stress before compression.

**Swell test 2** is attributed to Holtz and Gibbs (1956). The sample is loaded at natural moisture content and is then allowed to swell under constant load.

**Swell test 3** was proposed by Sullivan and McClelland (1969). Initially the sample is flooded whilst keeping the sample volume constant, by increasing the applied vertical stress. Once swelling is complete the sample is unloaded to the required vertical stress.

The stress paths followed by each test is shown in fig 3.14. The stress path dependency of these tests can be clearly seen. The final stress state and void ratio condition varies considerably. Also the hypothesis that as a compacted sample swells, the clay "packets" may distort as they reduce in strength is implicitly confirmed in these results.

### 3.4 SUMMARY

We can draw a number of general conclusions concerning the effects of suction on soil behaviour:

- All soils can be considered as particulate and essentially frictional, with yielding occurring through interparticle slip.

- The interparticle forces generated from suction increase stability at the grain contacts. Applied forces can cause instability, and induce grain slippage.

- Soil stiffness increases with increasing suction, although there is probably an upper limit to this effect.

- The treatment of applied stress and suction separately is essential in developing an understanding of partly saturated soil behaviour.

- The fabric of a partly saturated soil is of greater importance than for a fully saturated soil.
A reduction in suction can cause swelling or collapse in the sample.

The actual behaviour is dependent on many factors including soil type, fabric, applied stress level and magnitude of reduction in suction. The interaction between swelling-collapse behaviour and applied stress is shown schematically in fig 3.8.

The greatest proportion of swelling takes place at low levels of suction and it is this that is suppressed first, upon application of a confining stress. This trend is also apparent in saturated soils. Compression and swelling characteristics are generally plotted on a logarithmic stress axis, (e - log $\sigma$), giving a linear relationship in this semi-log space. If we were to plot on an arithmetic stress scale we would see that the majority of the compression, or swelling, also occurs at low confining stresses in saturated samples.

The magnitude of swelling experienced following wetting up of a sample can be controlled by the application of a vertical stress.

The applied vertical stress influences the extent of collapse upon wetting. With increasing applied stress, the magnitude of collapse reaches a maximum and then reduces.

This is probably an indication of the applied stress overcoming the stabilising influence of the matrix suction, and altering the fabric of the soil (through compression, and perhaps plastic deformation) before wetting, and subsequent collapse occurs.

The point of inflection between swelling and collapse behaviour following a reduction in suction could be defined as the critical stress.

This stress, is not however, a unique value, and is very much path and soil type dependent.

Swelling and swell pressure determinations are very path dependent.
<table>
<thead>
<tr>
<th>Author</th>
<th>Effective Stress Equation</th>
<th>Stress Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Donald (1956)</td>
<td>$\sigma = \sigma + p''$</td>
<td>$p'' = \text{pore pressure deficiency below atmosphere}$</td>
</tr>
<tr>
<td>Hilf (1956)</td>
<td>$u = u_a + u_m$ where $u_i = (-)T{1/r_1 + 1/r_2}$</td>
<td>$T = \text{surface tension of water}$ $r_1, r_2 = \text{interior and exterior radii of curvature of menisci}$</td>
</tr>
<tr>
<td>Croney et al (1958)</td>
<td>$\sigma = \sigma - \beta u_m$</td>
<td>$\beta = \text{Factor indicating number of bonds under tension, contributing to soil strength}$</td>
</tr>
<tr>
<td>Bishop (1959)</td>
<td>$\sigma = \sigma - u_i + \chi(u_m - u_a)$</td>
<td>$\chi = \text{parameter relating to degree of saturation}$</td>
</tr>
<tr>
<td>Aitchison (1960)</td>
<td>$\sigma = \sigma + \psi p''$</td>
<td>$\psi = \text{parameter with values varying between 0 - 1}$ $p'' = \text{pore pressure deficiency}$</td>
</tr>
<tr>
<td>Jennings (1960)</td>
<td>$\sigma = \sigma + \beta p''$</td>
<td>$\beta = \text{empirical, statistical factor relating to contact area}$</td>
</tr>
<tr>
<td>Richards (1966)</td>
<td>$\sigma = \sigma - u_i + \chi_m(h_m + u_a) + \chi_s(h_s + u_a)$</td>
<td>$\chi_m = \text{effective stress parameter for matrix suction}$ $h_m = \text{matrix suction}$ $\chi_s = \text{effective stress parameter for solute suction}$ $h_s = \text{solute suction}$</td>
</tr>
</tbody>
</table>

*Table 3.1 Effective stress equations for partly saturated soils (after Maswoswe, 1985)*
$P = \text{applied stress}$  
$\sigma = \text{normal stress}$  
$\tau = \text{shear stress}$  
$U_W = \text{pore pressure}$

**Figure 3.1** Forces in fully saturated soil

$P$  
$\sigma$  
$\tau$  
$U_W$

**Figure 3.2** Schematic representation of Bishop's $\chi$ factor  
(after Skempton, 1961)

$pore air pressure u_a$  
$pore water pressure u_W$

$\chi A$
Figure 3.3 variation of Bishop's factor with soil type and degree of saturation (after Jennings and Burland, 1962)

Figure 3.4 forces in fully saturated soil

P = applied stress
σ = normal stress
σ_s = normal stress due to suction
τ = shear stress
U_W = pore pressure
Figure 3.5 grain column analogy

Figure 3.6 effect of oedometer swelling tests on Pleistocene clay samples with different solutes (Richards et al 1984)
Figure 3.7 isotropic compression and swelling on saturated Kaolin (after Burland 1967)

Figure 3.8 oedometer compression curves for silty sand. (after Jennings and Burland, 1962)
figure 3.9 particle arrangements (after Mitchell, 1992)
Chapter 3  page 64

void ratio

log vertical stress

final void ratio following soaking at various vertical stresses

swell

dry compression curve

collapse

Figure 3.10 Swell / collapse behaviour following wetting up of a dessicated clay soil (after Burland and Ridley, 1996)

Figure 3.11 Swelling of compacted clay soil at constant water content (after Gehling et al, 1995)
\[
\gamma = 12.7 \text{ kN/m}^3 \quad \omega = 15 \text{ o/o}
\]

**Figure 3.12** Swell collapse behaviour of compacted clay (after Gehling et al, 1995)

**Figure 3.13** Three "standard" swell test procedures (after Schreiner et al, 1994)
Figure 3.14 Stress paths of three "standard" swell tests (after Burland and Ridley 1996)
Chapter 4

Soil suction and its measurement

4.1 DEFINITIONS OF SOIL SUCTION

In saturated soil mechanics, we often experience positive pore pressures. The positive pore pressure can be used to develop the classical effective stress equations ($\sigma^* = \sigma - u$).

In many situations however, there is a net deficiency of soil moisture. Under these conditions the pore fluid will be under tension (whether the voids are filled wholly with fluid or contain some air as well). The soil will exert a net attraction for free moisture in this situation and the measurement of pore pressures will register negative gauge pressures. These negative gauge pressures are more commonly referred to as suctions.

Soils suction is the result of a combination of physical and chemical effects within the soil matrix. Suction concepts were first developed in the field of soil physics, by researchers studying plant water relationships. Buckingham (1907) is often cited in reference to early work, as is Schofield (1935). Indeed the British Soil Water Physics group recently held a seminar in honour of Schofields work, "pF at 60".

It was not until the 1950's that interest in soil suction by the geotechnical community developed, following much work on the subject at the Road Research Laboratory by Croney, Coleman and Black amongst others.

Definitions of the components of soil suction were established at the 1st International Conference on Expansive Soils, Aitchison (1965):

**Total Suction**

_The negative gauge pressure relative to external gas pressure on the soil water to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane._

**Matrix Suction**

_The negative gauge pressure on the soil water to which a solution identical in composition to the soil water must be subjected in order to be in equilibrium through a porous permeable wall with_
Osmotic Suction

The negative gauge pressure to which a pool of pure water must be subjected in order to be in equilibrium through a semi-permeable membrane with a pool containing a solution identical in composition to the soil water.

In thermodynamic terms these definitions are acceptable, but they present difficult concepts, particularly from a practical viewpoint.

Ridley (1993) proposed a new visualisation, which simplifies the existing definitions without altering the underlying concepts. These definitions were developed with respect to the ability to measure the individual components:

Total Suction

The stress required to extract a water molecule from the liquid phase of the soil matrix into the vapour phase (i.e. the ability of the soil to resist the loss of water through the latter's change of state from the liquid phase into the vapour phase).

Matrix Suction

The stress existing on the water molecule in the liquid phase of the soil matrix (i.e. the ability of the soil to resist the loss of water without the latter changing its state).

Differential Suction

The difference between the measured total and matrix suctions (as defined above)

4.1.1 Terminology

At the 1st International Conference on Unsaturated Soils (1995) terminology for the various suction components was discussed. In view of the significant number of symbols currently being used a consensus was reached as to common nomenclature for future publications. These were:

Matrix suction = \(u_n - u_m\)

Osmotic suction = \(\pi\)

Total suction = \(\psi\)
Units of Pascal (including kPa and MPa) were also adopted. It should be noted that a negative pore water pressure is equivalent to a positive suction, i.e. pore pressure = -300 kPa = suction = 300 kPa.

4.1.2 Matrix suction

The matrix suction is the stress in the liquid phase of the pore water. Its derivation with respect to the capillary model (Buckingham, 1907) was discussed in section 2.4.1.1. Equation 4.1 describes the matrix suction:

\[(u_a - u_w) = \frac{2\tau}{r}\]

where

- \(r\) = radius of capillary tube
- \(u_a\) = ambient air pressure
- \(u_w\) = pore water pressure
- \(\tau\) = surface tension

If we take \(u_a\) as a constant reference pressure and, assuming that \(\tau\) does not vary, we see that the maximum matrix suction (defined as a negative pore water pressure) is a function of the capillary radius, fig 4.1

This concept of the capillary model works well for inert granular material, where a finite, generally stable pore structure predominates in controlling matrix suction.

For clay soils the situation may be somewhat different. Clay particles differ in two important ways from the idealised granular material. They are elongated plate-like particles, and are electrically charged, resulting in adsorption forces close to the particle surface.

For the purposes of defining matrix suction it is sufficient to note that the water close to the clay particle surface differs in both composition and behaviour from that at some distance. Clay-water interaction is discussed more fully in chapter 2.

These adsorption forces may affect, and even control, the matrix suction under certain conditions. The tensile strength, salinity and density of the adsorbed water layer differs significantly from that in the "free" water, hence it may be difficult to define matrix suction in this situation through reference to the capillary model alone.
However, the measured matrix suction will define the affinity for moisture (through the liquid phase) and as such will account for both capillary and adsorption forces. Thus the measured value is representative of the actual matrix suction, and it is this which influences the soil behaviour.

4.1.3 Osmotic suction

The mechanism of osmosis is discussed in detail in chapter 5. The addition of salt to pure water reduces the specific free energy of the solution in comparison to that of pure water. The effect of this is a reduction in the "escaping tendency" of the moisture, and results in an increased ability to prevent loss of moisture through change of state (liquid to vapour) i.e. evaporation. This ability of a salt solution to retain, or attract, moisture can be demonstrated by separating pure water (solvent) from a salt solution with a semi-permeable membrane, fig 4.2. If the membrane is permeable to pure water but impermeable to salt molecules, the salt solution will exert a net attraction for the pure water. Flow will occur from solvent to solution until the head difference balances the osmotic attraction (suction) of the solute. In thermodynamic terms this is seen as a reduction in the partial vapour pressure, fig 4.3.

This reduction means that a greater energy is required to remove moisture from the main body of the soil, noting that a change in state from liquid to vapour must occur in the process.

Osmotic suction can be viewed as a separate suction component, acting broadly in addition to the matrix suction.

Osmotic effects may exist not only between the liquid and vapour phases at the air - water interface but also within the various states of water in the clay-water system. Marinho (1994) gave a simplified model of the possible osmotic forces within the clay-water system, fig 4.4.

4.1.4 Total suction

As noted earlier, matrix and osmotic suction can be viewed as broadly separate suction components. The total suction is thought to be the sum of the constituent parts and can be defined as follows:

\[ \psi = \frac{RT}{gm} \log_e \left( \frac{P}{P_o} \right) \]  

4.2

where

\[ \begin{align*}
\psi & = \text{universal gas constant} \\
T & = \text{absolute temperature} \\
g & = \text{acceleration due to gravity}
\end{align*} \]
Chapter 4  page 71

m = molecular weight of water
P/P₀ = relative humidity, where:
  P = partial vapour pressure
  P₀ = saturated vapour pressure

Burland and Ridley (1996) identified a concept offering a practical explanation of total suction, fig 4.5. Using this we can develop a visualisation of the individual components of suction. If we consider a closed system, fig 4.6(a) containing pure water and having a flat air-water interface, the partial vapour pressure will be equal to the saturated vapour pressure, hence the total suction will be zero. If a curved meniscus exists at the interface the partial vapour pressure will reduce, and the total suction will increase (it will be numerically equal to the matrix suction), fig 4.6(b). Alternatively, the addition of salt to the pure water would also cause a reduction in the partial vapour pressure, representing osmotic suction, fig 4.6(c).

The total suction is the sum of the effect of these components, Fig 4.6(d), and can be defined as:

\[ \psi = (u_a - u_w) + \pi = \left( \frac{RT}{gm} \right) \log_e \left( \frac{P}{P_0} \right) \]

4.3

Summary

Matrix suction (including both capillary and adsorption effects) is measured through direct contact with the pore fluid, in the liquid phase. Inherent in the measurement is the assumption that the salinity of the pore fluid does not affect the result. This is an acceptable proposition as most measurement techniques are sufficiently long in duration and offer no real impediment to the diffusion of salts.

Osmotic suction is the component related to the measurable effects of salinity on the overall suction. The direct measurement of osmotic suction \textit{in situ} has not been performed satisfactorily. Extraction of the pore fluid and subsequent measurement of the both salinity and its osmotic potential are the only available options. The key questions, which remain unanswered, are what effect the extraction process has on the composition of the pore fluid, and what the relationship is between the behaviour of the extracted pore fluid and that pertaining in the soil. There is some correlation in the published data, but it is far from conclusive.

The hypothesis that the measured total suction is equivalent to the sum of the matrix and osmotic components is \textit{probably} accurate. The experimental evidence is not available for a conclusive answer, however that which is, combined with the strong theoretical basis which exists allows this hypothesis to be broadly accepted.
4.2 INFLUENCE MATRIX AND OSMOTIC SUCTION HAVE ON EACH OTHER

Taking total suction as being the sum of osmotic and matrix suction we will consider the effect matrix and osmotic suction have on each other.

4.2.1 Matrix on osmotic

Matrix suction effects are confined to the tension in and at an air-water interface, and the transport of moisture without change of state.

As such, the state of tension within the pore fluid does not influence the effect or distribution of dissolved salts. Neither will it directly influence the relative escaping tendency (i.e. the reduction due solely to the presence of salts). Hence it can be seen that matrix suction does not affect the osmotic suction.

4.2.2 Osmotic on matrix

Here we must consider two conditions, one for inert granular materials and one for clay materials.

Firstly the granular material. The interaction between moisture and the soil grains is related to the pore size and the surface tension of the fluid through equation 4.1

The pore structure will be unaffected by the presence of salts. The type and concentration of the soil salts present will influence the surface tension. The effect is however minimal as can be seen from fig 4.7, where the maximum matrix suction is plotted against pore size for a pure water and for a 3.0 molar NaCl salt solution with an equivalent osmotic pressure of 15 MPa.

For a clay soil the effects of osmotic forces are different. The interaction between clay and water as a result of dissolved salts in the pore fluid will change permeability, compressibility, and packing of the clay soil but will not influence the matrix suction directly. However, the matrix suction could be influenced as a result of the changes to the soil brought about by the presence of salt.

4.3 SUCTION MEASUREMENT

The area of suction measurement has been the subject of much attention at Imperial College in recent years. Schreiner (1987) reviewed the components of suction, their measurement and influence on soil behaviour as part of his doctoral studies. Ridley (1993) undertook a comprehensive study on the
measurement of soil moisture suction. He developed an instrument (the Imperial College tensiometer) capable of measuring suctions quickly and directly to in excess of 1500 kPa (see section 4.3.4). Marinho (1994) extended some of the ideas identified by Ridley, in particular he discussed the mechanisms controlling soil suction and the concepts of cavitation in water and their influence on suction measurement. The research undertaken at Imperial College in relation to soil suction and its measurement has been used as a general background to this review and may not be explicitly referenced on all occasions.

Table 4.1 gives a summary of the most common measurement techniques, together with an indication of their application and range.

Reviews of suction measurement have been undertaken by many authors. The techniques are commonly grouped under headings relating to either the suction component measured (total, matrix or osmotic), or whether the technique makes a direct or indirect measurement.

Many instruments measure, or are capable of measuring more than one suction component and hence such headings are perhaps unsuitable.

The terms direct and indirect refer to whether the instrument actually measures the suction component itself or an intermediate parameter which can be related to suction through separate calibration or theoretical support.

Table 4.1 identifies the parameter measured by each instrument. It can be seen that tensiometers are the only instruments that record suction directly. One might also question the pressure plate and osmotic tensiometer as both measure a reduction in positive pressure resulting from pore water tension and not the tension itself.

Therefore the measurement techniques will be reviewed in the order in which they appear in table 4.1

4.3.1 Psychrometers

Psychrometers measure relative humidity of the atmosphere. Relative humidity is the relationship between the partial pressure of the water vapour in the air ($P/_{w}$), and the saturation vapour pressure ($P_{sat}$), at the same temperature. The relative humidity can be directly related to total suction as has been shown in equation 4.2

It is not practical to measure the partial pressures directly. However by comparing the temperatures of an evaporating (wet bulb) and non-evaporating (dry bulb) surface in the environment in question the RH can be defined.
Psychrometers allow measurement of the wet and dry bulb temperatures. These are based on the Seeback and Peltier effects.

In 1821 Seeback discovered that a circuit of two dissimilar metals will develop an electromotive force (EMF) if the two junctions of the materials are at different temperatures, fig 4.8.

Peltier (1834) discovered that a junction of two dissimilar metals will either increase or decrease in temperature when a current is passed (depending on direction), fig 4.9.

The instruments are calibrated over salt solutions of known concentration, and hence known relative humidity. Thus a relationship between generated EMF and total suction can be defined for each instrument.

By placing a soil sample in a sealed environment (with a psychrometer installed) and allowing sufficient time for the soil and air to reach equilibrium with each other the relative humidity of the sealed air space will reflect the total suction in the soil.

There is a need for good temperature control in order to achieve reliable measurements. This is due in part to the mechanism used for determining the RH, i.e. measuring the temperature of wet and dry surfaces, and in part with the need for equilibrium conditions to exist for realistic measurements to be made.

There are two types of psychrometer, thermocouple and transistor or thermistor psychrometers. They both operate on similar principles. Much of their respective development has taken place in parallel, with each having its own supporters.

i) Thermistor / transistor psychrometer

A thermistor is a resistor whose resistance decreases rapidly with increase in temperature. A matched pair of thermistors can be used as a psychrometer. A drop of water is placed on one whilst the other is left dry (wet and dry bulb). When exposed to the air in equilibrium with a soil sample, water evaporates from the wet bulb thermistor. The effect of the evaporation of moisture from the wet bulb thermistor is a suppression of its temperature, generating an EMF (in accordance with the Seeback effect).

The concept of this temperature depression is shown diagrammatically in fig 4.10. The dry bulb temperature is measured (t1), and the wet bulb temperature (t2) defined indirectly via the generated EMF. The maximum depression of the wet bulb temperature will occur when the partial vapour pressure is equal to the saturated vapour pressure. This occurs at the dew point (P), and will remain at this value until all moisture has evaporated, when the temperature will rise again. The temperature differential (t1 - t2) can
be determined from the recorded voltage. This will allow the equivalent saturated vapour pressure (P) to be calculated, hence defining the partial vapour pressure (P) at temperature t1. Together with the saturated vapour pressure (Po), defined from the dry bulb temperature, the relative humidity and hence total suction can be calculated.

The development and operation of these instruments is discussed in detail by Richards (1965). A typical instrument set up and response is given in fig 4.11.

At lower relative humidities moisture will evaporate more quickly. This reduces the duration that an instrument can continue to make a measurement. This is offset to some extent by a much quicker response, and shorter equilibrium time.

More recently, Richards and Peter (1987), introduced the use of matched transistors to replace the thermistors, resulting in a cheaper instrument. These are reviewed in detail by Woodburn et al (1993). The range of transistor psychrometers is between 100 kPa and 71 MPa and typical response times are measured in minutes to hours.

Research reported at the 1st International Unsaturated Soils Conference 1995, Truong and Holden, shows that these instruments exhibit hysteretic behaviour. It is suggested that monotonic measurements be made, increasing in suction each time.

ii) Thermocouple psychrometer

Spanner (1951) introduced the thermocouple psychrometer. Consisting of two thermocouples in one instrument, one measuring the ambient temperature (dry bulb) and the other used to define the wet bulb temperature. Fig 4.12 shows a typical thermocouple psychrometer, and response in both standard and dew point mode.

Fig 4.13 illustrates the thermodynamic principles of operation. By passing a current such that cooling occurs at the wet bulb junction, moisture will form on this junction. When the flow of current is stopped, an EMF will be generated, as noted by Seeback. The junction is initially cooled beyond the equivalent vapour pressure (P) temperature. As the junction warms up the EMF will reduce. When the temperature of the cool junction reaches, P, the rate of warming up will slow as evaporation occurs. This is seen as a plateau in the rate of reduction of EMF, as shown in fig 4.12ii (a). This identifies t2. t1 is measured by the dry bulb thermocouple, allowing P and Po, hence relative humidity to be identified.

This plateau becomes more pronounced at higher suction, as the effect of the evaporation is more pronounced.
The overall working range of these instruments is 100-8000 kPa, although a number of factors serve to reduce this range. At lower values the voltage plateau may be indistinct, < 500 kPa, Ridley (1993). Furthermore the effects of variance in the ambient temperature are magnified at lower suction, <1000 kPa, and can cause a variability in output above 4000 kPa.

A secondary effect of the initial cooling of the wet bulb junction is that a corresponding heating of the reference dry junction occurs as a result of the current. For an accurate estimation of the RH the dry bulb must be held at the ambient temperature. This issue is also of concern as the efficiency of the thermocouple itself is temperature dependent. Hence calibrations should be carried out at all temperatures for which measurement are to be made.

One of the main advantages that the thermocouple psychrometers have over the thermistor / transistor psychrometers is that they can generate moisture on the wet bulb through the Peltier effect. The moisture must be placed and replaced on thermistor / transistor psychrometers.

iii) Dew point hygrometer

A relatively new application of the thermocouple psychrometer is that of the dew point hygrometer.

This concept is based on the precept that at the dew point, the junction will neither gain or lose water. If this point can be held, the output can be related to the RH at the ambient temperature. By knowing the cooling coefficient of the instrument it can be cooled as necessary to reach and maintain the dew point.

The cooling coefficient is generally given by the manufacturer, as is the temperature at which it was determined, allowing the coefficient to be adjusted to reflect the temperature under which it is being used. A typical output is shown in fig 4.12ii (b).

There are two distinct advantages of the dew point hygrometer over the standard psychrometer:

1) The output voltage is much larger
2) The output is stable for a considerable period of time

Wescor market thermocouple psychrometers which can be readily used in both modes in a single instrument.
Summary

Richards and Peter (1987) made a comparison of the two types of instrument, noting the advantages and disadvantages of each. This is a good indication of the similarity between the instruments and the basic differences.

Advantages

Transistor / thermistor psychrometers have a larger range than thermocouple psychrometers (0.1 - 71 MPa as opposed to 0.1 - 8 MPa).

Thermocouple psychrometers can make measurements in two different modes (standard and dew point)

Thermocouple psychrometers can generate the necessary moisture for wet bulb measurements in-situ

Disadvantages

Transistor / thermistor psychrometers need the moisture for the wet bulb to be placed and replenished manually. Also the relatively large drop size can add a significant amount of vapour to the measuring environment.

The thermocouple type can suffer from ageing and corrosion of the junctions.

4.3.2 Filter paper method

The use of an absorbent material such as filter paper to determine the suction in a soil sample is well documented. Relationships between the filter paper moisture content and suction for various types of filter paper have been defined by numerous authors.

Once a relationship has been defined, the suction can be determined directly, by bringing a filter paper into suction equilibrium with the soil sample and subsequently measuring the filter paper moisture content.

The principle of operation is that if an absorbent material is placed in intimate contact with a soil sample, moisture will be transferred (commonly from soil to filter paper) until both media exert a common attraction for the available moisture. Furthermore, it is assumed, that if the sample is large, relative to the filter paper, the effect of the moisture transfer on the suction existing in the soil will be negligible.

Used initially in the field of soil science by Gardner (1937), the technique has gained wider acceptance
in the field of soil mechanics since the 1960's.

With the filter paper in contact, the passage of soil salts is not impeded, moisture flow occurs in the liquid phase, and the matrix suction of the sample will be determined.

The total suction of the sample can be measured if the filter paper is brought to a state of suction equilibrium with the soil without direct contact between filter paper and soil sample. This is commonly achieved by separating the paper and sample by a fine wire mesh, and enclosing both in a sealed environment. In this situation the filter paper will absorb moisture from the sample through the vapour phase, coming to equilibrium with the partial vapour pressure of the sealed environment, which in turn is controlled by, and in equilibrium with, the soil sample.

There have been a number of comprehensive reviews of the application and calibration of the filter paper method, and it is not intended to repeat these here. The salient points will be outlined and discussed. The reader is referred to the reviews by Al-Khafaf and Hanks (1974), Ridley (1993), Fredlund & Rahardjo (1993), Marinho (1994), Houston et al (1994) and Ridley and Wray (1995) for comprehensive reviews of the technique and its application.

Types of filter paper

Numerous types of filter paper and filter membranes have been used, Sibley and Williams (1990). Two types of filter paper dominate, Whatman 42 and Schleicher and Schuell 589. Recently, Miller and Nelson (1992) introduced a further paper, Thomas Scientific No. 4705-F10.

Calibration

These have been performed by a combination of methods, depending on the suction level under consideration, table 4.2.

Generally under 1500 kPa contact methods are used, and above this non-contact methods. These limits reflect the abilities of the measuring instruments for the contact range and the sensitivities for the non-contact methods.

There have been many calibrations published for both types of filter paper, Ridley (1993) summarised them, table 4.3.

The most widely accepted calibrations at present are the Chandler & Gutierrez (1986) for the Whatman No. 42 and the Greacen, Walker & Cook (1987) for the Schleicher and Schuell no. 589.
Total suction measurements

Until recently it has been assumed that a single calibration was acceptable for both in contact (matrix) and out of contact (total) measurements. El-Ehwany and Houston (1990) first identified that different calibrations were necessary. They also noted that the total suction calibration was far less sensitive in the low suction range. Houston et al (1994) and Lee and Wray (1995) have noted the deviation from the in-contact calibration for total suction measurements at low suctions, (below 800 kPa).

Gourley and Schreiner (1995) gave data for a comprehensive calibration of a field probe using filter paper to measure total suction. They suspended filter papers above known salt solutions in sealed containers whose size and shape reflected the proposed field application. Tests were undertaken investigating the effects of varying temperature, equilibrium time and suction. They developed a complex expression, equation 4.4, to calculate the total suction for any given temperature, time and filter paper water content, within certain limits.

\[
P_k = 10^{5.82 - \frac{T}{15} - 0.096W_p} \left( \frac{2.2 \times 10^6}{\sqrt{T - 3.87}} \right) + \frac{500}{t} + \frac{\sqrt{t}}{82} + \frac{\sqrt{\sqrt{t}}}{82} + \frac{\sqrt{\sqrt{\sqrt{t}}}}{82}
\]  

where \( P_k \) = total suction  
\( W_p \) = filter paper water content (%)  
\( t \) = equilibrium period (days)  
\( T \) = temperature (°C)

This calibration can only be applied to suction measurements made under comparable experimental conditions. Any variance from this set up would require a fresh calibration. It was not the intention of the authors to develop a replacement for the Chandler et al calibrations, (1986 and 1992), merely to define a specific calibration for a particular application. They note that the total suction calibrations in the low range (<3 MPa) use contact methods, and should be reassessed under non contact conditions.

It is often assumed that suction becomes more sensitive to changes in RH as the actual suction level reduces. The relationship between suction and RH is plotted in fig 4.14, where it can be seen that there is no change in sensitivity in the low suction range. Furthermore, even significant changes in the ambient temperature have only a small effect on the relationship. However during measurement of suctions below say, 1000 kPa, the partial vapour pressure \( P \) lies very close to the saturated vapour pressure line, \( P_o \), fig 4.13. A very small reduction in ambient temperature could cause the partial vapour pressure to reach the saturated vapour pressure line resulting in moisture condensing onto the filter paper. This would have a significant effect on the filter paper moisture content and hence measured suction. This was noted by Houston et al (1995). The reduction in temperature required to cause condensation has been calculated, table 4.4, and it can be seen that at low total suctions it is likely that some condensation may occur during
the measurement. The time for subsequent evaporation to occur may be significant under these near equilibrium conditions.

Fig 4.15, after Gourley and Schreiner (1995), shows data from filter paper suction measurements at 3, 5, 14 & 29 days for a number of known total suctions (salt solutions). The approach to a final equilibrium value is asymptotic in all cases. We can highlight the point at which each filter paper has a suction 1000 kPa and 500 kPa in excess of the final value. There is a general trend seen in both cases with the time for equilibrium increasing as the lower suction values.

The numerical error, after a given time has been allowed for equilibrium, is more significant at low suctions. This error is exacerbated at low suctions when expressed on a percentage basis of the final value. It appears that at low suctions, a significant increase in the equilibrium time is required for an accurate measurement of total suction.

We should remember however that the filter paper is least sensitive to moisture content change at these low suctions. This will assist to an extent in negating the errors caused by possible condensation, however only at low suctions.

Ridley (1995) noted that one of the fundamental aspects of defining a calibration for use with the filter paper was to ensure that the conditions under which the calibration was performed were identical to those under which the subsequent measurements are made.

**Hysteresis**

All porous media will display hysteresis between wetting and drying. There are two contributors to this effect, the angle of wetting and the pore structure. The contact angle between a liquid and a solid surface varies depending on whether the liquid is being added or removed. The pore structure consists of interconnected pores of various sizes and the controlling pore size for drying may be different from that for subsequent wetting. This is readily seen in the moisture characteristic curve for soils, and is discussed more fully in chapter 2. These effects are equally applicable to other porous media such as a filter paper as well as for a soil.

The existing filter paper calibration curves are for dry papers undergoing wetting up. This is perfectly acceptable as most uses are for single measurements, or perhaps the development of a soil moisture characteristic curve from a wet to a dry state.

If however it is desired to establish the wetting up phase of a soil sample the situation is somewhat different. Firstly a method of introducing a small amount of moisture, preferably under controlled
conditions is required. Secondly a measurement is needed of the resultant sample suction.

If moisture is added to the sample and a dry filter paper is subsequently used, two potential problems occur:

Firstly, as the moisture is introduced from the surface, one must wait until this moisture has equilibrated throughout the sample (a condition that is difficult if not impossible to define) before placing the filter paper. If equilibrium conditions do not exist, lower suctions will be apparent at the surface than in the centre of the sample, when the dry filter paper is placed. Moisture will initially be absorbed by the filter paper, and may subsequently be lost as the suction at the surface of the sample increases as equilibrium develops throughout the sample. Thus the filter paper calibration determined by wetting up dry papers may not be applicable.

A further problem is that even if suction equilibrium is attained throughout the sample prior to placing of the filter paper, the effect of adding a dry filter paper will be to reverse the movement of moisture within the sample. Moisture will initially travel from the surface to the centre of the sample, during wetting. When a dry filter paper is placed the direction of flow will be reversed as the filter paper absorbs moisture from the sample. The suction in the sample may increase by a small amount as a result, and a reduction in the observed hysteric behaviour of the soil may result, as the soil moisture characteristic wetting path will now include a small aspect of drying, fig 4.16.

Ideally the filter paper should be used to both wet the sample and make the suction measurement. By placing a wet filter paper in contact with the sample this is possible. A small amount of moisture will be added to the sample, and the equilibrium filter paper moisture content can be used to determine the sample suction, moisture moving in a monotonic sense throughout.

Clearly however, the calibration for dry filter papers is not applicable. Ridley (1995) recognised this, and published a limited calibration for wet filter papers undergoing drying, fig 4.17. Wet filter papers were placed in a pressure plate apparatus and brought to known suctions, by increasing the ambient air pressure, for a period of 7 days. They were then removed and the moisture content determined. The calibration could then be applied to the wetting up of soil samples using initially wet filter papers. As would be expected the calibration for wetting up filter papers was different from that for drying out. The calibration shown includes experimental data from this study, which is discussed in more detail in section 8.3.1

4.3.3. Suction plate and pressure plate (axis translation technique)

The background to this technique is discussed in more detail in chapter 5. Its use for measurement of
suction only will be discussed here.

A suction plate consists of a mercury manometer connected to the base porous ceramic of the measuring chamber. When the sample is placed, the manometer will measure a suction directly by depression of the level in the manometer, fig 4.18. The system is limited to suctions not exceeding 100 kPa, thought to be due to the inability of water to sustain higher tensions. Schofield (1935) proposed raising the ambient air pressure to increase the working range of such apparatus.

Hilf (1956) developed a system in which he demonstrated that an increase in the ambient air pressure caused an equivalent increase in the pore water pressure. Thus by raising the air pressure, the pore water pressure could be brought into the positive range measured by standard pore pressure systems. The limitation of the system was the air entry value of the porous stone separating the sample chamber from the pore water measuring system. This barrier is necessary to prevent the raised air pressure from affecting the pore water measurements.

As the process is one of moving the axis used for measuring the pore pressure into the positive range it is termed the "axis translation technique". A typical axis translation system, for a pressure plate, is shown in fig 4.19.

It consists of a sealed sample chamber containing the soil sample, separated from the pore pressure measuring system by a high air entry porous ceramic.

In order to measure the suction in a particular sample an air pressure, \( u_a \) (greater than the suction in the sample) is applied to the cell, raising the pore pressure, \( u_w \), into the positive range. This is performed under undrained conditions. The suction is the difference between the air and water pressures at equilibrium:

\[
\text{Suction} = u_a - u_w
\]  

4.5

In practice there will be some exchange of moisture in order for a measurement to be registered, which will cause a change in the actual sample suction. The direction of flow however, can be controlled. If the applied air pressure is in excess of the actual suction, moisture will flow from the sample to the pwp system, fig 4.20, causing an increase in suction. Alternatively, if after the initial application of air pressure, the pwp system is vented to atmosphere momentarily, equilibrium will be achieved by the sample taking up moisture from pwp system to sample, fig 4.21, reducing the actual suction. The general assumption is that the uptake of moisture is small in comparison to the sample size and the suction change due to the measurement technique is negligible.
Limitations of the axis translation technique

The limitations of the axis translation technique are discussed in more detail in section 5.1.1 but can be summarised here.

Equipment

- Maximum suction is limited to the air entry value of the porous media. Commercially available ceramics have a maximum air entry value of 15 bar. An alternative is to use a cellulose semi-permeable membrane, the AEV is thought to be in excess of 100 bar.

- Diffusion of air can occur below AEV, slowing response and introducing errors in the measurement.

Materials

- The air phase must be continuous, in order that the applied, elevated air pressure does not cause volume changes to occur. Sample volume change will cause an over estimation of the suction to be recorded. A degree of saturation less than 80 % is thought to be necessary to ensure a continuous air phase.

4.3.4. Tensiometers

Tensiometers are most commonly used for measuring matrix suction, although it should be possible to use them to measure total suction also. There are a number of variations, all of which are based on common principles.

Tensiometers measure the tensile stress in the pore water by bringing water in the measuring instrument into suction equilibrium through direct contact.

There are three different types of tensiometer, one for field use (with laboratory applications) and two for the laboratory use (with field applications).

The principle of a tensiometer is shown schematically in fig 4.22. It consists of a reservoir of water with a measuring instrument at one face and a porous stone (in intimate contact with the soil) at the other.

Under equilibrium conditions both the instrument and the soil will exert equal attraction for the available moisture, thus by measuring the tensile stress in the water in the tensiometer, the tension, or suction, in the soil is defined.
The main limitation of tensiometers are that they cannot measure suction beyond 100 kPa. It is a widely held belief that this is due to the inability of water to sustain tensions in excess of this without cavitation occurring. This limit is further reduced in field applications, as the water column must extend from the point of contact with the soil (often several metres below ground level) to the point of measurement (ground level). This reduces the maximum measured suction by 10 kPa for each metre head of water.

Stannard (1992) presented a thorough review of the "theory, construction and use" of tensiometers. He identified three methods of measuring the tension in the instrument, vacuum gauge, mercury reservoir and transducer, fig 4.23.

Oberg (1995) developed a transducer based system which makes measurements local to the porous tip. Tadepalli and Fredlund (1991) used a tensiometer with a flexible, fine bore tube and porous tip for measuring suctions during oedometer testing in the laboratory. The 100 kPa suction limit still applies to both applications.

Ridley and Wray (1995) stated that the response of the instruments is directly influenced by:

- The sensitivity of the measuring system
- The volume of water in the system
- Permeability of the porous filter
- Amount of undissolved air in the system

i) Osmotic tensiometer

Working on the assumption that water cannot sustain high tensile stresses, Peck and Rabbidge (1969) enhanced the range of measurement for tensiometer by developing an osmotic tensiometer, fig 4.24. The instrument was used primarily in laboratory applications.

The water in the instrument is replaced with an aqueous solution of Polyethylene glycol (PEG). A membrane permeable to water but not to the PEG molecules was placed on the inside face of the 15 bar porous stone, retaining the PEG solution. When placed in water a large positive stress is generated in the tensiometer as water is drawn in by a process of osmosis. At equilibrium the positive stress balances the osmotic potential of the PEG solution. This represents the initial reference pressure.

When subsequently placed against a soil sample, a reduction in the (positive) reference pressure occurs due to the samples' affinity for water. The reduction in the reference pressure is equal to the suction in the sample. This process of transferring the initial pressure into the positive range overcomes the traditional limitations of tensiometers, as all measurements are positive. The suction measurements are
of matrix suction as the membrane is permeable to the soil salts but not to the large PEG salts.

The reference pressure is commonly in the range of 1500 - 2000 kPa, therefore suctions up to this value can be measured.

A number of difficulties are associated with the use of the osmotic tensiometer, Peck and Rabbidge (1969) and Bocking and Fredlund (1979).

- The tensiometer is filled with de-aired water during assembly to minimise the risk of entrapped air. The aqueous solution of PEG is subsequently injected through a filling port to displace the water. Mixing and dilution between the water and PEG solution will occur, resulting in a reduced osmotic potential. The extent of this reduction is undefined.

- Long term drift of the reference pressure occurred, fig 4.25. A number of possible causes were identified:
  - Diffusion into the "O" ring seals
  - Flaws in, or damage to, the membrane
  - Drift of strain gauges
  - Leakage through the membrane
  - Breakdown of solute
  - Breakdown of membrane

It is unclear which are the main causes, and how to resolve them. The most likely are associated with the performance of the membrane and its sealing. Interestingly, neither authors thought it necessary to incorporate any form of inhibitor (antibiotic) against biological degradation of the membrane, although both noted the possibility of such an action occurring.

Also the reference pressure was affected by the ambient temperature. Bocking and Fredlund (1979) quantified this at approximately 20 kPa per °C. They noted significant scatter although a linear, non-hysteric trend could be justified.

ii) Imperial College tensiometer

The underlying assumption in all tensiometer practice is that water cannot sustain a tension in excess of 100 kPa.

Stannard (1992) stated that at 15 °C pure water would vaporise (cavitate) when the tension exceeded 9.7m
head of water. This was further reduced to 8.7 m to account for impurities and undissolved air. This derivation is made from the vapour pressure curve for water, fig 4.26.

This is an acceptable hypothesis for water with a free surface. However if the free surface is a curved menisci, the surface tensile forces present will enable this boundary to be crossed without change of state. Furthermore if a free surface is not apparent, the tensile force that can be sustained is significantly higher.

Tabor (1992) demonstrated that, theoretically, water could sustain tensile stresses as high as 500 MPa. Physical measurements of the tensile strength of water have been made by numerous authors, Marinho (1994), and stresses of up to 17 MPa have been recorded.

Standing (1991) observed the ability of pore pressure transducers to measure high suctions. Using a mid-height pore pressure probe during triaxial stress path tests, he noticed that upon undrained unloading, suctions of up to 400 kPa were recorded. These suctions were sustained for up to two hours before tension breakdown occurred.

Following up Standings observation, Ridley (1993) developed a miniature tensiometer based on a commercial pore pressure transducer. It was capable of measuring suction up to 1500 kPa.

The key elements that allow the successful measurement of high suctions are the use of a high air entry ceramic filter (15 bar), minimising the volume of water in the system and saturating under very high water pressures prior to use.

A modified version was introduced, Ridley and Burland (1995), fig 4.27. This new instrument incorporates an integral strain gauged diaphragm, slightly larger porous ceramic (still 15 bar air entry value), and a reduced water volume.

The response of the instrument is quick, fig 4.28, often within minutes. The coefficient of permeability for the porous ceramic is $2.59 \times 10^{-11}$ m/s. In many situations this will control the response, however for some expansive clays, and many partly saturated soils, the material permeability may be lower and this will control the rate of response of the instrument.

Recently the instrument has been used successfully to make field measurements and continuous measurements lasting several weeks have been made in both laboratory and field situations.

The fundamentals of enabling pore pressure measuring systems to sustain large tensile stresses has been studied by Ridley (1993) and Marinho (1994). Both identify a number of possible causes of tension breakdown. These can be related to one of three phenomena:
Cavitation. The formation of bubbles within the fluid.

- Loss of adhesion between fluid and instrument surface.

- Air entry through the porous ceramic.

**Cavitation**

There are a number of hypotheses as to the instigation of cavitation. It is unlikely that cavitation within the body of the pure liquid causes the tension breakdown that is observed during experimental work. Harvey et al (1944) developed a crevice model to explain the inconsistencies between the theoretical and measured values of tensile strength. This states that the expansion of pre-existing bubbles or loss of adhesion is the prime cause.

Harvey et al (1944) also recognised that the application of a high initial positive pressure was useful in stabilising the existing air in the instrument. Fig 4.29 shows the effects of pre-pressurisation on air contained within the microscopic crevices on the inside of a tensiometer. The tensiometer which has not been subjected to pre-presurisation will release air (in the form of an expanding bubble) from its crevices when the tension exceeds atmospheric pressure, as this is in effect, the pre-pressurisation the body of fluid has received.

By applying a high initial pressure, the air is suppressed into the crevice, creating a high matrix suction component at the air-water interface, remembering that:

This must be overcome in order to extract the air, and if the applied positive pressure is greater than the subsequent measured negative pressure, the air should not be removed. Air will come out of solution at the free water surface (atmospheric conditions) and not at this air-water interface.

**Loss of adhesion**

It is difficult to separate, experimentally, the loss of adhesion from the cavitation effect. It is likely that they are intrinsically linked, and the overall effect only, can be observed. Scrupulous cleaning should be undertaken to reduce the possibility of debris remaining and causing a tension breakdown. Pre-pressurisation will also help in reducing the possibility of bubbles forming in contact with any microscopic debris should it be present.
Air entry through the porous stone

Porous ceramics are generally classified in terms of their air entry value (AEV). When saturated, porous stones can resist the passage of air due to the tensile stresses developed in the air-water interface within the porous structure. The air pressure that can be resisted is directly related to the pore size of the ceramic through the equation describing matrix suction, equation \[ \text{AEV} \].

Air can pass through porous media in two ways. Firstly where the pressure differential is greater than the AEV of the media. Secondly air can pass in solution, from the high pressure side to the low pressure side, coming out of solution once on the low pressure side.

When a measurement is made moisture moves from the tensiometer to the sample as the sample has a greater initial suction. Therefore the sample is the low pressure side, hence air can only pass in solution from porous stone to sample in this particular situation.

Air could however be drawn through the porous ceramic if the suction exceeds the AEV, fig 4.30. When a measurement is made, air will be apparent at the interface between ceramic and sample. Both sample and tensiometer will exert an attraction for the available moisture. It is likely that flow between the two will occur as film flow across the surface as opposed to mass flow through the pores at this interface. The pressure differential at the ceramic interface will exist between the air (at atmospheric pressure) and the water held within the ceramic (at the value of the suction in the sample).

This situation is no different than if the water was under atmospheric conditions and the air pressure raised to the AEV of the ceramic. Air will pass through the ceramic without change of phase irrespective of whether it is pulled (tension) or pushed (compression). Tension breakdown will result once this condition is reached.

Ridley sought clarification that this hypothesis was the limiting factor in tensiometer performance. Using I.C. tensiometers he fitted them with commercially available porous ceramics having AEVs of 1, 5 and 15 bar. Following pre-pressurisation (saturation) at 4000 kPa he determined the point of tension breakdown for each by allowing the moisture to evaporate from the surface of the ceramic. Tension breakdown occurred at 170, 720 and 1800 kPa for the 1, 5 and 15 bar ceramics respectively. This confirms the hypothesis that air drawn through the porous ceramic causes tension breakdown, and that this will occur at roughly the AEV of the material.

4.3.5. Electrical conductivity (porous blocks)

The principle is that the electrical conductivity of a porous material will vary depending on the moisture
content of the material. The effective working range is related to the degree of saturation of the media. It is necessary for the degree of saturation to change for a measurable change in electrical conductivity, hence the minimum measurable suction is related to the point of desaturation, and the maximum when a residual degree of saturation is reached. When placed in contact with a soil sample, moisture will flow either into or out of the porous block until the suction in both soil and block are in equilibrium. The electrical resistance of the porous media can then be measured via two electrical contacts embedded in the porous block, fig 4.31.

Gypsum, in the form of plaster of Paris, is the most common porous material used. Research on the use of such devices was reported by Bouyoucos and Mick (1940). They identified the ideal attributes of the porous media to be:

- A hard, porous structure capable of rapid absorption and rapid drying with regard to water
- A large absorption capacity to ensure a useful order of sensitivity
- A moderate range of solubility to minimise the effects of changes in the soil salinity

Determination of the moisture characteristic curve for the porous media is generally performed in a pressure plate apparatus with conductivity measurements being made in place of moisture content. A relationship can therefore be formed between the electrical resistance of the porous block and suction, fig 4.32.

Studies were made of the equilibrium times, and the effects of temperature, Aitchison, Butler and Gurr (1951), and salinity, Aitchison and Butler (1951). In 1965, Aitchison and Richards summarised the concepts of porous blocks and noted a number of advantages and disadvantages:

Advantages
- Sensitive over a wide range of suctions (30 - 1500 kPa)
- Economical, in terms of material costs, field installation and measurements.
- Good response to a change of suction

Disadvantages
- Significant hysteresis between wetting and drying cycles.
  The suggestion for overcoming this problem is that the instrument be used over a small part of its range, and calibrated over this working range.
- Absolute suction values may be poorly defined, as time to equilibrium can be lengthy (noting that initial response is good however)
- Accuracy seriously affected by soil salts. Resulting errors can be significant

Under ideal conditions porous blocks provide a useful indication of matrix suction in the field. Their
successful application is however quite restricted.

4.3.6. Thermal conductivity sensors

The thermal conductivity of a porous medium varies with degree of saturation. As water is a much better thermal conductor than air, heat dissipation increases with degree of saturation.

Therefore, for a unit generation of heat, dissipation will occur more efficiently at higher degrees of saturation. The heat that is not dissipated results in a temperature increase in the porous medium. It is this temperature change that is measured, commonly as a voltage output.

In practice a probe, fig 4.33 (Sattler and Fredlund, 1989), is placed in contact with a soil sample and allowed to come to equilibrium from either an initially wet or dry state.

Once equilibrium has been reached between the soil and sensor, a controlled amount of heat is generated within the central core of the sensor. After a specified period of time a measure is made of the residual increase in temperature.

There have been two commercially available sensors, the MCS 6000 from Moisture Control Systems, and the AGWA-II from Agwatronics Inc. Both are no longer available.

Fredlund and Rahardjo (1993) have been involved in much recent research into the use and limitations of such instruments, (over 100 AGWA-II instruments have been calibrated and used at the University of Saskatchewan, Canada).

They (Fredlund and Rahardjo, 1993) report much data on the calibration and use of such instruments which can be summarised as follows:

i) The calibration for such instruments is bilinear with a change in gradient occurring at about 175 kPa. Above this threshold the response is slower and less sensitive.

ii) The response time of the instruments increases with increasing suction. (during calibration using a pressure plate, equilibrium times approached 200 hours at 300 kPa. (AGWA-II)

iii) Measurements on a variety of laboratory samples, from silts to plastic clays showed a great variance in equilibrium times. Equilibrium times of several weeks were common.

iv) Initially wet sensors have a much increased equilibrium time compared to initially dry sensors.
v) Positive pore pressures could allow water to come into contact with the electronics in the central core causing the instrument to fail.

vi) Poor contact between instrument and soil causes erroneous results, much higher values of suction being recorded than are actually present.

vii) Thermal conductivity sensors can be used *in situ*. Despite fairly long equalisation times they could be used to monitor seasonal changes in field suctions.

A two point "linear" calibration is provided by the manufacturer for each instrument. A more rigorous calibration of the AGWA sensors was undertaken by Fredlund and Wong (1989) and Sattler and Fredlund (1989), fig 4.34. They installed several sensors in a soil paste and subjected them to monotonic increases in suction in a pressure plate apparatus. This calibration was then applied to both initially wet and initially dry sensors. It makes the implicit assumption that there is no hysteresis in the behaviour of the porous medium.

However the data provided by Sattler and Fredlund (1989) does not fully support this, with many instances of initially wet sensors giving lower suctions than initially dry sensors. One would expect a different calibration as moisture ingress and egress from a porous medium will display a marked hysteresis.

### 4.3.7. Osmotic suction measurement

As defined earlier, (section 4.1.3), the osmotic component of soil suction is that related to the effects of the soil salts on the retention of soil moisture. The most convenient method for determination of osmotic suction indirectly is to measure the total and matrix components separately, defining the osmotic suction as follows:

\[
\pi = \psi - (\mu_g - \mu_w)
\]

where \(\pi\) = osmotic suction

\(\psi\) = total suction

\((\mu_g - \mu_w)\) = matrix suction

This statement is widely accepted, and is reinforced by thermodynamic theory. However in order to confirm this experimentally, it is necessary to measure each component independently. Osmotic suction cannot be readily determined from an intact soil sample, saturated or partly saturated. A sample of the moisture held within the soil must be extracted for this purpose.
There are two methods for obtaining the pore fluid extract used for electrical conductivity measurements, squeezing and leaching.

The two methods of osmotic suction measurement differ in the extraction method. Both use electrical conductivity for the actual measurement.

The electrical conductivity of water varies with the salt content. An increase in salt content corresponds with an increase in conductivity. A detailed study into the effects of salinity on the agricultural use of soils was published by the United States Dept. of Agriculture (1969). The work gives relationships between electrical conductivity and osmotic pressure for various concentrations of different soil salts, also giving an average relationship for a combination of soil salt types, fig 4.35. These results can be used to assess both the soluble salt content and the osmotic component of the soil.

i) **Squeezing technique**

A sample of the pore fluid is extracted by consolidating the soil sample in a steel cylinder to overcome any *in situ* suctions. One way drainage occurs and the soil moisture is in effect squeezed from the sample, into a syringe connected to the base porous stone.

This method is said (Fredlund and Rahardjo, 1993) to work better when the sample has a high initial water content and hence low matrix suction. The pressure required for extraction is then lower. It is felt that the extraction pressure can affect the composition of the extract.

ii) **Leaching or saturation extraction technique**

There are two methods of obtaining the saturation extract. One involves obtaining a sample of soil moisture by filtration of a prepared soil slurry. The other uses the soil slurry directly.

The saturation extract is obtained by mixing a known quantity of dry soil with water at ratios of 1:1, 1:2, or 1:5 (soil:water). The moisture is drawn off by filtration and its conductivity measured.

The slurry method (saturated paste) uses a slurry of soil prepared at the saturated moisture content. The conductivity of the paste is measured.
Both the squeezing and leaching techniques make the assumption that the extracted pore fluid is identical in composition to that of the *insitu* pore fluid. Back calculation of the salt concentration and the osmotic potential of the soil moisture under initial conditions is necessary in both instances.

Schreiner (1987) reviews both methods. He notes that in the squeezing method the consolidation pressure would have the effect of stressing the double layers causing expulsion of water to the free water zone of the sample. However as the cations cannot be simply expelled from the double layer, the effect will be one of dilution of the salt concentration of the free water and hence the extract. This effect will be exacerbated by samples with high initial matrix suctions, i.e. low water contents and degree of saturation. This effect will also be more pronounced where the osmotic suction is high, particularly with a low water content.

Schreiner also notes that using the saturation extract will result in a solution that reflects the salt concentration of the free water only and not of the soil as a whole. Again this is due to the fact that salts are inherent in both the double layer and the free water of a soil, but the sample will only reflect the removable salts, i.e. those of the free water. However when extrapolating to the required water content of the soil, the calculation involves all the water, free and double layer, that is released by oven drying, hence a possible cause of the overestimation noted by Krahn and Fredlund (1972)

The data of Krahn and Fredlund (1972) fig 4.36, supports the trend of equation 4.6 however the data is far from conclusive. Indeed it would appear to conflict with the hypotheses of Schreiner (1987). The results showed good agreement for the plastic clay, but far were less conclusive for the low plasticity till (the opposite trend to that suggested by Schreiner). Fredlund and Rahardjo (1993) suggest that the errors are due to inaccuracies in the original measurement of matrix suction. This is reflected in the original publication (Krahn and Fredlund 1972) by a statement that the matrix suction measurements for the till were at the limits of the equipment (pressure plate). The data however shows that the measurements for the till were in fact lower than those for the more plastic clay. Some confusion may be generated by incorrect labelling of the figures in the original paper!

In summary, the relationship given in equation 4.6 is probably reasonable. However it has not yet been possible to substantiate this experimentally with any real degree of confidence or repeatability. The difficulty in proving this principle lies with the measurement of osmotic suction. The measurement of osmotic suction requires that a sample of the soil moisture be removed from the sample, and it is clear that this soil water sample may differ from that *in situ*.
<table>
<thead>
<tr>
<th>Device</th>
<th>Suction</th>
<th>Parameter measured</th>
<th>Range (KpA)</th>
<th>Equilibrium Time</th>
<th>Lab / Field</th>
</tr>
</thead>
<tbody>
<tr>
<td>transistor psychrometer</td>
<td>total</td>
<td>relative humidity</td>
<td>100 - 71,000</td>
<td>minutes</td>
<td>lab, field?</td>
</tr>
<tr>
<td>thermocouple psychrometer</td>
<td>total</td>
<td>relative humidity</td>
<td>100 - 8,000</td>
<td>minutes</td>
<td>lab, field?</td>
</tr>
<tr>
<td>filter paper (non contact)</td>
<td>total</td>
<td>moisture content</td>
<td>400 - 30,000</td>
<td>7-14 days +</td>
<td>lab, field</td>
</tr>
<tr>
<td>filter paper (contact)</td>
<td>matrix</td>
<td>moisture content</td>
<td>30 - 30,000</td>
<td>7 days</td>
<td>lab, field</td>
</tr>
<tr>
<td>suction plate</td>
<td>matrix</td>
<td>pore water tension</td>
<td>0 - 90</td>
<td>hours</td>
<td>lab</td>
</tr>
<tr>
<td>pressure plate / membrane</td>
<td>matrix</td>
<td>effects of pore water tension</td>
<td>0 - 10,000</td>
<td>hours</td>
<td>lab</td>
</tr>
<tr>
<td>tensiometer</td>
<td>matrix</td>
<td>pore water tension</td>
<td>0 - 90</td>
<td>min</td>
<td>lab / field</td>
</tr>
<tr>
<td>osmotic tensiometer</td>
<td>matrix</td>
<td>effects of pore water tension</td>
<td>0 - 1,500</td>
<td>hours</td>
<td>lab / field</td>
</tr>
<tr>
<td>Imperial College tensiometer</td>
<td>matrix</td>
<td>pore water tension</td>
<td>0 - 1,500</td>
<td>min</td>
<td>lab / field</td>
</tr>
<tr>
<td>porous block</td>
<td>matrix</td>
<td>electrical resistance</td>
<td>30 - 3,000</td>
<td>weeks</td>
<td>lab, field</td>
</tr>
<tr>
<td>thermal conductivity</td>
<td>matrix</td>
<td>conductivity</td>
<td>0 - 300</td>
<td>weeks</td>
<td>lab, field</td>
</tr>
<tr>
<td>squeezing</td>
<td>osmotic</td>
<td>conductivity</td>
<td>30 - 3,000</td>
<td>hours</td>
<td>lab</td>
</tr>
<tr>
<td>leaching</td>
<td>osmotic</td>
<td>conductivity</td>
<td>30 - 3,000</td>
<td>hours</td>
<td>lab</td>
</tr>
</tbody>
</table>

Table 4.1 summary of common measurement techniques (after Ridley 1993)
<table>
<thead>
<tr>
<th>TECHNIQUE</th>
<th>CALIBRATION RANGE</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Suction plate (contact)</td>
<td>0 - 1 atm</td>
<td>Direct suction measurement</td>
</tr>
<tr>
<td>Field samples (contact)</td>
<td>0 - 2 MPa</td>
<td>Using plastic soils which unload undrained from their in situ mean effective stress. Suction will be equal to mean effective stress</td>
</tr>
<tr>
<td>Oedometer samples (contact)</td>
<td>0.1 - 2 MPa</td>
<td>Unloading undrained leaves sample suction equivalent to consolidation stress</td>
</tr>
<tr>
<td>Pressure plate (contact)</td>
<td>0 - 1.5 MPa</td>
<td>Axis translation technique</td>
</tr>
<tr>
<td>Desiccator (non-contact)</td>
<td>0.5 - 30 MPa</td>
<td>Suspending filter papers above salt solutions of known vapour pressure</td>
</tr>
</tbody>
</table>

*Table 4.2 calibration methods for filter paper (after Marinho 1994)*
<table>
<thead>
<tr>
<th>RESEARCHERS</th>
<th>PAPER</th>
<th>M/C RANGE (%)</th>
<th>LOG&lt;sub&gt;10&lt;/sub&gt; SUCTION (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fawcett and Collis George (1967)</td>
<td>W42</td>
<td>m/c &lt;43</td>
<td>4.777 - 0.06(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;43</td>
<td>2.271 - 0.023(m/c)</td>
</tr>
<tr>
<td>McQueen &amp; Miller (1968)</td>
<td>S&amp;S 589</td>
<td>m/c &lt;54</td>
<td>5.246 - 0.0723(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;54</td>
<td>1.895 - 0.0103(m/c)</td>
</tr>
<tr>
<td>Al-Khafaf &amp; Hanks (1974)</td>
<td>S&amp;S 589</td>
<td>m/c &lt;83</td>
<td>5.117 - 0.0337(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;83</td>
<td>1.983 - 0.009(m/c)</td>
</tr>
<tr>
<td>McKeen (1980)</td>
<td>S&amp;S 589</td>
<td>m/c &lt;66</td>
<td>4.9 - 0.0624(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;66</td>
<td>1.25 - 0.0069(m/c)</td>
</tr>
<tr>
<td>Hamblin (1981)</td>
<td>W42</td>
<td></td>
<td>5.281 - 0.0822(m/c)</td>
</tr>
<tr>
<td>Chandler and Gutierrez (1986)</td>
<td>W42</td>
<td></td>
<td>4.85 - 0.0622(m/c)</td>
</tr>
<tr>
<td>Greacen, Walker &amp; Cook (1987), also (ASTM, D-5298 1992)</td>
<td>S&amp;S 589</td>
<td>m/c &lt;54</td>
<td>5.058 - 0.0688(m/c)</td>
</tr>
<tr>
<td></td>
<td>W42</td>
<td>m/c &gt;54</td>
<td>1.882 - 0.0102(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &lt;45.3</td>
<td>5.327 - 0.078(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;45.3</td>
<td>2.413 - 0.0135(m/c)</td>
</tr>
<tr>
<td>Chandler, Crilly &amp; Montgomery-Smith (1991)</td>
<td>W42</td>
<td>m/c &lt;47</td>
<td>4.842 - 0.0622(m/c)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m/c &gt;47</td>
<td>6.05 - 2.48log(m/c)</td>
</tr>
<tr>
<td>Miller &amp; Nelson (1992)</td>
<td>TS 4705-F10</td>
<td>m/c &lt;43</td>
<td>4.883 - 0.0599(m/c)</td>
</tr>
</tbody>
</table>

Table 4.3 summary of calibrations for various filter papers (after Ridley 1993)

<table>
<thead>
<tr>
<th>measured suction (kPa)</th>
<th>relative humidity (%)</th>
<th>temp drop to dew point (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1000</td>
<td>99.262</td>
<td>0.120</td>
</tr>
<tr>
<td>500</td>
<td>99.630</td>
<td>0.060</td>
</tr>
<tr>
<td>100</td>
<td>99.926</td>
<td>0.012</td>
</tr>
</tbody>
</table>

Table 4.4 comparison between suction and temp change required for condensation
**Figure 4.1** Effect of capillary radius on maximum matrix suction (Janssen and Dempsey 1980)

**Figure 4.2** Principles of osmosis
$P_0 = \text{sat. vapour pressure}$

$P = \text{partial vapour pressure}$

$P_0 = \text{sat. vapour pressure line}$

relative humidity

$= \frac{P}{P_0}$

$\text{figure 4.3 reduction in partial vapour pressure due to the osmotic effect of salts}$

$\text{figure 4.4 osmotic suction in the soil system (after Marinho 1994)}$
relative humidity is a function of:
1. salt concentration
2. meniscus curvature

figure 4.5 visualisation of suction components (after Burland & Ridley 1996)
Figure 4.6 Interaction between matrix, osmotic and total suction
3 molar NaCl solution
osmotic suction = 15 MPa

pure water

matrix suction (kPa)

effective pore radius (mm)
figure 4.8 Seeback effect

figure 4.9 Peltier effect

\[ Q = \frac{P - P'}{U} \]

\[ Q \left( \frac{P}{P_o} \right) \]

\[ P_o = \text{sat. vapour pressure} \]
\[ P = \text{partial vapour pressure} \]
\[ P' = \text{equivalent sat. vapour pressure} \]

relative humidity = \( \frac{P}{P_o} \)

figure 4.10 thermodynamic principles of thermistor psychrometer
figure 4.11 thermistor psychrometer and a typical output

figure 4.12(i) thermocouple psychrometer

figure 4.12(ii) typical response
"plateau" in EMF due to evaporation

initially cooled wet bulb

warming of wet bulb

Po = sat. vapour pressure
P = partial vapour pressure
P' = equivalent sat. vapour pressure
relative humidity = P/Po

Figure 4.13: Thermodynamic principles of thermocouple psychrometer

Figure 4.14: Relationship between relative humidity, suction and temperature
Figure 4.15 Relationship between total suction and equilibrium time

Figure 4.16 Effect of hysteresis on filter paper measurements
Chapter 4  

This study

Dry calibration

Wetting calibration

filter paper moisture content (%)

figure 4.17 comparison between filter paper calibrations for initially wet and initially dry filter papers

figure 4.18 suction plate apparatus
Sample suction = $U_a - U_w$

**Figure 4.19** Pressure plate apparatus

**Figure 4.20** Pressure plate measurement
(sample suction < applied air pressure)
**Figure 4.21** Pressure plate measurement (sample suction > applied air pressure)

**Figure 4.22** Principles of tensiometer operation
mercury reservoir

water reservoir

vacuum gauge

water (under tension)

porous ceramic

figure 4.23 tensiometer measurement methods

electrical connection

pressure transducer

filling port

"O" ring seal

PEG solution

porous ceramic

(membrane on inner face)

figure 4.24 osmotic tensiometer (after Peck and Rabbidge 1969)
figure 4.25 drift of reference pressure (Peck and Rabbidge 1969)

figure 4.26 maximum tensile stress of water (after Stannard 1992)
Figure 4.27: Imperial College tensiometer (after Ridley and Burland 1995)

Figure 4.28: Response of I.C. tensiometer
water under atmospheric pressure

water under atmospheric pressure

water subjected to high initial positive pressure

tensile stress $> 1$ atmosphere

tensile stress $> \text{initial positive pressure}$

figure 4.29 effects of pre-pressurisation of air
Figure 4.30: Air entry through porous ceramic

- When $S = < 1500$ kPa, air enters the water-air interface through the porous ceramic tensiometer.
- When $S = > 1500$ kPa, air bubbles form due to tension breakdown.

---

15 bar porous ceramic

water

air / water interface

air bubble (tension breakdown)

clay soil
Chapter 4 page 114

**Figure 4.31** Schematic layout of gypsum block

**Figure 4.32** Calibration curve for gypsum block (after Richards 1974)
figure 4.33 thermal conductivity sensor (Sattler and Fredlund 1989)

![Diagram of thermal conductivity sensor](image)

figure 4.34 calibration for thermal conductivity sensor (Sattler and Fredlund 1989)
Figure 4.35 Osmotic pressure of saturation extracts (after USDA Agricultural Handbook 1969)

Figure 4.36 Osmotic, matrix and total suction measurements on Regina Clay (after Krahn and Fredlund, 1972)
Figure 4.37: Osmotic, matrix and total suction measurements on glacial till (after Krahn and Fredlund, 1972)
Chapter 5

Suction controlled laboratory testing

5.1 AN OVERVIEW OF PREVIOUS WORK

5.1.1 Axis translation technique

Conventional pore pressure measuring systems cannot generally make measurements below -100 kPa without breakdown of the instrument occurring.

Schofield (1935) recognised this fact and suggested that the range could be extended by enclosing the sample and raising the ambient air pressure. Hilf (1956) developed such a system and demonstrated that an increase in the ambient air pressure caused an equivalent increase in the pore water pressure. Thus by raising the air pressure, the pore water pressure could be brought into the positive range and be measured by standard instrumentation.

Olson and Langfelder (1965) proposed that an increase in air pressure would isotropically compress both the soil particles and pore water, leaving the effective stresses unchanged. Also, as both soil and water can be considered incompressible there would be no volume change. As such it is a valid technique for overcoming the limitations of conventional instruments when measuring negative pore pressures.

Fredlund (1973) performed a comprehensive series of 19 tests on compacted kaolin samples to re-assess the validity of this statement. Tests were carried out both one dimensionally in an oedometer and isotropically in a triaxial cell. All tests were of the "null" type with no flow of water allowed into the pore pressure measuring system (although a small but finite flow is required to register a change on the pressure transducers). A variety of air pressure changes were performed, both increasing and decreasing. For the hypothesis to be valid, all stress changes should be of a common value, i.e.

\[
\Delta \sigma_x = \Delta \sigma_y = \Delta \sigma_z = \Delta u_w = \Delta u_a
\]

Furthermore there should be no volume change following a change in the air pressure. The results showed clearly that the hypothesis is valid. There were some small volume changes and these were attributed to:

i) An imperfect testing technique

ii) Air diffusion through the porous stone
iii) Water loss from the sample through evaporation or diffusion
iv) Secondary consolidation

As the process is one of moving the axis used for measuring the pore pressure into the positive range it is termed the "axis translation technique". It is applicable both to the measurement and control of suction (see 4.3.3).

These concepts have been applied to many laboratory procedures. Bishop and Donald (1961) developed a double walled triaxial cell at Imperial College to carry out tests on partly saturated compacted soils. Barden and Sides (1970) used a modified Rowe cell to undertake controlled suction, one dimensional consolidation on compacted soil. Escario (1980) developed a shear box apparatus, again using the axis translation technique to undertake suction controlled shear tests. More recently Wheeler and Sivakumar (1993) used a double walled triaxial cell for testing partly saturated soils.

Limitations of the axis translation technique

The limitations of the axis translation technique can be defined in two broad categories, equipment and materials.

4. Equipment

• The use of elevated air pressures requires robust equipment design to resist the effects of these raised pressures. The enclosed sample may preclude the replacement of instrumentation during a test.

• The availability of air supply can hinder testing. At Imperial College the main air supply has a maximum rating of 800 kPa. The use of compressed air bottles are required beyond this range. Their use requires uprated controls and even the smallest leak in the system can curtail a test due to the finite air supply available. This can be particularly pertinent considering the test durations associated with partly saturated soils. For a triaxial system the working range of suctions is further limited by the need for the air pressure to be equal to the sum of the suction and the confining cell pressure, $\sigma_3$.

• Commercially available porous ceramics have a maximum air entry value of 1500 kPa. Beyond this figure air can pass freely into the measuring system.

• Air can also pass through the ceramic below the air entry value in solution, coming out of solution as free air in the low pressure measuring system. This causes an underestimation of the suction, a dramatic slowing of the response of the system and introduction of errors in the
moisture change measurements. It has been suggested, Bocking and Fredlund (1980), that a correction for the effects of diffused air can be made.

Materials

- Bocking and Fredlund (1980) concluded that for the axis translation technique to be applicable, the air phase within the sample had to be continuous. This point was also noted by the earlier authors Hilf (1956) and Olson and Langfelder (1965).

- If occluded air bubbles are present in the sample, the application of an elevated air pressure will cause the pore fluid to compress. A pore pressure gradient will develop through the sample, and irreversible volume changes will occur. Due to the pore pressure gradient an excess air pressure - beyond the true sample suction - will be required to register a positive pore pressure in the measuring system. This over estimation of the suction is exacerbated with stiff samples where the pore pressure parameter "B" is relatively small.

- Bocking and Fredlund also noted a significant number of publications concerning air permeability of partly saturated soil. They summarised that for a degree of saturation in excess of 85% the air permeability was effectively zero, i.e. all pore air being occluded. It is widely accepted that an upper limit of 80% saturation should apply for use of the axis translation technique, although even at this value there may be a percentage of pore air that is occluded.

- The mechanism of desaturation and resaturation of soils is not clearly understood. Whilst drying occurs from the exposed outer surfaces of a soil, it is not clear how this progresses through the soil mass. Whether a wetting/drying front forms or generalised cavitation occurs has not been clearly demonstrated. Enhancing the ability of a standard tensiometer to measure negative pore pressures by raising the pore air pressure could hinder, or even alter the process of moisture change, and hence the behaviour of the soil.

Summary

There are some limitations to the use of the axis translation technique. The primary one is that the degree of saturation must be less than about 80%.

This effectively excludes reconstituted clays as, even pure kaolin (the largest of the clay minerals and hence that which desaturates most readily) does not reduce below a degree of saturation of 80% until the suctions approach 1500 kPa. Thus the materials that can be tested using this method are either:

1) Sand / silt / clay mixtures
ii) Compacted materials

Both of the above materials could have Sr< 80% at a suction within the range of the axis translation technique. However both then fall into the second main restriction of the technique, the effect the elevated air pressures have on the mechanism of moisture distribution and movement within the soil mass.

The limitations placed on testing by the axis translation technique coupled with the uncertainty over the effects of raised air pressures on the air-water interaction and movement of moisture lead to the conclusion that the technique is not applicable to the proposed testing schedule in this study.

Furthermore, considering the ability to measure suctions to high values under atmospheric conditions (Ridley 1993), it is fundamental that the subsequent control of suction should also be made under atmospheric conditions. A study of the application of the osmotic potential of salt solutions for this purpose was therefore undertaken.

5.1.2 Osmotic potential for control of soil suctions

Low and Anderson (1958) discussed the use of the osmotic potential of particular solutions for the control of soil suction. Their paper explored the possibilities of using such solutions to determine the thermodynamic properties of soil water, i.e. the partial molar heat conductivity and entropy. They offered no experimental evidence for their hypothesis. Their theories were based on a salt free soil only, but they suggested that by using a combination of anion and cation exchange membranes, the osmotic pressure of a soil containing free salts could be determined.


Peck and Rabbidge (1966) were the first to apply this concept for the measurement of soil suction, developing an osmotic tensiometer. This instrument was a forerunner to that published later by Peck and Rabbidge (1969). The operation of the osmotic tensiometer is discussed in detail in section 4.3.4.

Zur (1966) used the osmotic potential of PEG 6000, coupled with a cellophane membrane to determine the moisture content - suction relationship for a number of soils drying from a saturated state. Tests on the permeability of the membrane to solute revealed a transfer of 3 - 5 % after 15 days. The soil was sterilised to prevent degradation of the membrane.

Kasiff and Ben Shalom (1971) applied the above principles to an oedometer, fig 5.1. Semi-permeable membranes were fixed to the top cap and base platen of the cell. PEG 6000 solution contained in a
burette was brought into contact with one side of each membrane via a common connection, the soil sample being in contact with the other side of each membrane. The solution was replaced on a daily basis to ensure that the concentration remained constant following moisture transfer from the sample. Modifications to this initial design were published by Kasiff et al (1973), these consisted of continuous circulation of the PEG solution and additional measures to ensure good circulation of the solution (grooves and mesh beneath the membrane). The top cap was also machined to reduce side friction during swelling.

Komornik et al (1980) applied the osmotic method to a triaxial cell, fig. 5.2. A concentric hole 19.72mm diameter was bored in the standard 38mm diameter 76mm long sample. This central core was lined with a semi permeable membrane and the PEG solution circulated internally. A standard latex membrane was used externally. They noted that, theoretically the membrane and PEG combination used by both Zur (1966) and Kasiff and Ben Shalom (1971) should not have functioned as the PEG was of molecular weight 6,000 and the MWCO of the membrane 12,000. The fact that this did allow osmotic pressure to be generated adds to the uncertainty concerning the function of such mechanisms. Noting leakage through the membrane they chose to use PEG 20,000, which reduced this leakage. It may however have been the degradation of the membrane that was allowing leakage and the use of a larger molecule solute just postponed the effects of degradation.

Delage et al (1987) presented a standard triaxial apparatus incorporating osmotic control, fig 5.3. They glued membranes to the top cap and base platen. The solution was circulated from a stock solution using a peristaltic pump. Again a membrane of MWCO 12,000 and PEG 20,000 were used.

Embodying their earlier work, Delage et al (1992) produced a suction controlled oedometer based on the design of Kasiff and Ben Shalom, fig. 5.4. They included a closed system for solution circulation with a graduated capillary tube to monitor changes in the overall solution volume. This allows the movement of moisture into and out of the sample to be monitored, hence moisture content changes can be determined during the test. Due to the effects of temperature on the stock solution volume, the apparatus was placed in a temperature controlled laboratory (+/- 0.5 °C) with the stock solution in a water bath, claimed to keep temperature constant to within +/- 0.01 °C. In order to overcome bacteriological attack of the membrane a broad spectrum antibiotic was added to the solution. This extended the working life of the membrane.

It can be summarised that the limitations of the existing systems are that for triaxial tests no measurement of changes in sample volume, moisture content or suction (independently) were undertaken. The oedometer systems all lack independent suction measurement, and with the exception of the Delage system (1992), moisture exchange. Even the Delage system suffers considerably from the effects of evaporative losses from the sample affecting both the suction, and moisture content measurements. However the following points should be noted:

• The osmotic technique appears to be a successful method in controlling soil pore water tensions
under atmospheric conditions in the laboratory.

- The technique has been used, although not comprehensively studied, since the initial applications in the 1960's.

- It is an adaptable method, having been applied to oedometer and triaxial apparatus as well as being used for the isotropic drying of samples under no confining stresses.

It was felt that a comprehensive study of the technique was needed, in order to understand more fully both the technique itself and the limitations of the components. A number of ideal requirements were established for such a system:

- Suction control
- Suction measurement
- Volume change measurement
- Moisture content measurement
- Radial stress measurement
- Stress path control
- Automation

The first principles to establish are those relating to the mechanism of osmosis generally, and its application to soils testing in particular. A detailed study of this was undertaken.

5.2 OSMOSIS

5.2.1 Principles of osmosis

Osmosis (from the Greek for "push" or "impulse") is the term used to describe the flow of a solvent into a solution, or for the flow of a dilute solution into a more concentrated solution when separated by a membrane permeable to the solvent and wholly impermeable to the solute. Strictly the flow should be one of solvent only to be termed osmosis, if solute is transferred the process is one of diffusion.

Although also adequately describing gas separation, osmosis is more commonly referred to for liquid separation only.

The transfer of solvent results in an increase in pressure on the solution side of the membrane. This excess
pressures will increase and ultimately prevent the flow of solvent into solution. The pressure at this equilibrium state is termed the **Osmotic Pressure** of the solution, fig. 5.5. It is described (Glasstone and Lewis, 1978) as "the excess pressure which must be applied to a solution to prevent the passage into it of solvent when the two liquids are separated by a perfectly semi-permeable membrane."

Glasstone and Lewis, (1978) report on early work on the production of a semi-permeable membrane and a number of quantitative measurements of osmotic pressure. Measurements of the osmotic pressure of sucrose are produced in tables 5.1 and 5.2.

In table 5.1 it is seen from the consistency of the ratio \( \pi/c \) (\( \pi = \) osmotic pressure, \( c = \) percentage concentration) that by varying only the concentration, osmotic pressure is directly proportional to its concentration. Table 5.2 shows results from varying temperature only for a sucrose concentration of 1%. Whilst \( \pi \) increases with temperature, it is clear that there is close proportionality between \( \pi \) and temperature, indicated by the stable value of \( \pi/T \).

From these early experiments it was shown that the following parallel between the properties of solutions and gasses could be drawn:

\[
c = \frac{1}{v}
\]

5.2

Where:

\( c \) = concentration of solution in moles per unit volume  
\( v \) = volume of solution containing one mole of solute

hence it is possible to write,

\[
\pi/c = \text{constant}
\]

5.3

replacing \( c \) with \( 1/v \) gives (at constant temperature),

\[
\pi v = \text{constant}
\]

5.4

This is analogous with Boyle's law \( P v = MRT \)

Also, from Table 5.2, at a given concentration, it was shown that

\[
\pi/T = \text{constant}
\]

5.5

Combining both equations gives,

\[
\pi v = RT
\]

5.6
Using the osmotic pressure values for a solution of known concentration it emerged that \( R \) has a value almost identical to the gas constant \( R \). Equation 5.6 is referred to as the Van't Hoff equation of osmotic pressure, and commonly written as,

\[
\pi = RTc \quad 5.7
\]

Equation 5.7 was shown to remain accurate for only the most dilute solutions and Morse and Frazer (1907) refined the basic Van't Hoff equation, replacing \( v \) with \( v' \) the volume of solvent associated with one mole of solute, giving,

\[
\pi v' = RT \quad 5.8
\]

For very dilute concentrations the difference is negligible, however at higher concentrations the divergence increases. The deviations are similar to those experienced between real and ideal gases, although even an ideal solution would not obey the Van't Hoff equation except at very low concentrations.

In their text, Glasstone and Lewis (1978) give a derivation of a relationship between osmotic pressure and vapour pressure of a solution compared to that of a pure solvent resulting in the following equation,

\[
\pi v' = RT \ln \frac{p}{p^o} \quad 5.9
\]

where,

\( \pi \) = Osmotic Pressure  \\
\( v' \) = Partial molal volume of solvent in the solution  \\
\( R \) = Gas constant  \\
\( T \) = Absolute temperature  \\
\( p^o \) = Vapour pressure of pure solvent  \\
\( p \) = Vapour pressure of solution

The assumptions in their derivation are that the gas behaves as an ideal gas and that the solution is incompressible. This relationship is independent of the mechanism of osmosis and should be viewed as such i.e. purely from a thermodynamic point of view.

Comparison between equations and 5.7 - 5.9 and measured data are given in Fig 5.6.

### 5.2.2 Mechanism of osmosis

This is best viewed initially in isolation from both the derivation, and the physical application of osmotic pressure.
Various mechanisms have been proposed to describe the actual process that generates osmotic pressure. Some have gained wider acceptance than others, however there is still no unified agreement on the mechanism in operation.

It is intended here to present the options and not attempt to enter the discussion on which characterises the true mechanism.

The main models available are:

1. Bombardment Theory - Solute
2. Bombardment Theory - Solvent
3. Vapour Distillation
4. Solvent Tension
5. Cluster Model

1. **Bombardment theory - solute**

A simple physical chemistry approach, indicated in fig 5.7. The solute molecules, exhibiting a Brownian type motion collide with the membrane, and, unable to pass through, create an osmotic pressure. The solvent molecules are of a size that allows free transfer through the membrane hence they have no influence on the generated pressure.

2. **Bombardment theory - solvent**

Again based on kinetic theory, however this theory is dependent on the solvent. The solvent molecules "bombard" the membrane, and as the membrane is permeable to the solvent they pass through. The presence of the larger solute molecules on the solution side means that, for a unit volume, there is a lower total number of molecules on the solution side than the solvent side. Hence there is an increased rate of collisions from the pure solvent side of the membrane and, initially more pass from solvent to solution than from solution to solvent.

The passage of solvent into solution results in an increase in pressure of the solution. Equilibrium is attained when there are equal numbers of solvent molecules passing through the membrane in both directions (with a higher pressure existing on the solution side of the membrane).

3. **Vapour distillation**

Implicit in this theory is the operation of the semi-permeable membrane, fig 5.8. The assumption
is that the membrane consists of a number of fine capillaries which remain dry during osmosis. Solvent transfer is by vapour distillation only. The vapour pressure of the solution is lower than that of the solvent (due to the reduction in free energy caused by the addition of the solute). Thus evaporation will occur from solvent to solution. This will occur until the increased pressure (in the solution) returns the vapour pressures to a common value.

4. Solvent tension

A more radical theory based on the concept of tension in liquids, and solvent tension in particular, providing the energy for generation of osmotic pressure. Hullet (1903) first proposed the solvent tension approach to osmosis, this was later expanded by Hammel and Scholander (1976) and Hammel (1979). The principles postulate that in solution, solvents behave in a fundamentally different way to solutes. It is said that the solvent molecule, in solution, has an enhanced tension in comparison with that of a pure solvent. Hence a greater attraction exists for solvent molecules in the solution than the pure solvent, causing solvent to flow into the solution.

There are many opposers to the solvent tension approach. Mauro (1979), suggests that a single pressure describes such a system both in terms of escape rate pressure and solution tension. He also states that the application of partial pressures, implicit in Hammels' theory, is invalid due to the strong molecular interaction which occurs in solution. This prevents any separation of the constituent components.

5. Cluster model

Presented by Watterson (1989 & 1991) this concept is also based on solvent tension, however here tension is derived from the dynamic grouping of liquid molecules resulting from intermolecular bonding. The solvent molecules exist as clusters not single entities, and tension is developed within the physical boundary of these clusters.

The formation of clusters is a dynamic process with polymerisation and depolymerisation of clusters occurring in both pure solvent and solution continuously. The limits of tension are the physical limits of the clusters, and transcend the membrane boundary. This action is said to create wave motions in the clusters, and as the process is one of dynamic continuum the effects will be experienced throughout the liquid phase. The dimensions of the cluster define the wavelength, larger clusters producing longer, lower amplitude waves and smaller clusters producing shorter wavelengths of greater amplitude.

The introduction of a solute disrupts these interactions as they do not form part of the clusters, hence the wavelengths in the solution are correspondingly shorter and of a greater amplitude than
those in the pure solvent.

The zones of shorter wavelengths (solution) produce higher tensions in the same physical sense as higher tension produces a higher pitch (and shorter wavelength) in a vibrating string. Thus the solvent in solution will have a higher tension than that of the pure solvent.

Furthermore, by making the assumption that clusters evaporate as entire units, Watterson highlights that as the clusters are smaller in solution fewer solvent molecules enter the vapour so the pressure is reduced by the ratio,

\[
\frac{P}{P_0} = \frac{u}{u_0}
\]

5.10

where,

\begin{align*}
P &= \text{Vapour pressure of solution} \\
P_0 &= \text{Vapour pressure of pure solvent} \\
u &= \text{Volume of solution} \\
u_0 &= \text{Volume of pure solvent}
\end{align*}

Osmotic pressure is developed simply by the greater tensile attraction of the solution, and as at equilibrium the cluster exchange occurs across the membrane boundary without energy transfer, the clusters obey the gas law,

\[
P_0u_0 = kT
\]

5.11

where,

\begin{align*}
k &= \text{Boltzmann's constant} \\
T &= \text{absolute temperature}
\end{align*}

Summary

The solute bombardment theory employs solute molecule behaviour as the osmotic pressure generator. The solvent bombardment and vapour distillation theories use solvent molecules being "pushed" across the membrane, whereas the solvent tension and cluster models suggest that the solvent is "pulled" across the boundary.

It can be seen that there are many differing opinions on the mechanism driving osmosis and that universal agreement has not been reached. The editorial comment in "Forum on Osmosis" published by the American Journal of Physiology (1979) suggested that the issue was not resolved and the reader should assess the options and use the model which best suited his needs. Watterson (1991) states that the current
situation is little removed from this position. Indeed, with new concepts being postulated, the arena is becoming more diverse and unclear.

In view of this somewhat clouded situation, the views expressed by Glasstone and Lewis (1978) appear to represent the most practical approach. Theirs is a general observation of the process of osmosis and is ideally suited to the practical application of the effect of osmosis. They state that "there is a growing opinion that the actual process in operation is not important - a fact first voiced by Van't Hoff (1892). The essential point is that, for some reason connected with the presence of solute molecules, the "escaping tendency" or "activity" of solvent molecules is less in a solution than in a pure liquid. This decrease of escaping tendency or activity is immediately evident in the fact that the vapour pressure of the solution is less than in the pure solvent. One result of this difference in the escaping tendency is that molecules will always tend to pass from solvent to the solution. Expressed in more precise thermodynamic terms, it may be said that the free energy of 1 mole of solvent molecules in the solution is less than in the pure solvent."

5.2.3 Application of osmotic pressure in controlling pore water tension

The osmotic potential of a particular solution is normally applied across a restrained semi-permeable membrane. The membrane separates the solvent (soil water) from the solute (salt solution). The magnitude of pressure developed across the membrane should be equivalent to the osmotic potential of the solution. In practice however, the magnitude of tension developed may be less than the theoretical potential.

Semi-permeable membranes - how do they work?

Ideally the membrane should be wholly impermeable to the solute, whilst allowing free transport for the solvent. Thus, with the solute effectively contained the full osmotic potential should be realised as a pressure differential across the membrane boundary.

The mechanism which allows this phenomenon to develop is not clear. There are three distinct opinions as to the actual process operating:

- **Porous membrane, with finite pore size.**

  A simple physical model, easy to visualise, where the membrane has a distinct pore size, defined in terms of molecular weight (Daltons) and operating as a classical filter. All molecules below the pore size (commonly the solvent) pass freely through the membrane, whilst those above (solute) are retained.
• **Soluble membrane**

The membrane is soluble to the solvent but insoluble to the solute. Thus as the solvent exists as a continuum across the boundary there is a direct transport mechanism available. The solute, however, is insoluble in the membrane and hence there is no direct link from solution to solvent. Intrinsic in this hypothesis is the concept of an attraction for water molecules by the membrane surface. This adsorption creates the above continuum transport link.

• **Soluble membrane with fine capillary core.**

In this hypothesis the outer surfaces of the membrane absorb and desorb, with transfer across the central core occurring in the vapour phase. As the vapour pressure is higher on the solvent side distillation transfer will be from solvent to solution only (or from dilute to concentrated solution).

Each of the above models of membrane function has some integrity. The first option, which can be readily visualised as operating like a sand sieve has much to support it. Commercially, membranes are classified in terms of molecular weight cut off (MWCO), with any solute molecules above this threshold being retained by the membrane, and the membranes performance does seem to support this.

Peck and Rabbidge (1969) demonstrated this behaviour with their osmotic tensiometer, (see 4.3.4). They placed the tensiometer in contact with three salt solutions each with an osmotic potential of approximately 400 kPa and observed the reduction in the reference pressure (which should be equal to 400 kPa). The solutions were potassium chloride (KCl), PEG 200 and PEG 1540. The results are shown in fig. 5.9.

The KCl and PEG 200 solutions behaved in a similar fashion, with a small initial decrease in the reference pressure, which decayed as the solute diffusion reached equilibrium. The low value of transient osmotic pressure indicates the speed of diffusion at the membrane boundary. The PEG 1540 reached a stable value of osmotic pressure of approximately 0.7 bar, far less than that expected. Placing the tensiometer in pure water caused all measurements to return towards the initial reference pressure.

Thus whilst not employing the most rigorous of experimental techniques, Peck and Rabbidge demonstrated the permeable and partially permeable behaviour of a membrane with respect to various molecular weight solutes. They also indicated that once diffusion occurs through the membrane such that the concentration of solute is common at the boundary the osmotic potential of a solute is effectively reduced to zero.

The second hypothesis, of a membrane soluble to solvent only appears to be weak. Furthermore a membrane that can be selectively soluble to different molecular weights of the same chemical compound is a difficult concept to appreciate.
The third hypothesis, of soluble membrane surfaces with vapour distillation across the central core seems at first inspection to be a radical opinion. However discussions with Spectrum Medical Industries (Roberts, 1994) suggest that this may be the actual process in operation.

Spectrum manufacture the dialysis membranes used in medical industries and applied to this research. They classify membranes by MWCO, using dilute solutions of dextran (a glucose polymer), plotting molecular weight against percentage retained. The MWCO is defined as the molecular weight of which at least 90% is retained, tending to support the sieve theory. The pore size is given as 2.4\(\mu m\) for a MWCO of 14,000, but SMI admit that this is a back calculation of the molecular weight and it is not possible to observe a pore structure, even using scanning electron micrographs. They feel that the true mechanism is probably a soluble membrane with vapour distillation occurring across the central core.

**Summary**

In conclusion, whilst the mechanism of osmosis and the function of semi-permeable membranes are not clearly understood, it can be seen that it is possible to utilise the osmotic potential of particular solutes via a semi-permeable membrane to apply and control solvent tension. It is therefore feasible to apply such a concept to the control of pore water tension in soils.

### 5.3 EXPERIMENTAL DEVELOPMENT

The aims of this research are to study the compression and swelling of a broad range of reconstituted soils and the effects of suction on this behaviour. Both triaxial and oedometer apparatus would be suitable for this type of study. A triaxial set up would be ideal, allowing more complex stress paths to be followed, and perhaps the study of both shrinkage as well as swelling.

However, considering that both the application of the technique and the principles of the operating mechanisms were not fully understood, a decision was made to employ an oedometer apparatus. This would still enable useful tests to be undertaken, but being a more straightforward piece of apparatus it would allow more attention to be devoted to developing an understanding of the processes involved in suction controlled testing. Also the duration of tests would be reduced significantly in an oedometer as the drainage path length is much shorter. This would be further exacerbated by the need to measure suction at one boundary, hence drainage could only occur from one face.
5.3.1 Current practice

At the start of this project, the most active research on osmotically controlled testing was being undertaken by Delage et al at the soil mechanics laboratories at Ecole Nationale des Ponts et Chaussees, in Paris. Their work, based on the earlier developments by Kassif and Ben Shalom (1971), included both triaxial and oedometer systems. A visit was made to the labs to discuss their experiences. The triaxial system was not functional at that time, and attention was centred on the oedometer apparatus.

It became clear that there were a number of experimental problems and fundamental shortcomings in their equipment. These can be summarised as follows:

- There is no measurement of the actual suction in the sample either before, during, or after testing.
- The relationship between concentration of PEG and applied suction has not been determined for this particular type of application. This is significant considering the above point.
- All data recording is undertaken manually, often significant time lapses occurred between readings, i.e. at weekends.
- Evaporative losses have not been quantified.
- Problems occurred in sealing the membranes to the top and base platens. It was not clear when a good or poor seal had been achieved. This would affect the level of suction generated - see section 5.2.3.
- Moisture content measurements are made by monitoring change in a burette system. As this operated as a closed system it is greatly affected by temperature variations and evaporation from the free surface of the solution.
- There is no control on the rate of loading, and no understanding of the effect this may have on the actual suction in the sample.
- In many instances, step loading was used when undertaking oedometer testing. Clearly this would reduce the sample suction initially incrementally and could induce compression or even collapse due to the temporary reduction in the sample suction (remembering that for materials such as the compacted silt used in their work the suction would have a significant stabilising effect).

Clearly a number of detailed studies were needed into the separate aspects of osmotically controlled testing. In the first instance an experimental set up was established which reflected that of Delage et al
(1992) to confirm the abilities of such systems.

5.3.2 Oedometer design number 1

This was based around a standard 4 inch (102 mm) oedometer, adapted for suction control, fig 5.10. One way drainage was established, from the base only. The top porous stone was replaced with a perspex disc. A piece of semi-permeable membrane was placed on the base porous stone and held in place by the sample ring and its "O" ring seal. PEG solution was circulated in a closed system from a stock solution container using a peristaltic pump. The stock solution was placed in a temperature controlled water bath, to reduce temperature effects on the measured moisture exchange. Cling film was used to effect a seal between the top cap and the sample ring to prevent evaporation. A kaolin slurry sample was poured and the top cap placed.

The response of the system was quite rapid. Overnight there was a significant removal of moisture from the sample, registered on the graduated capillary tube. This initial experiment demonstrated both the usefulness of the mechanism and some of the shortcomings outlined earlier, 5.3.1. Having confirmed the abilities of the PEG system to control suctions under atmospheric conditions it was necessary to undertake a comprehensive study of the constituent parts of the experimental set-up to produce a more useable, well defined system. The key areas for study were:

- Measurement of lateral stress
- Recording of moisture exchange
- Evaporation throughout the system
- Calibration of the PEG osmotic potential
- Refinement of the circulation system

5.3.3 Oedometer design number 2

5.3.3.1 Measurement of lateral stress

In order to define the stress state in a soil fully it is necessary to measure both the major and minor
principle stresses as well as the pore water pressure. Oedometers commonly measure only the vertical stress. The proposed system would also need to control and measure the pore water pressure. There is no commercially available radial stress measuring oedometer that would lend itself to PEG suction control. It would be necessary to develop a system for measurement of radial stress. A review of applications for this purpose was carried out and is included in appendix A.

Measurement of the lateral stress in the oedometer is not a new concept. There have been many developments in this arena. There are two common approaches, the semi-rigid system and the compensating system.

- **Semi-rigid system**

  The lateral stresses cause yielding in a strain gauged thin section of the oedometer ring. The induced strains are converted to lateral stresses via a separate calibration.

- **Compensating System**

  The yielding of a strain gauged thin section of the oedometer ring occurs. The thin section is more pliant than in the semi-rigid system. The deformation is brought back to a condition of zero lateral strain by applying a compensatory pressure behind the thin section. Thus the pressure required to achieve this condition is related to the active lateral soil pressure.

Following the review it became clear that the development and incorporation of a radial stress measuring system would be a lengthy and complex process. It was felt that, considering the experimental difficulties which would be faced in developing the PEG system, radial stress measurement would be omitted from this project. A simple system could have been incorporated, however without confidence in the performance of the measuring system the results would at best have led to confusion, and at worst to erroneous and misleading data. It is hoped that the review in appendix A will provide useful information for a future project on the incorporation of such a system.

### 5.3.3.2 Measurement of moisture exchange

In a saturated soil, measurement of changes in the moisture content of a soil can be readily equated to sample volume change, as both the soil and pore fluid may be assumed to be incompressible. In an partly saturated soil the pore fluid is compressible, consisting of both fluid and air, and this direct correlation is no longer applicable.
Even for a saturated soil it is unwise in many situations, to use sample volume change measurements alone to compute soil moisture changes. Also many "saturated" soils are often partly saturated to some degree. It is prudent therefore to measure both volume change and moisture changes independently where possible.

The inaccuracies in volume measurement are reduced significantly in the oedometer where the volume change will be one dimensional. The compression of the top cap can give an accurate and confident measure of the sample volume change. However whilst the proposed stress paths should not cause the initially saturated samples to desaturate, the pore pressures will certainly be negative and as such reliance should not be placed on volume change measurement alone to compute soil moisture changes.

A further requirement of moisture change measurement for the development of this equipment was that it needed to be automated. The systems used to date, both in the literature and during early work at Imperial College relied on manual recording of the moisture transfer by monitoring changes in the level of solution in a graduated fine bore capillary tube. For tests of long duration, such manual recording is both unacceptable and impractical.

A review of devices for measuring volume change in the laboratory was undertaken by Alva-Hurtado and Selig (1981). Although they concentrated on devices for triaxial apparatus, they provided a useful general reference for such devices. They categorised the systems into three groups:

- Devices measuring volume of fluid entering or leaving the measuring system
- Devices measuring the volume of fluid entering or leaving the sample
- Devices measuring directly, changes in the sample volume

Devices measuring the fluid entering or leaving the sample directly are most applicable for this work. They use a variety of types of instrument:

i) Burette system

Similar to the graduated capillary tube method used in the Delage osmotic system, fig 5.11, these are commonly twin tube systems with a layer of silicon oil separating flow from the cell from the back pressure system. By placing the measuring cylinder within the larger tube, there is no differential pressure across the boundary, hence errors caused by pressure change on the volume of the measuring cylinder are eliminated.

There are three drawbacks to this type of system:
The resolution and range of measurements have opposing effects, improving one is at the expense of the other.

Change in direction of flow can cause the menisci to change in shape and often direction of curvature, also a change of pressure within the system can cause the menisci to change shape.

They do not lend themselves readily to automated reading. There are a number of possibilities for incorporating automated measurement including differential pressure transducers, however none are really competent or robust enough in design and operation to form a good solution.

ii) Rolling diaphragm type volume gauge.

Menzies (1975) developed a system, fig 5.12, based on the above twin tube method, but using a rolling diaphragm as opposed to silicone oil to separate the sample fluid flow from that of the back pressure system. A displacement transducer is used to monitor movement of the diaphragm, hence moisture movement within the sample. Problems of ensuring adequate de-airing and the necessity to overcome the seal friction are the main shortcomings. More recently it has been noted that acrylic cylinders absorb water for a considerable period of time (many months) and this can affect moisture calculations particularly for longer term tests.

iii) Mercury pot systems

Based on the Bishop self compensating mercury pot system, a number of authors have incorporated displacement transducers to record the movement of the mercury pot following displacement of the mercury by soil moisture movement. The use of servo controlled motors to maintain a "null" position have also been employed in some instances, with the calibration of a stepper motor against volume change.

The three systems outlined above have an number of factors in common:

- They all measure the volume of fluid displaced.
- They all incorporate a back pressure system.
- They are all closed systems.

A final type of system identified by Alva-Hurtado and Selig (1981) was a gravimetric system. Here the mass of fluid change is measured and not the volume.

Two separate types of gravimetric system were noted.
Mitchell and Burn (1971), fig 5.13, used an acrylic cup suspended from a force transducer. The cup contained pore water and was directly connected to the base of the sample. A silicone oil surface prevented any losses due to evaporation. Pore fluid movements were detected as changes in the overall mass of the container. The system was enclosed in a sealed cell allowing a back pressure to be applied. There are three drawbacks noted:

- The force transducer is not used efficiently, as it has to support the acrylic cup and contents as well as monitor changes in the overall mass.
- The range of measurements is limited to the capacity of the container. This is a function of both the direct capacity of the container and resolution required from the transducer.
- It is not possible to have a reversal of the direction of flow.

A beam balance system was presented by Davis (1972) fig 5.14. Here the force transducer is used to measure an imbalance in mass only. This occurs between two interconnecting pots, both containing silicone oil and water. Flow of water into one pot from the soil sample will displace silicone into the other pot, which in turn will displace water. The net result is a change in the relative mass of both pots due to the difference in densities between silicone oil and water. However the difference in density between soil moisture and silicone oil is small and the measuring system must be very sensitive. Any errors in measurement are accentuated when the actual moisture transfer is calculated.

The review of volume change measurement systems undertaken by Alva-Hurtado and Selig was aimed at the triaxial apparatus. The need for a back pressure system is an additional complexity associated with triaxial systems. For an oedometer system operating under atmospheric conditions this is not a requirement and the application of a gravitational mechanism is simplified.

Discussions with Mr. Steve Ackerley (the senior technician) resulted in the suggestion that the stock solution container could be sited directly on an electronic balance. This would allow the mass of the stock solution to be weighed and not its volume. There are two distinct advantages to this approach:

- Temperature changes do not affect the measurements
  Whilst the actual volume of the stock solution may vary with temperature the mass will be unaffected. This negates the need for temperature control measures such as the water bath (Delage et al 1992). Existing laboratory temperature control (+/- 1 °C) is adequate.

- Automatic recording is possible
  Modern electronic balances have a variety of data output options and can be programmed to read at specified intervals.

The stock solution needs a free surface to allow for expansion of the solution, a glass beaker was used.
The overall level changes as moisture moves into or out of the sample (recorded as changes in mass). The inlet and outlet circulation tubes are suspended in the solution, fig 5.15.

**Electronic balances**

The balances used are supplied by *Sartorius Ltd.* The capacity is 3100 g, with a resolution of 0.01 g. (equivalent to +/-0.005 cc, comparable with volume gauges in the laboratory). They have proved accurate, robust and stable, and are ideally suited to the laboratory environment.

They have a number of optional data outputs. As well as a visual display, there is an RS232 port at the rear of the balance. The output can be either in text format or as raw voltage.

The raw voltage readings are processed internally, filtered and stabilised prior to output. The display and text output is in SI units.

In order to read the balances a simple program, written in BASIC by Dr. Lidija Zdravkovic for monitoring instrumentation in her laboratory equipment, was adapted to read two balances simultaneously when coupled with an expansion card containing two RS232 ports. The card was developed and manufactured by Mr. Steve Ackerley. The user defines the number of balances to read, the scan interval, and file name under which to store the data. The program also has the ability to record all other instrumentation (displacement transducer, tensiometer, and load cell).

The program can, however, only record the data. It cannot subsequently interpret the data in order to control a test or follow a particular stress path.

As particular stress path tests were envisaged, the triaxial control program "TRIAX", written by Dr. D.Toll was utilised to record all other data. Initially it was considered that this could also be used to record the balance data.

The maximum electrical output from the balance is 1.7 volts at a zero balance reading, the output reducing as the mass increases. The MSL modulator/demodulator has a sensitivity dependent on the operating voltage. At 1.7 volts its sensitivity is 320μV, thus taking the maximum mass of 3100g = 1.7 V, a sensitivity of 320μV equates to 0.6 g.

It is possible to increase the sensitivity of the data acquisition by using the balance close to its weighing capacity. The output voltage reduces as the limit is approached and the sensitivity of the data acquisition is improved. This is at the expense of working range of the balance. The options are given in table 5.3.

It should be noted that an allowance needs to be made for the effects of electrical noise and interference.
This would normally reduce the sensitivity by a factor of about 3. Therefore to achieve comparable sensitivity of $\pm 0.005 \text{g}$, the working range would be limited to 36g. Whilst 36g of moisture exchange would probably be acceptable for a significant proportion of the proposed testing, it is a far from ideal situation. It was decided to continue to use a separate monitoring system for the balances at this stage.

Two balances were used, one for each test frame. Both were checked for long term stability and drift fig 5.16, and fig 5.17. The stability averaged better than $\pm 0.01 \text{g}$. The manufacturer recommends that the balance be switched on for at least 30 minutes prior to use to allow it to warm up. It can be seen from the results that considerably longer is required for long term stability measurements. In general the balances were not switched off once they had been set up and if the power had been turned off, they were left for at least 24 hours prior to use.

5.3.3.3 Evaporation control

During testing, measurement of small changes in stock solution mass (of the order of 10 - 20 g) would be necessary. As tests could run for one or two months, the accuracy of moisture content determinations from the stock solution measurements could be significantly affected by evaporative losses from the closed system. It was necessary to quantify and reduce where practical these losses to ensure both accuracy and confidence in the data.

Evaporative losses could occur at two distinct points in the apparatus, from the stock solution, or from the top of the sample itself. Evaporation from the stock solution would have two effects:

i) Change the concentration of the solution.

ii) Introduce errors into the measurements of moisture transfer between soil sample and solution.

At this early stage of development it was thought to be possible to use the existing relationship between PEG and osmotic potential, (Williams and Shaykewich 1969), to establish the suction in the soil samples. If this was to be the method of both control and measurement (as had been used by all previous researchers in this field) then any changes in concentration due to evaporation, which would be undetected, could have a significant impact on the actual suction in the sample. If such losses were defined, then the actual suction pertaining could be estimated from this data, allowing either correction in the post analysis stage or an ongoing adjustment of the concentration to its original value.

There are two broad categories which cause changes in the mass of solution, moisture transfer between sample and solution, and evaporation. The sum of these changes is recorded so the combined effects are known. Separation of the two effects is not straightforward, and could only be attempted during final analysis of the data by comparing initial and final moisture contents of the sample, and assuming any
additional losses were due to evaporation.

This approach is clearly only acceptable when a correction of the actual suction in the sample is a product of the post processing only. Such a definition cannot be undertaken during a test. In a saturated sample there is a secondary check on the moisture change from the compression of the sample. Assuming the sample is saturated then:

\[ \Delta V_{\text{sample}} = \Delta V_{\text{moisture}} \]  

where \( \Delta V_{\text{sample}} \) = change sample volume
\( \Delta V_{\text{moisture}} \) = change in volume of moisture

In an partly saturated soil, the volume change and soil moisture changes are uncoupled, and there is no check on the accuracy of this correction either during the test or even at the start and completion of a test.

A knowledge of the causes, extent and linearity of evaporation is vital to enable any analysis to be undertaken with confidence. The losses should be reduced to a minimum, reducing their effect on the generated suction, and accuracy of the results.

There are two approaches that could be followed in order to separate the evaporative losses.

The first is that of Delage et al (1992). They used a "dummy" system in which the PEG system was identical to that of the test apparatus but where all losses would be from evaporation. This system also mirrored changes due to temperature effects.

However the dummy system used is not comparable as only the stock solution container is replicated. It does not contain the pumped system, the tubing and fittings or the cell, all of which have potential for losses. For these reasons also, the effects of temperature may not be completely accounted for.

A more robust assessment was used to assess the potential losses, firstly from each part of the system in isolation, then from the system as a whole.

A number of key areas were identified for assessment:

1) The open surface of the stock solution container
2) The tubing and fittings
3) The gap between the top cap and the cell wall
1) Evaporation from the stock solution

The greatest losses are likely to occur from the open surface of the stock solution container.

A common approach to this type of problem is the addition of a secondary, less dense, liquid to the exposed surface, such as silicone oil. A series of tests were performed to investigate the effectiveness of this approach.

A 0.01 molar PEG solution (with an osmotic potential of 400 kPa) was used throughout the tests on evaporation. The solution containers were placed on the electronic balance and monitored continuously.

Firstly the evaporation of the solution on its own was determined. This was from a free surface of 80mm diameter and gave an average rate of 0.408 g/hr, fig 5.18. For a test duration of two weeks the losses from the exposed surface alone would be approaching 140 g, which would be an order of magnitude greater than the actual moisture loss from the sample.

The addition of a paraffin surface reduced this figure to 0.019 g/hr, a marked improvement although it was felt that some of the losses could be due to the paraffin itself evaporating.

Silicone oil was thought to be a key liquid and prior to assessing its abilities, a long term test was undertaken to identify whether the silicone oil itself suffered from evaporation. A trial lasting one month showed no overall evaporation. Some fluctuation did occur when the laboratory temperature control broke down, and the variation was in excess of 5 °C.

Silicone oil is available in a variety of viscosities. The silicone oil used in the laboratory has a viscosity of 200 centistokes. The addition of a 30mm layer of this to the surface of the PEG solution reduced the evaporation to 0.0036 g/hr, fig 5.19. The 200 cs oil was replaced with a more viscous one of 1000cs. The losses reduced to 0.00133/hr, fig 5.20. Silicone oils in excess of 1000 cs tend to be too viscous to pour easily, this was felt to be the limiting viscosity.

The area available for evaporation is fundamental to the extent of losses that occur. Reducing the free surface area from 80mm diameter to 25mm diameter (with 200cs oil) caused a reduction in losses from 0.0036 g/hr to 0.0002 g/hr.

The drawback with reducing the area to a very small value is that the change in level of the solution increases by the square of the reduction in diameter. The level change should be kept to a minimum to reduce the effect of the submerged weight of the inlet and outlet tubes. However these two factors oppose each other.
A 1.5 litre glass beaker was the container that would be used for testing and from this point it was used in all evaluation. The free surface was 100mm diameter.

A number of ideas for reducing the free surface area were assessed. The most successful option was to use a glass culture dish floated on top of a 30mm thick layer of 1000 cs oil. The losses were 0.00011 g/hr. This allows a relatively large diameter solution container to be used (100mm diameter), whilst presenting a very small area for free evaporation.

A combination of reducing the free surface area and minimising the losses from that free area give an acceptable method of minimising evaporation from the stock solution. The working version needed to have access for inlet and outlet tubes. The glass blowing department within the college cut a central hole in the culture dish and formed a lip to allow it to float, fig 5.21. This arrangement gave losses of 0.0005 g/hr. Hence for a 20 day test the losses would be 0.24g.

2) Evaporation from tubing and fittings

A wide range of peristaltic tubing is available together with numerous types of fittings and connectors.

The choice is usually dictated by the environment under which it will have to operate, (temperature, pressure and the medium that is being pumped) as well as the required lifespan and the level of hygiene.

The criteria for this application seem quite easily satisfied, room temperature, low pressure, salt solution as a pumped medium, and a lifespan of one month or more of continuous pumping. It is not surprising that very few of the available types of tubing do not satisfy this criteria. One additional requirement was that there should be no appreciable evaporative losses through the tubing itself, or through the associated fittings.

The stock solution container used for evaluation was as before, (1.5 litre beaker, filled with 0.01 M PEG solution, and covered with a 30mm thick layer of 1000cs silicone oil and a bored out glass culture dish floated on top) and comparison made of a variety of types of tubing and fittings.

Using a single piece of silicon peristaltic pump tubing, the solution was circulated in a closed loop (there were no fittings in the system at this stage). The losses were of the order of 0.005 g/hr, (one order of magnitude greater than for the stock container on its own). Clearly evaporation does occur through the walls of the tubing.

A simple trial using glass tubing throughout except at the pump head resulted in an increase in losses, although this is more likely to be as a result of the addition of two fittings between the glass and silicon
In order to reduce the losses, the length of flexible tubing was kept to a minimum by using glass tubing throughout except at the pump head itself. A number of combinations of pump head tubing and types of fittings were evaluated. These trials were run for up to one month each, in order to be representative of the test conditions themselves.

All combinations resulted in overall losses of 0.01 g/hr or greater.

A PVC based tubing, Tygon, claimed by its manufacturers, Watson Marlow, to have the lowest gas permeability of its entire range appeared to offer no improvement. Also the use of precision connectors by Omnifit made no improvement even though they were both expensive and manufactured specifically to connect glass tubing to flexible tubing.

A miniature submersible pump was also evaluated, fig 5.22. Here the connectors would be submerged and only the inlet and outlet from the oedometer cell itself would be exposed.

The rotating action of the pump in some way increased the evaporation, even with the pump running without any tubing crossing the silicone surface the evaporation increased.

There was an initial instability, probably related to the wetting of submersed parts, followed by losses of the order of 0.002 g/hr, as compared with 0.0005 g/hr for the stock solution on its own. These losses might have been acceptable if they were uniform throughout. The initial instability could not however, be predicted with any accuracy allowing competent correction.

At this stage it seemed that the set up would have to use an external pump. Tygon tubing was used throughout. In order to separate the tubing effects from that of the fittings a single piece of Tygon tubing was used in a continuous, closed loop. The peristaltic pump was connected and test commenced. Following five days of very stable data the tubing was cut, a brass connector inserted, and the test resumed. This fitting was manufactured in the Imperial College workshop, fig 5.23 and forms a very tight fit with the tubing (the tubing needs to be submerged in boiling water for 30-45 seconds to soften enough to stretch over the fitting).

The results, fig 5.24 clearly show the difference between the closed system, (the tubing itself being essentially impermeable), and that with the fitting in place. The overall losses are 0.0007 g/hr and this compares well with that for the stock solution only (0.0005 g/hr).

This value of evaporative loss is representative of that for a working oedometer as there are two connections in the trial and two in the working system, (one inlet and one outlet from the oedometer cell).
3) **Evaporation from the gap between the top cap and the cell wall**

This is the final source of evaporative losses. Delage et al (1992) sealed this space with cling film. Considering the experiences to date it seemed prudent to define these losses and compare them with other forms of seal. Using a glass beaker with an 80mm diameter opening a comparison was made between the free surface losses and that of both cling film and a latex rubber seal (of the type used for membranes in triaxial testing), fig 5.25 shows the results. The latex seal is clearly the best option although for a real evaluation this should be performed on a soil sample in the oedometer cell.

Using the final design of cell this was performed where the cutting ring (containing a kaolin sample with a suction of 500 kPa) was clamped to a base plate. The top cap was placed and the gap sealed initially with a latex membrane, and, in a separate experiment, with a nitrile rubber membrane. This set up, fig 5.26, was used to evaluate the losses from both membranes. The results are given in fig 5.27 with the latex membrane proving the most efficient.

4) **Summary**

To summarise, it can be said that the overall evaporative losses from the system are both linear and of an acceptably small magnitude. The total losses from the constituent sections are as follows:

1. Stock solution = 0.0005 g/hr
2. Tubing and fittings = 0.0002 g/hr
3. Top cap = 0.00286 g/hr

Total cumulative losses = 0.00356 g/hr

The key details are a reduction in the specific rate of evaporation from the stock solution (silicone oil) coupled with a significant reduction in the free surface area (glass culture dish). The use of Tygon tubing for circulation with a minimum number of fittings, and a latex rubber seal formed between the top cap and the cell wall. Any future attention to reducing the evaporative losses should be aimed at the losses from the top cap, although the current level of evaporation is small, linear and easily accounted for during data processing.

5.3.3.4 **Circulation pumps**

During development a number of pump options for circulation of the PEG were explored. An early decision was made to concentrate on the peristaltic pump design. This offered controlled pumping rates,
acceptable pressure and most importantly the ability to use continuous tubing. Considering the extended running periods and the need to monitor the overall solution mass, a pump system which has no tube fittings, or pump seals to consider offers a better control over solution losses through evaporation and leakage.

The operating principle is shown schematically in fig 5.28. It consists of a number of rotating rollers in a circular, concentric pump head. Each roller occludes a section of tubing, transports its contents through the pump head and expels the contained solution at the outlet. An initial trial with a pump where the rollers were friction driven by a central spindle proved the usefulness of a peristaltic pump and also the need for a direct drive mechanism for the rollers. After one week the rollers would start to slip and circulation would stop. This is primarily due to the tubing deforming and reducing the tension on the rollers which in turn begin to slip on the central spindle.

Watson Marlow produce a range of small, direct drive 12 and 24 volt pumps. The first one used had two rollers and resulted in occasional loss of pressure as the pumping action was transferred from one roller to another. Subsequently, a three roller design was used, which has proved satisfactory.

The rate of circulation has to be sufficient to ensure that moisture being drawn through the membrane, (and the subsequent diluted solution), is flushed away. The flow rate needs to reflect the rate of moisture transfer. Two measurements were made of the rate of moisture transfer. Firstly of that from a kaolin slurry sample, and secondly from a kaolin sample which had been pre-consolidated to 200 kPa from a slurry. The rates of moisture transfer were 0.18 ml/min and 0.004 ml/min respectively.

Using a peristaltic pump operating at 10 rpm, coupled with a 1.6mm bore tubing, a flow rate of 1 ml/min was recorded. This is sufficient to ensure the removal of any dilute zones and continuous circulation of fresh solution.

5.3.3.5 Semi-permeable membrane

The semi-permeable membrane used is Visking. It is a regenerated cellulose membrane. The function of semi-permeable membranes has been discussed earlier in section 5.2.3.

The choice of membrane was based on advice from both Spectrum Medical Industries, in the USA and their UK suppliers, Medicell Ltd.

The molecular weight cut off (MWCO) is defined as 12-14,000, that is to say, it will retain any particle with a molecular weight in excess of 12-14,000. Any particles smaller than this can pass through the membrane.
If this membrane is matched to a soluble salt solution whose molecular weight is greater than this MWCO value an osmotic potential will be developed across the membrane boundary. The mechanism in operation is discussed in more detail in section 5.2.2.

The membrane is supplied dry, as flat tubing, with a protective glycerol coating. Prior to use this coating must be removed by washing, once wet the membrane must not be allowed to dry out. When wetted some distortion takes place, also it becomes quite supple. Trimming of the membrane must take place just prior to use and only when the membrane has been washed.

The membrane is susceptible to bacteriological attack. This manifests itself by a rapid degradation of the membrane and loss of osmotic potential across its boundary. Its working lifespan can be as short as a few days. There are a number of options for extending the working life of the membrane. The three main possibilities are:

i) Test in a sterile environment.
ii) Test at low temperatures.
iii) Use antibiotics to fight infection.

i) Sterile environment

Equipment and soil would require sterilising, and testing would need to be conducted in a hygienic environment. This would not really be appropriate to soils testing.

ii) Low temperature

Bacteria will not develop at low temperatures, below 5 °C. Again this is not really a suitable option for soils testing environment.

iii) Antibiotics

Delage et al (1992) mention the use of an antibiotic to increase the useful life of the membrane. Advice from the Dept. of Chemistry at Imperial College yielded two possible antibiotics, Ampicillin and Tetracyclin, which could be added to the solution to reduce the effects of bacteriological attack. Both are classed as broad spectrum antibiotics, i.e. effective against a wide range of bacteria. Both come in powdered form, ampicillin is water soluble and tetracyclin needs to be dissolved initially in a 70% alcohol base. The advice was to use ampicillin first and if not effective, to try tetracyclin and finally a combination of both.
To date ampicillin has been used at a concentration of 100 mg/litre and has extended the lifespan of the membrane to in excess of one month.

5.3.3.6 Polyethylene glycol solution (PEG)

This is a water soluble salt available in a range of molecular weights from 200 to 35,000. Applied through semi-permeable membranes, initially by agriculturists studying plant growth, it has been used to generate osmotic pressure allowing control of soil suction. Whilst any water soluble salt coupled to a suitable membrane would allow the generation of osmotic pressures, the use of PEG is well documented in soils research. The understanding of its application and limitations has been less well appreciated.

Polyethylene glycol (PEG) is a long chain molecule, HO(CH₂CH₂O)nCH₂CH₂OH, prepared from ethylene oxide and water. It is available in a range of molecular weights from 200 to 35,000. As mentioned earlier the molecular weight used is 20,000 and is matched to a membrane of MWCO 12-14,000. The commercial analysis of PEG 20,000 gives a minimum molecular weight of 17,000, allowing a significant safety margin over the membrane retention limits. Its solubility varies with molecular weight, although above 10,000 it is fairly uniform, at about 50%. At the lower molecular weights solubility varies more readily with molecular weight, and the variability in molecular weights from commercial samples could have an effect on the solubility and hence pressure developed.

5.3.3.7 Preliminary experimental evaluation

With the suction controlled development at this stage, a number of evaluation tests of the system were performed.

The oedometer cell used for this series of tests is shown in fig. 5.29, and is based on that used by Schreiner (1988) during his research at Imperial College. The cell has concentrically opposed drainage outlets in the base (which could be used for PEG circulation purposes). Additionally the cell wall is clamped to the base with an "O" ring seal, allowing a semi-permeable membrane to be sealed in place, separating the PEG solution from the soil sample. The evaporative control measures were in place as was the automated recording of moisture change in the sample.

A kaolin slurry was prepared for a series of tests to evaluate the overall ability of the system to develop and control suctions under laboratory conditions. The kaolin was mixed at 1.5 x W₁, and allowed to hydrate for a number of weeks prior to use. Before placing, the sample was mixed once more, under vacuum, to remove any entrapped air.
The tests were conducted purely under suction loading, no vertical stress being applied. Measurements of moisture transfer from the sample to the PEG solution and compression of the top cap were made.

The concentration of PEG was determined from a correlation, published by Williams and Shaykewich (1969), fig 5.30, between osmotic pressure and concentration of PEG. This relationship had been widely accepted in the field of suction controlled testing in soil mechanics research.

The PEG salt was mixed with distilled water, Ampicillin antibiotic added to prevent degradation of the membrane, and the solution allowed time to dissolve thoroughly prior to use.

A piece of semi-permeable membrane was washed, trimmed and clamped between the base and cell wall with an "O" ring seal. The PEG solution was poured into a 1.5 litre beaker, inlet and outlet tubes placed and the surface of the solution covered with a 30mm deep layer of 1000 cs silicone oil and a glass culture dish. The solution was circulated and all entrapped air flushed from the system.

The kaolin slurry was carefully placed to avoid trapping any air, to a depth of 30mm and the top cap was positioned. The latex seal attached to the top cap was clamped to the cell wall effecting a seal to prevent evaporation from the top of the sample.

At this stage no formal underdrainage beneath the membrane was used. The inlet and outlet fittings were of the same diameter and the effect of pumping the PEG solution caused the membrane to bow upwards slightly away from the base of the cell, as indicated in fig 5.29. With a slurry being used as a sample, the curvature of the membrane was impressed on the sample, and the PEG solution could circulate beneath adequately.

This continual flushing of the PEG solution is necessary in order to counter the effects of dilution caused by moisture being drawn through the membrane. As discussed in section 5.2.3, the suction applied to the sample is that acting across the membrane boundary. If a dilute zone exists at the membrane, the concentration of this dilute zone will dictate the suction generated in the sample, not the concentration of the bulk solution.

With the sample placed and the PEG solution circulating, vertical compression of the top cap and moisture drawn from the sample were monitored until equilibrium conditions were observed in both.

Upon completion of suction controlled compression the sample was removed and the suction measured using the I.C. tensiometer. A final moisture content was also taken.

This test was repeated for slurried samples subjected to a range of PEG solutions between 200 and 350 g/litre. The results are discussed in detail in section 9.2. For the purposes of the development of the
apparatus, the final equilibrium suction values only are reproduced, together with the relevant concentration of PEG and theoretical values from Williams and Shaykewich (1969) in table 5.4.

It can be seen that the measured suctions from this series of tests were both considerably less than the theoretical values and far from consistent.

There are five logical possibilities to explain these inconsistent shortfalls:

- The tensiometer does not record the true suction in the sample.
- Dilution of the stock solution reduces its osmotic potential.
- A dilute zone forms at the membrane boundary reducing the "effective" potential of the solution.
- The membrane does not transmit the suctions generated by the solution.
- The PEG solution does not generate the suctions suggested by Williams and Shaykewich (1969).

Considering these hypotheses in the above order:

- **The tensiometer does not record the true suction in the sample**

As discussed in section 4.3.4, the performance of the tensiometer has been rigorously evaluated. It can be assumed with confidence that it makes accurate, reliable and repeatable measurements of matrix suction providing some rudimentary precautions are taken:

- **Good contact is made and maintained between the tensiometer and the sample.**

This is best achieved by using a soft, fingernail sized piece of the parent soil as a contact medium between the porous ceramic of the tensiometer and the soil sample in question.

- **Evaporation of moisture from the sample is prevented during measurement.**

Prior to measurement the samples were wrapped in two layers of clingfilm with a small hole cut to allow the tip of the tensiometer to make contact with the sample. A plot of a typical measurement is shown in fig 5.31. As the samples had reached equilibrium conditions in the oedometer cell, in terms of moisture exchange (sample to PEG solution) and compression prior to removal, and were at suctions less than the point of desaturation, it is fair to assume that they were also in suction equilibrium. Stable suction values were measured, and can be taken as representative of the true suction in the sample, errors due to evaporation are not discernable from the instrument response and are not likely to be significant.
Dilution of the stock solution reducing the osmotic potential of the PEG solution

Moisture is drawn from the sample to the stock solution under the suction gradient of the PEG solution during drying of the slurried samples. One effect of this is a dilution of the stock solution and a reduction of its osmotic potential. Continuous measurements were made of the mass of the stock solution and hence the extent of dilution can be computed and a corrected solution concentration defined and its actual osmotic potential defined. These are produced in table 5.5.

The effects of dilution are considerable, although the corrected values are still at least 10% less than the theoretical values (excluding the results from the measured 298 kPa test!) and the inconsistencies remain.

A dilute zone formed at the membrane boundary reducing the "effective" potential of the solution

As noted earlier the applied suction is that which exists across the membrane boundary and not that of the bulk solution. The lack of any formal drainage beneath the membrane may have allowed a dilute zone to form at the membrane boundary, reducing the applied suction. The membrane was raised slightly above the base of the cell due to the flow rate of the PEG solution and the circulation was continuous, ensuring a reasonable chance that dilute solution would be flushed away. This would need attention however, when a vertical stress were applied, or a more rigid sample tested. A positive system of underdrainage would need to be established to ensure the solution was flushed effectively across the underside of the membrane.

The membrane does not transmit the suctions generated by the solution

As discussed earlier, section 5.2.2 the mechanism which allows semi-permeable membranes to operate is unclear. It is uncertain therefore the effect that this may have on the realisation of the osmotic potential of the PEG solution as a matrix suction in a soil sample.

The PEG solution does not generate the suctions suggested by Williams and Shaykewich (1969)

The published relationship, Williams and Shaykewich (1969) between osmotic pressure and concentration of PEG is a correlation of a number of previous authors data together with some measurements made for that particular study. As such, and considering the good agreement between the data it would be expected that the relationship is competent and accurate.

Summary

At this stage the overall development had proved both successful and necessary. There were two principle areas which required further attention:
i) Underdrainage, to ensure efficient and complete flushing of the PEG solution beneath the membrane

ii) Determination of the actual suctions generated across the membrane.

5.3.4 Additional apparatus development

5.3.4.1 Underdrainage

Much attention has been placed on various types of underdrainage for laboratory equipment. Three types predominate, a series of concentric grooves interconnected by one or two central cross channels, a continuous spiral groove with inlet at the periphery and outlet at the centre of the cell base, or no formal underdrainage at all.

The main purpose of underdrainage in most laboratory equipment is to enable diffused or trapped air to be flushed out, as this affects measurements of both volume change and the pore pressure. It should also be remembered that a porous ceramic medium is generally used as a barrier between the sample and the pore pressure measuring system.

The barrier in this apparatus is a soft, flexible membrane. If either of the above positive types of underdrainage were used it would be necessary to provide some form of additional reinforcement for the membrane itself.

The criteria for the underdrainage in this instance is to provide support for the membrane and to ensure comprehensive flushing of the solution to eliminate localised dilution of the stock solution following moisture transfer across the membrane. It does not have to account for diffused air as the pressure will be higher on the solution side of the membrane (the sample having negative pore water pressures). The circulation system needs to be robust enough to operate under high applied vertical stresses.

In order to assess the effectiveness of the various options, a cell was adapted to allow vertical stresses to be applied to the top of the membrane whilst observing the effectiveness of flow occurring beneath the membrane, fig 5.32.

Using the 100mm diameter oedometer cell used for the earlier tests, a perspex cover was made to seal the top of the cell (replacing the steel one used previously). With the membrane clamped in place at the base, the cell could be filled with water and subjected to an air pressure. This allowed a load to be applied to the membrane, whilst observing flow under the membrane. The base of the cell was painted matt grey and a 10mm grid drawn to assist in comparison of the effectiveness of different types of drainage.
With the membrane clamped in place, water was circulated beneath the membrane flushing out any trapped air. The cell was then filled with water, the top cap sealed in place and 500 kPa air pressure applied simulating a 500 kPa vertical stress on a sample.

During this phase water was used in place of PEG. This was pumped continuously, and when steady state conditions were achieved, potassium permanganate dye was introduced at the inlet. The perspex top cap allowed observation of the flow pattern of the dye beneath the membrane, under a simulated sample load of 500 kPa.

Five different types of drainage were assessed, these are shown diagrammatically in figs 5.33 & 5.34. The results of the flow tests are discussed below.

**Trial number 1. No formal drainage**

The effects of the applied cell pressure caused the membrane to seal against the base preventing flow from inlet to outlet.

**Trial number 2. Woven nylon mesh, two diametrically opposed inlets, one central outlet.**

The dispersive nature of the mesh was quite effective, fig 5.35. It was felt that if the solution could be introduced from a number of points on the periphery then complete circulation may be achieved.

**Trial number 3. Woven nylon mesh, peripheral inlet, single central outlet.**

This was an advance on the idea of introducing the solution from a discrete number of points. Utilising a coarse, porous, metal filter material, a peripheral ring was manufactured approximately 10mm wide and 5mm deep and fitted into a channel cut into the base of the cell. In addition the central outlet received a core of the same filter material. The nylon mesh was retained.

As can be seen from fig 5.36 the circulation did not improve beyond that of the earlier trial. The dispersion still emanated from two distinct points, above the two inlet points.

**Trial number 4. Woven nylon mesh, peripheral inlet with circulation groove, single central outlet**

A concentric groove was machined to allow the incoming fluid to achieve a constant pressure across the underside of the porous filter.
Despite repeating this test a number of times, rotating the filter material on each occasion to eliminate the effects of any preferential flow paths within the fabric, the results were unchanged from the previous two trials, as can be seen from fig 5.37.

A review at this stage drew some general conclusions:

- The woven mesh is an effective dispersant.
- The use of various peripheral filters had little effect.

Once the path of least resistance has been achieved, preferential flow will occur on this path to the exclusion of any further dispersion.

Following many observations it can be concluded that the mesh, has unique and very useful properties of flow. The mechanism in operation appears to be such that flow in any one square of mesh can continue in the same direction within that mesh boundary. Thus a change of direction of flow of 90° occurs at each boundary, i.e. each intersection of mesh fibres. This is indicated in fig 5.38 and is primarily a function of the mesh being woven.

**Trial number 5**  Woven nylon mesh, single inlet and outlet diametrically opposed.

By combining the inlet and outlets such that they were both diametrically opposed and a maximum distance apart, the dispersive properties of the woven mesh could be employed to best effect. By using one inlet and one outlet at opposite sides of the base the shortest drainage path, directly between the inlet and outlet is only marginally shorter than the distance to the most extreme edge of the periphery. Hence full dispersion across the surface should be achieved.

The effects were dramatic, full dispersion across the base was achieved in less than one minute, fig 5.39. The trial was repeated a number of times with comparable results. A final check was made to ensure that the observed dispersion was not a result of diffusion of the dye. Once dispersion was complete, pure water was pumped through and the inflow of clear water displaced the dye in an identical pattern.

**5.3.4.2 Calibration of the osmotic potential of Polyethylene glycol (PEG)**

It seemed prudent, in light of the initial measurements, to undertake a fresh calibration, not of the osmotic pressures generated by the PEG solution, but of the suctions generated across a physical semi-permeable membrane. Both a literature review and a series of physical measurements were undertaken.
Literature review

As noted earlier, the accepted relationship between PEG and osmotic pressure is that published by Williams and Shaykewich (1969), fig 5.30. Furthermore, it is a fairly rigorous relationship in that it is a correlation of a number of previous authors work together with some measurements undertaken on behalf of the authors. The agreement between the data are good.

It has also been shown that neither the mechanism of osmosis or the function of semi-permeable membranes are clearly understood.

We have seen that the addition of salt to a solution reduces the partial vapour pressure in relation to that of a solvent, and (in a closed environment) this can be measured as a reduction in the relative humidity in the air above the solution.

All data published by Williams and Shaykewich were determined using psychrometers to measure the relative humidity. The equivalent osmotic pressure was then calculated using the equation

\[ \pi v' = RT \ln \frac{p}{P_0} \]

where,

- \( \pi \) = Osmotic Pressure
- \( v' \) = Partial molal volume of solvent in the solution
- \( R \) = Gas constant
- \( T \) = Absolute temperature
- \( p \) = Partial vapour pressure
- \( P_0 \) = Saturated vapour pressure

In this situation the "semi-permeable membrane" is in effect perfect, it allows the moisture to pass freely (as vapour) between solution and measuring instrument whilst retaining the salt solution (PEG). As such it will allow full transmission of the effects of the PEG salts.

The application of osmotic pressure in controlling matrix suctions is made through a physical membrane. Hydrostatic pressure differentials across this membrane boundary are used to effect a suction on the soil sample.

As discussed earlier, (section 4.3.4 and 5.2.3) Peck and Rabbidge (1969) employed these principles of a physical semi-permeable membrane coupled with PEG solutions to produce an osmotic tensiometer fig 4.24. The recorded positive pressure within the tensiometer (filled with PEG solution) balances the osmotic
potential of the PEG solution. They obtained a calibration of concentration of PEG against measured "osmotic pressure", (in reality they were measuring matrix suction). These results are plotted together with Williams and Shaykewich data on fig 5.40 and it can be seen that there are significant differences in the developed pressures. The reasons behind these will be discussed in detail later.

Physical calibration

A two stage calibration was undertaken, the results of which were published in the 1st International Conference for Unsaturated Soils, Dineen and Burland (1995)

The initial calibration was of the suction generated across the membrane in isolation. Once complete, an assessment of the suctions generated in a soil sample in the new oedometer set up was carried out. This allowed an assessment of the efficiency of system by comparing the suctions generated across the membrane in isolation with those measured in a soil sample in the oedometer.

i) Suctions generated across the membrane in isolation

Initially, two basic experiments were carried out to confirm the simplicity of the osmotic mechanism and its ability to control matrix suctions.

Firstly a small disc of Visking membrane (approximately 15mm diameter) was washed and placed directly on top of the tensiometer. A small pool of PEG solution (20g / 100ml) was carefully placed on top of the membrane, and agitated.

The suctions developed are shown in fig 5.41. Two important points are noted, the simplicity and effectiveness of the mechanism for generating matrix suctions under atmospheric conditions and the speed of response of measurement, an equilibrium value in 15 minutes once compliance was overcome. The maximum suction of 370 kPa was less than the osmotic potential of the solution (450 kPa). Considering the possibility of a dilute zone forming at the membrane boundary due to moisture being drawn from the tensiometer a second test was undertaken.

A simple fixture was used to retain and seal the membrane against the tensiometer. The fixture also allowed the solution (15g / 100ml) to be flushed using a syringe of stock solution. The effects of a dilute zone occurring at the membrane boundary can be seen from fig 5.42, as moisture is transferred to the solution from the tensiometer. This clearly demonstrates that dilution does indeed occur and can have a dramatic effect on the generated suctions. From this result, continuous agitation would appear necessary. Again the maximum suction of 180 kPa was less than the osmotic potential 270 kPa.
A more comprehensive range of controlled calibration tests were now undertaken. These utilised a fixture which clamped and sealed the membrane to the tensiometer, and allowed the solution to be circulated continuously in alternate directions. A stock solution of 1 litre was used throughout. The suction generated across the membrane was measured directly for PEG solutions of a range of concentrations. A typical result is shown in fig 5.43 (for a PEG solution concentration of 30g/100ml).

This experiment was repeated for solution concentrations of 17.5, 20, 22.5, 25, 30 and 35g/100ml. These results are plotted in addition to the data from Williams and Shaykewich (1969) and Peck and Rabbidge (1969) on fig 5.44. It can be seen that the results from this study lie between those of the previous authors.

The three data sets give significantly different correlations of osmotic potential (suction) against concentration of PEG salt. If we examine the experimental technique for each it gives a good indication of the reasons for these anomalies.

The Williams and Shaykewich data defines the osmotic potential of the PEG solution. Determined by measuring the relative humidity of the vapour at equilibrium above a solution in a sealed container and converting these values to osmotic potential. All measurements were made using psychrometers. As mentioned earlier the "semi-permeable" membrane is theoretically perfect. One could clearly expect these data to realise the highest values.

Both the Peck and Rabbidge study (1969) and the current one employ physical semi-permeable membranes measuring hydrostatic pressure differentials across this boundary. When one considers the limitations of our understanding of the mechanisms of both osmosis (as opposed to defining osmotic pressure) and the function of semi-permeable membranes it is perhaps not surprising that these values are less than the true osmotic potential of the solution.

These represent a fundamental difference from relative humidity data, as there is direct measurement of suction across a physical membrane. These physical measurements are under conditions comparable to those of the test environment, and hence are more applicable.

The key experimental differences between the two methods of direct suction measurement for the purposes of calibration of the PEG, (Peck and Rabbidge, 1969 and Dineen and Burland, 1995) lie in the ability to define the actual concentration of the solution and to remove a dilute zone should one form at the membrane boundary.

The Peck and Rabbidge system involves filling the osmotic tensiometer with de-aired water during assembly, to prevent any air becoming trapped within the chamber and affecting the response of the instrument. PEG solution is then injected through the filling port to displace the water. The authors admit that there is some uncertainty over the actual concentration of the solution which finally fills the chamber.
due to unknown mixing during displacement of the water. The instrument is then placed in water and a positive, reference pressure measured. The reduction in this reference pressure when placed in contact with a soil sample is defined as the suction in the sample. For the purposes of calibration they determined the reference pressures only for PEG solutions of a range of concentrations.

During determination of the reference pressure for a particular solution, moisture would be drawn into the instrument chamber. This action could result in a dilute zone forming at the membrane boundary as there is no positive mechanism for mixing the solution. This would reduce the effective concentration and hence the measured reference value.

It is interesting to note that the opposite effect would be noted when a measurement was made. The flow is reversed and a more concentrated zone could form at the membrane boundary within the instrument. This could result in an artificially lower suction measurement. This does not affect the calibration curve however.

In practice, the volume of moisture transferred in either direction is very small, equal to the displacement of the transducer diaphragm. This may then be a secondary effect, however there is no clear system of either quantifying or eliminating it. It must be said therefore that for calibration purposes the reference pressures could be less than the true value of the bulk solution.

The instrumentation set up used for this study does not suffer from either of the problems associated with the Peck and Rabbidge measurements. The initial concentration is clearly defined during preparation. The stock solution is sufficient, 1 litre, to ensure any dilution effects caused by moisture flow from the tensiometer are negligible. There is continuous, positive agitation and flushing of the solution across the membrane surface to prevent any concentration gradients developing.

The effect of these experimental differences are clear when the results are viewed. The Peck and Rabbidge results lie below those of the present study. This is most likely as a direct result of dilution of the PEG solution during filling of the instrument chamber.

The aim of this work is to apply a known, controlled suction to a soil sample. The most suitable calibration is that which reflects the actual suctions applied to the sample during testing. Measuring the suctions generated across the membrane is the correct approach. Having determined the suctions generated across the membrane in isolation, an obvious extension to this initial calibration is to confirm if these values can be realised in a soil sample under test conditions.

A further calibration exercise was undertaken involving the oedometer set up.
ii) Suctions generated under test conditions

The experimental system as a whole was used to perform a second calibration of the PEG, fig 5.45. All current design measures were incorporated, including pumps, underdrainage and measures to prevent evaporative losses. As such the calibration would be carried out under conditions which would closely reflect the final testing environment.

One modification following earlier trials was to revise the tubing diameters. The inlet was kept at 1.6mm bore but the outlet was increased to 3.2mm. This coupled with raising the cell such that there was a 150mm drop from the cell outlet to the stock solution ensured that there was no overall outflow restriction. The membrane was drawn flat onto the base of the cell by the syphon effect of the outflow.

The top cap was modified to incorporate an Imperial College tensiometer, fig 5.46. A spring loaded plate was used to retain the tensiometer. This assisted in maintaining good contact with the sample and also allowed the tensiometer to be changed as necessary.

A kaolin sample, re-constituted at 1.5 times the liquid limit was consolidated one dimensionally to 200 kPa in a 4" (102 mm) oedometer. The sample was then extruded and trimmed to 100mm diameter and 25mm thick and placed in the suction controlled oedometer. With the top cap, containing a tensiometer, in place, a PEG solution of known concentration was circulated through the base drainage system until equilibrium conditions were observed (in terms of suction, moisture exchange and compression of the sample). This process was repeated for a number of PEG solutions of increasing concentrations.

Throughout this exercise, moisture loss from the base of the sample, matrix suction at the top of the sample and vertical compression were recorded continuously.

For the purposes of the calibration the final equilibrium suction and solution concentration (corrected for dilution) at each stage only are reproduced on fig 5.47, together with the previous calibration data.

It can be seen that there is excellent correlation between the two calibration exercises undertaken for this study. This is indicative of the overall efficiency of the oedometer design in that the full matrix suction generated across the membrane in isolation (defined in the first calibration) is realised in the top of the soil sample during the second calibration.

Subsequent to this work there still remained some concern that PEG salts might be crossing the membrane reducing the measured suction. This is unlikely as the molecular weight cut off of the membrane is 14,000 and the molecular weight of the PEG salt used is 20,000 (>17,000). However further confirmation was sought using a different membrane, Spectrapor 1, which has a MWCO of 6000 - 8000. The initial experimental set-up was used, tensiometer and membrane in isolation. A series of tests were undertaken
using PEG solutions of similar concentrations as the earlier work. Comparing both data sets as a single set, the maximum deviation from the median was no more than 15 kPa for a range of measurements between 200 - 800 kPa, and no obvious deviation for one particular series of tests. This scatter is well within experimental error and is a good indication that the PEG molecules do not cross the membrane.

5.4  
OEDOMETER DESIGN NUMBER 3

As a result of the developments described in section 5.3, there was now a significant understanding of the individual components of the osmotically suction controlled oedometer. A thorough and rigorous development process had been followed, resulting in an automated system which allowed independent control and measurement of suction. The oedometer cell used for this work served a useful purpose although it had many limitations. A new cell would be needed to allow meaningful testing schedules to be undertaken. The key features of this design were identified;

i) The ability to clamp and seal the semi-permeable membrane in isolation from the sample ring.

ii) To provide a positive location for the underdrainage mesh.

iii) To be capable of fitting into the standard oedometer frame.

iv) To have a separate sample ring, into which samples could be trimmed. This would also seal against the semi-permeable membrane.

v) To incorporate a suction probe in the top cap, and allow sealing between the sample ring and top cap.

vi) To have a single inlet and outlet, diametrically opposed, with the outlet less restrictive than the inlet.

A detailed drawing of the design is given in figure 5.48. All components were machined from stainless steel. This system incorporates all the earlier developments and satisfies the above criteria.

5.4.1  Loading systems

There are a number of methods of applying loads in an oedometer test. The common methods are step loading, constant rate of strain and constant rate of loading.
Step loading

Conventional loading systems for oedometer tests involve the application of step loads, commonly doubling the applied stress at each increment. For a saturated sample this is unlikely to cause a problem as initially the load is taken by the pore fluid, with a gradual transfer to the soil structure as dissipation and subsequent compression take place.

For a partly saturated soil this loading pattern may result in observed behaviour that reflects the loading regime rather than the soil behaviour. In many situations the soil will be in a meta stable state, with the suction in the soil enhancing the stability of the soil structure. The application of a step loading will be to cause a stepwise decrease in suction which could allow significant compression or collapse to occur. This is reinforced by the data from Cui and Delage (1996) where comparison between step loading and continuous loading is made for isotropic compression on partly saturated compacted silt. They observe a much reduced isotropic yield stress $P_u$ for the step loading tests, fig 5.49. They summarise that step loading is unsuitable for suction controlled testing, and apply a continuous loading system during their testing.

For a predominately saturated soil with negative pore pressures, step loading will have an even greater influence. The pore pressure response will be much quicker and hence the suction will reduce by a higher proportion of the applied load, as the osmotic system will have had little opportunity to counteract the reduction is suction.

The control of pore pressure is fundamental to the study of soils where a moisture deficiency exists. The cyclic pore pressure regime which would exist if step loading were to be used is unacceptable. It should be noted however that this mechanism has been used in many situations for testing of such soils.

Continuous loading

The application of continuous loading offers much improvement for oedometer loading. The fundamental difficulty is that having determined the rate of loading, be it constant rate of loading or constant rate of strain, it is not possible to check the effect this loading regime has on the actual value of suction.

The assumption is that if the rate of loading is sufficiently slow, the generated excess pore pressures (in excess of the desired negative value) will dissipate, providing there is an outside influence for this course of action, e.g. osmotic mechanism.

The obvious weakness in this approach lies in selecting a suitable rate of loading. The influence of soil, fabric, and state of pore water will be significant. If the chosen rate of loading deviates even by a small
amount from the ideal, for a test duration of several weeks, the resultant suction could be significantly
different from the desired value. To date there has been no measurement of the actual suction in the
sample during testing. Indeed no data is reported on checks performed at the completion of testing to
confirm final suction values.

**Suction controlled loading**

When we consider the parameter that is perhaps of greatest influence, and certainly that whose influence
is being investigated, is the soil suction, it would seem prudent to use this as one of the controlling
parameters in such testing.

The limitation in the ability to undertake such testing has been the ability to measure the soil suction in
the sample during the test. The development of the I.C.tensiometer (see 4.3.4) has allowed quick, accurate
measurements of soil suction over a wide range, and provides the opportunity to be used for continuous
measurements during testing.

The incorporation of the tensiometer into the top cap of the oedometer allows the actual suction to be
measured at one boundary face whilst it is independently controlled at the other boundary face.

These measured values can then be used to define and vary the rate of loading through a software control
system. This would allow the sample suction to be maintained and the true influence of this parameter on
the soil behaviour to be investigated.

Two systems have been used, stepper motor controlled loading and air actuator controlled loading. The
stepper motor system was used initially, and raises the lower platen (containing the oedometer cell) into
the load cell, which is restrained in a loading frame. With the air actuator, the cell is fixed and the load
cell attached to the actuator which applies the load to the sample. Both systems work equally well,
although the air actuator system is a simpler mechanism.

5.4.2 Measurement and control systems

Two separate P.C.’s were used to monitor the tests data. One, utilising a *BASIC* program written by Dr.
Zdravkovic, monitored the mass of the stock solution of PEG. This allowed the moisture content of the
sample to be computed independently from volume change measurements.

The second P.C. is used to monitor all remaining parameters and control the stress path followed. A
diaphragm type strain gauged load cell is used to monitor the applied vertical stress. A commercially
available displacement transducer records the compression and the I.C. tensiometer measures the sample suction.

All data is gathered on an "MSL" modulator demodulator. The software, "TRIAX", developed by Dr. D.Toll for stress path triaxial testing allows definition of instruments to be recorded, recording interval, and stress path to be followed (which can be in a number of linked stages, each related to the real time measured values).

As suction is the key parameter to much of this study the control stages were defined such that the suction was maintained within a narrow pre-defined limit (commonly 5 kPa). The rate of loading is based on these measured suction values and reflects the rate at which the osmotic system is withdrawing moisture from the sample. This also allows for the influence of the soil itself on the movement of pore water, as it is the measured suction which is the fundamental controlling parameter.

5.5 Summary

The measurement and control of stress, both total and pore fluid, is essential to the investigation of soil behaviour. For a saturated soil the measured values are positive, and coupled through the principles of effective stress. For a partly saturated soil these principles do not apply, also the pore fluid is under tension and is not easily measured using conventional instrumentation.

Following Schofield's hypothesis (1935), the use of an elevated air pressure to transfer all stress states into the positive range allowed research into partly saturated soils to advance.

The limitations of this technique are:

i) The air phase must be continuous ($S_i<80\%$), this excludes a significant range of soils.

ii) The measurement and control of suction are provided by the same source and cannot be separated.

iii) The effects of an elevated air pressure on movement of moisture and desaturation of a soil are unclear.

Ridley (1993) developed an instrument capable of measuring soil suctions directly, under atmospheric conditions to in excess of 1500 kPa. This made it a fundamental requirement that any future suction control techniques should also operate under atmospheric conditions.
An elegant alternative, for control of suction under atmospheric conditions, is the use of the osmotic potential generated by a particular salt solution across a semi-permeable membrane boundary.

The addition of salt to a solution of pure water suppresses the specific free energy of the solution in comparison with pure water. If the salt solution and pure water are separated by a membrane that is permeable only to the water and not to the salt molecules, an osmotic pressure will be established across this threshold.

This principle can best be applied by the use of a large molecular weight soluble salt such as polyethylene glycol (PEG). When combined with a membrane that allows soil water and salts to move freely whilst retaining the PEG molecules, matrix suction in the sample can be controlled.

Equilibrium conditions will be achieved by water exchange between the two solutions until the head difference balances the osmotic pressure.

Such a system has been used quite successfully by a number of researchers. It was first adapted for soils work by Kassiff and Ben Shalom (1971), and more recently Delage et al. (1993) undertook further development.

A comprehensive review of existing practice highlighted the need for a re-assessment of both the understanding of the mechanism of osmosis and the application of its principles to the control of suction in oedometer testing. One of the results of this study has been the development of a new suction controlled oedometer. This oedometer includes a number of features not considered in the earlier systems:

i) Moisture exchange

In the new system the MASS not VOLUME of the moisture exchanged between the soil sample and the PEG solution is recorded. This is done by siting the stock solution on an electronic balance. Data is recorded on a computer via the RS232 port on the balance. This approach is independent of temperature change and therefore eliminates the complexities associated with the measurement of volume.

ii) Evaporation

A detailed investigation into evaporation from the surface of the stock solution led to the following measures. Firstly a layer of viscous silicone oil is placed on top of the PEG solution and although not eliminating evaporation it provides a significant reduction. Secondly the specific area for evaporation is minimised by placing a glass culture dish on the exposed surface of the oil. This combination results in a reduction of evaporative losses from the stock solution to a level such that it does not require any subsequent attention in the calculation of the moisture exchange. This study of evaporative losses also
resulted in two additional revisions:

- The use of PVC tubing as opposed to the more common silicone based tubing. The latter was found to give significant evaporative losses through the walls of the tubing.

- A latex membrane seal between the top cap and the cell. Trials showed other seals, (i.e. cling film) to be inadequate in preventing moisture loss

iii) Underdrainage

The underdrainage from beneath the semi-permeable membrane consists of a layer of woven nylon mesh with a single inlet and outlet diametrically opposed on the base of the cell (there are no circulation groves in the base). A layer of semi-permeable membrane is then clamped on top of the mesh, separating the PEG solution from the soil sample. A series of tests showed that this simple approach provides the most effective answer. A combination of drainage length and the natural dispersive nature of a woven mesh is the key to ensuring adequate circulation of the solution and flushing of the underside of the membrane.

iv) Solution circulation

A peristaltic pump is used for circulating the PEG solution as this provides controllable flow rates and operates with continuous tubing, a major advantage in reducing evaporative losses.

v) Suction measurement

An Imperial College tensiometer, Ridley & Burland (1996), is incorporated into the top cap, allowing continuous measurements of suction to be made during testing. A fixing mechanism is incorporated which allows the tensiometer to be changed if necessary during a test.

vi) Data capture

All data is captured and recorded on a computer. This includes vertical compression measurements, moisture transfer from the base of the sample to the stock solution, and matrix suction at the top of the sample. Utilising a stress path software program the user can define the stress path to be followed through stress, strain or suction measurements.
<table>
<thead>
<tr>
<th>Concentration (c) (%)</th>
<th>Osmotic Pressure (π) (mm Hg)</th>
<th>Ratio (π/c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>535</td>
<td>535</td>
</tr>
<tr>
<td>2</td>
<td>1016</td>
<td>508</td>
</tr>
<tr>
<td>4</td>
<td>2082</td>
<td>521</td>
</tr>
<tr>
<td>6</td>
<td>3075</td>
<td>513</td>
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</tbody>
</table>

*Table 5.1. Osmotic pressures of sucrose (after Glasstone and Lewis 1978)*

<table>
<thead>
<tr>
<th>Temperature (T) (°K)</th>
<th>Osmotic Pressure (π) (mm Hg)</th>
<th>Ratio (π/T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>280.0</td>
<td>505</td>
<td>1.80</td>
</tr>
<tr>
<td>286.9</td>
<td>525</td>
<td>1.83</td>
</tr>
<tr>
<td>295.2</td>
<td>548</td>
<td>1.85</td>
</tr>
<tr>
<td>305.2</td>
<td>544</td>
<td>1.79</td>
</tr>
<tr>
<td>309.2</td>
<td>567</td>
<td>1.83</td>
</tr>
</tbody>
</table>

*Table 5.2. Influence of temperature on osmotic pressure (after Glasstone and Lewis 1978)*

<table>
<thead>
<tr>
<th>Operating range (volts)</th>
<th>Sensitivity (μV)</th>
<th>Sensitivity (g)</th>
<th>Working range (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>320</td>
<td>0.6</td>
<td>0 - 3100</td>
</tr>
<tr>
<td>1.3</td>
<td>40</td>
<td>0.07</td>
<td>730 - 3100</td>
</tr>
<tr>
<td>150 mV</td>
<td>5</td>
<td>0.009</td>
<td>2827 - 3100</td>
</tr>
<tr>
<td>20 mV</td>
<td>0.625</td>
<td>0.001</td>
<td>3064 - 3100</td>
</tr>
</tbody>
</table>

*Table 5.3. Comparison of Sartorius balance working range and resolution*
<table>
<thead>
<tr>
<th>PEG concentration (g/l)</th>
<th>measured suction (kPa)</th>
<th>theoretical suction (kPa)</th>
<th>shortfall in suction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>350</td>
<td>480</td>
<td>27</td>
</tr>
<tr>
<td>250</td>
<td>298</td>
<td>720</td>
<td>58</td>
</tr>
<tr>
<td>300</td>
<td>765</td>
<td>1040</td>
<td>26</td>
</tr>
<tr>
<td>350</td>
<td>378</td>
<td>1350</td>
<td>72</td>
</tr>
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</table>

*Table 5.4 comparison of theoretical and measured suctions of PEG solutions*

<table>
<thead>
<tr>
<th>measured suction (kPa)</th>
<th>initial theoretical suction (kPa)</th>
<th>final theoretical suction (kPa)</th>
<th>shortfall in suction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>350</td>
<td>480</td>
<td>390</td>
<td>11</td>
</tr>
<tr>
<td>298</td>
<td>720</td>
<td>600</td>
<td>50</td>
</tr>
<tr>
<td>765</td>
<td>1040</td>
<td>880</td>
<td>13</td>
</tr>
<tr>
<td>1040</td>
<td>1350</td>
<td>1150</td>
<td>10</td>
</tr>
</tbody>
</table>

*Table 5.5 comparison of theoretical and measured suctions of PEG solutions following correction for dilution by sample moisture exchange*
figure 5.1 osmotically controlled oedometer
(Kassiff and Ben Shalom, 1971)

figure 5.2 osmotically controlled triaxial cell
(Kormornik et al, 1980)
**Figure 5.3** Osmotically controlled triaxial cell
*(Delage et al, 1987)*

**Figure 5.4** Osmotically controlled oedometer
*(Delage et al, 1992)*
Salt molecule

Water molecule

$h = f(c)$

Pure water

Salt solution
concentration = $c$

Semi-permeable membrane

Figure 5.5 Principles of Osmosis

Figure 5.6 Comparison of Calculated and Measured Osmotic Pressures
(after Glasstone and Lewis 1978)
**Figure 5.7** Bombardment theory of osmosis - solvent

**Figure 5.8** Vapour distillation theory of osmosis
figure 5.9 response of osmotic tensiometer to salt solutions (after Peck and Rabbidge 1969)

figure 5.10 osmotically suction controlled oedometer - design 1
Figure 5.11: Burette Gauge (after Alva-Hurtado and Selig 1981)

Figure 5.12: Menzies Type Vol. Gauge (after Alva-Hurtado and Selig 1981)

Figure 5.13: Direct Weighing Device (after Alva-Hurtado and Selig 1981)

Figure 5.14: Beam Balance Device (after Alva-Hurtado and Selig 1981)
Figure 5.15: Moisture transfer determined by mass

Figure 5.16: Balance No. 1 stability
Figure 5.17 Balance No. 2 stability

Figure 5.18 Evaporation of PEG solution from free surface

Approx. 1000 cc solution
Surface = 80 mm dia.

\[ y = -0.40794x + 1687.889 \]
**Figure 5.19** Evaporation of PEG solution with 200 cs silicone oil surface

**Figure 5.20** Evaporation of PEG solution with 1000 cs silicone oil surface
Figure 5.21 stock solution measures to prevent evaporation

Figure 5.22 trial with submersible pump to reduce evaporation
external fitting machined to suit tubing size.

1.6 or 3.2 mm bore (depending on tubing size)

Figure 5.23 PEG tubing fitting (made in-house)

Figure 5.24 comparison of evaporation through Tygon tubing (continuous and joined tubing)
Chapters page 178

free water = -0.393 g/hr
cling film = -0.034 g/hr
latex = -0.002 g/hr

figure 5.25 evaporative losses from top cap seals

figure 5.26 top cap sealing mechanism
Figure 5.27: Comparison of evaporation through latex and nitrile membrane seals.

Figure 5.28: Operating principles of peristaltic pumps.
**figure 5.29 oedometer design No. 2**

**figure 5.30 correlation between concentration of PEG and osmotic potential (after Williams and Shaykewich 1969)**
Figure 5.31 Typical suction measurement on Kaolin sample at equilibrium

Figure 5.32 Experimental set-up for assessment of underdrainage options
<table>
<thead>
<tr>
<th>TRIAL</th>
<th>INLET</th>
<th>OUTLET</th>
<th>ADDITIONAL FEATURES</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1 &amp; 3</td>
<td>2</td>
<td>Whole membrane</td>
</tr>
<tr>
<td>2</td>
<td>1 &amp; 3</td>
<td>2</td>
<td>Woven nylon mesh under membrane</td>
</tr>
<tr>
<td>3</td>
<td>1 &amp; 3</td>
<td>2</td>
<td>Porous filter used as central core and peripheral drainage material. Nylon mesh under membrane</td>
</tr>
</tbody>
</table>

*figure 5.33 schematic layout for underdrainage trials 1 - 3*
<table>
<thead>
<tr>
<th>TRIAL</th>
<th>INLET</th>
<th>OUTLET</th>
<th>ADDITIONAL FEATURES</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>1 &amp; 3</td>
<td>2</td>
<td>Porous filter used as central core and peripheral drainage material. Circulation groove machined beneath porous filter. Nylon mesh under membrane</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>3</td>
<td>Woven mesh under membrane</td>
</tr>
</tbody>
</table>

figure 5.34 schematic layout for underdrainage trials 4 - 5
inlet = 1, 3  
outlet = 2  
(10) = elapsed time (sec) from inflow of dye

*figure 5.35 trial number 2*

inlet = 1, 3  
outlet = 2  
(10) = elapsed time (sec) from inflow of dye

*figure 5.36 trial number 3*
inlet = 1, 3
outlet = 2

(10) = elapsed time (sec)
from inflow of dye

figure 5.37 trial number 4

figure 5.38 flow characteristics of woven mesh
inlet = 1
outlet = 3

(10) = elapsed time (sec)

from inflow of dye

figure 5.39 trial number 5

---

Williams and Shaykewich, 1969
(Psychrometer)

Peck and Rabbidge, 1969
(Osmotic Tensiometer)

figure 5.40 comparison between osmotic potential calibrations
(Williams & Shaykewich, 1969 and Peck & Rabbidge, 1969)
**Figure 5.41** Demonstration of osmotic potential of PEG

**Figure 5.42** Effects of dilution at the membrane boundary
**Figure 5.43** Typical result from initial PEG calibration (30 g/100 ml)

**Figure 5.44** Comparison between existing PEG calibrations and measurements with I.C. tensiometer
figure 5.45 oedometer set-up for PEG calibration

figure 5.46 tensiometer clamping mechanism
Figure 5.47 comparison between existing and new PEG calibrations (membrane and oedometer)
Figure 5.49: Comparison of yield curves for step and continuous loading (after Cui and Delage, 1996)
Chapter 6

Soil properties

6.1 INTRODUCTION

Four soils have been tested during the course of the research presented in this thesis. They form a broad range of soil types and provide both links to existing soil test data and to the abilities to test "difficult" soils.

6.2 SOIL TYPES, ORIGINS AND MINERALOGY

Four clay soils have been used, Kaolin, London, Bangali and Black Cotton Clays. They include soils which contain a dominance of each of the three main clay minerals, kaolinite, illite and montmorillonite. A limited range of both SEM and XRD analyses were carried out, principally to confirm existing mineralogical data where available and provide some general classification of the soil constituents. The SEM study was undertaken by Mr Nick Royall, and the XRD study by Mr Martin Gill in the Dept. of Geology at IC.

The SEM, as well as providing a visual image of the soil, allows individual chemical element analysis to be undertaken. The relative ratios of the elements present can be used to indicate the dominant clay minerals. This is useful in confirming the XRD analysis which defines minerals and not elements.

The XRD study was split into two separate studies, one for the clay mineralogy, and one for the rock mineralogy. A summary of both the clay and rock analyses are given in tables 6.1 and 6.2 respectively.

The clay mineralogy study is undertaken after the soil has been subjected to a range of treatments, which allow separation of the various clay minerals. Measurements are made on the sample in an air dry state, after it has been submerged in ethylene glycol, after heating to 400° C and after heating to 550° C. This allows four mineral structures to be separated Chlorite, Kaolinite, Illite and Montmorillonite. The accuracy of any quantitative determinations are related to the purity of the mineral standard used for comparison. This usually relates to an accuracy of about 5%. The results are also cross referenced with the SEM results, thus whilst not absolute, the mineralogy can be determined with a reasonable degree of accuracy and confidence.
The rock study gives an indication of the rock minerals present, and the SEM confirms their presence and gives an indication of their appearance and condition.

6.2.1 Kaolin

The Kaolin used is Speswhite Kaolin supplied by English China Clays. It has been used extensively in geotechnical research both at IC and elsewhere. It provides a readily available, consistent, source of a kaolinite soil. It is a naturally occurring soil, provided commercially by ECC. It has a clay fraction of 77 % (< 0.002 mm), and a colloidal content of 42 % (< 0.001 mm). The principle clay mineral is Kaolinite, with only traces of the other mineral groups. The main rock mineral present was quartz.

6.2.2 London Clay

The London Clay had been provided for an earlier research project by the Transport Research Laboratory (TRL). It was obtained from Summerlease clay pit located near Colnbrook in Buckinghamshire, at a depth of between 2 - 5m. The clay fraction is 71 % with a colloidal content of 61%. The principle clay mineral is interstratified illite / montmorillonite. Single clay minerals are not always apparent, and often two interlayer with one another. They may form as regular alternate layers or a more random occurrence. Bravastite is the name given to a combination of illite / montmorillonite. The behaviour of such a clay mineral will reflect both the individual minerals and the relative proportions of each present. A small amount of Kaolinite is also present together with a trace of Chlorite. Quartz and dolomite were the main rock minerals present, although there were also small quantities of gypsum, albite and calcite present.

6.2.3 Bangali Clay

The Bangali Clay was provided by the Overseas Centre of TRL. Bangali is located in the northeast of Kenya on a main transport route (A3) between Nairobi and Somalia. The soil was provided as bagged samples excavated by hand, and transported to the U.K. This material is one of many causing design and maintenance problems on this main highway route through Kenya.

It is from a region defined as being on the "grey-clay plains". These soils are described by Sombroek, Mbuvi and Okwaro (1976) as "imperfectly drained, dark greyish brown to reddish brown, of sandy clay to clay texture, and calcareous from the surface or from shallow depth". A number of rock minerals were present, table 6.2. The SEM analysis showed the granular material to be very angular in appearance, and it was suggested that the presence of feldspar suggests that further degradation may occur on an engineering timescale, ultimately yielding albite. This was confirmed by Martin Gill (XRD analysis).
Both the XRD and SEM study indicate that the clay mineralogy is principally calcium montmorillonite. The clay fraction is 68% of which 59% can be termed colloidal (ASTM).

6.2.4 Black Cotton Clay

Another Kenyan soil, this material is from the Nairobi - Mombassa road 68 km from Nairobi. These samples were collected as hand dug, bagged samples as part of the research undertaken by Dr. Denys Schreiner at IC (Schreiner, 1988). The samples used were from a depth of 600 - 850 mm, and are described as "moist very dark grey to black fissured and shattered hard to stiff slightly silty clay with occasional fine to coarse gravel".

The XRD and SEM analysis both concur that the only clay mineral present to any significant degree is calcium montmorillonite. The clay fraction is 78% of which 72% can be termed colloidal (<0.001 mm).

Quartz, albite and calcite are the main rock minerals present in the Black Cotton Clay.

6.3 PARTICLE SIZE ANALYSIS

Particle size analysis were carried out on all four clay soils. This involved both sieve and sedimentation analysis. The results are presented in figs 6.1 - 6.4, with a comparative summary plot in fig 6.5.

6.3.1 Kaolin

The Kaolin grading is uniform, with a narrow range of particle sizes covering a large percentage of the material. The whole of the sample is contained within the clay silt classification, with >90% falling below the fine silt size. There is a small residual proportion of the material larger than silt.

6.3.2 London Clay

A well graded material with a trace of fine sand and the remaining material falling across the silt / clay gradings. The clay content is slightly above 70% with a colloidal content of 62%.
6.3.3 Bangali Clay

This grading for this material falls between clay and medium sand. The granular material accounts for 35% of the sample (42% > 0.001 mm), with a gap in the grading for coarse silt.

6.3.4 Black Cotton Clay

A similar grading curve to the Bangali Clay, but with a higher clay content (75% > 0.002 mm). There is a gap in the grading curve for coarse silt.

6.4 SPECIFIC GRAVITY DETERMINATIONS

Specific gravities were measured for the London Clay and Bangali Clay. Previously published values were used for Kaolin and the Black Cotton Clay. The specific values for all materials are given in table 6.3. Four measurements were made for the Bangali Clay, giving an overall specific gravity of 2.76, with an average error of 0.023. Three measurements were made for the London Clay, giving an overall specific gravity of 2.64, with an average error of 0.01.

6.5 SOIL PARAMETERS

A summary of measured and assumed soil parameters is given in table 6.3. Where possible the actual soil parameters have been determined experimentally during this research, and compared with existing, published data. The drained angle of shearing resistance $\phi'$ is used in the analysis of the suction controlled oedometer tests. These values were obtained from the literature in each case.
<table>
<thead>
<tr>
<th>Soil type</th>
<th>Chlorite (%)</th>
<th>Illite (%)</th>
<th>Kaolinite (%)</th>
<th>Montmorillonite (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolin</td>
<td>4</td>
<td>5</td>
<td>82</td>
<td>8</td>
</tr>
<tr>
<td>London Clay</td>
<td>1</td>
<td>52</td>
<td>16</td>
<td>32</td>
</tr>
<tr>
<td>Bangali Clay</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>Black Cotton Clay</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>100</td>
</tr>
</tbody>
</table>

*Table 6.1 summary of clay mineralogical analysis*

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Main rock minerals present</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolin</td>
<td>quartz</td>
</tr>
<tr>
<td>London Clay</td>
<td>quartz, dolomite, gypsum, albedo, calcite</td>
</tr>
<tr>
<td>Bangali Clay</td>
<td>quartz, gypsum, calcite, feldspar, orthoclase</td>
</tr>
<tr>
<td>Black Cotton Clay</td>
<td>quartz, albedo, calcite</td>
</tr>
</tbody>
</table>

*Table 6.2 summary of rock mineralogical analysis*
<table>
<thead>
<tr>
<th>Property</th>
<th>Kaolin</th>
<th>London Clay</th>
<th>Bangali Clay</th>
<th>Black Cotton Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>2.61 (Ridley 1993)</td>
<td>2.64</td>
<td>2.76</td>
<td>2.65 (Schreiner 1988)</td>
</tr>
<tr>
<td>Liquid limit</td>
<td>64</td>
<td>73.7</td>
<td>94.5</td>
<td>116</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>32</td>
<td>29</td>
<td>42.08</td>
<td>48.5</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>32</td>
<td>44.7</td>
<td>52.42</td>
<td>67.6</td>
</tr>
<tr>
<td>e&lt;sub&gt;l&lt;/sub&gt;</td>
<td>1.67</td>
<td>1.95</td>
<td>2.61</td>
<td>3.02</td>
</tr>
<tr>
<td>ψ</td>
<td>23° (Martins, 1983)</td>
<td>20° (Skempton, 1961)</td>
<td>28° (Knight &amp; Blight, 1965)</td>
<td>27° (Schreiner, 1988)</td>
</tr>
<tr>
<td>Clay fraction (&lt;0.002 mm)</td>
<td>77</td>
<td>71</td>
<td>68</td>
<td>78</td>
</tr>
<tr>
<td>colloids (&lt;0.001 mm)</td>
<td>42</td>
<td>62</td>
<td>59</td>
<td>72</td>
</tr>
<tr>
<td>Activity</td>
<td>0.416</td>
<td>0.629</td>
<td>0.794</td>
<td>0.866</td>
</tr>
</tbody>
</table>

Table 6.3 summary of material properties
**Figure 6.1** Particle size distribution for Kaolin

<table>
<thead>
<tr>
<th></th>
<th>fine</th>
<th>med.</th>
<th>coarse</th>
<th>fine</th>
<th>med.</th>
<th>coarse</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Figure 6.2** Particle size distribution for London Clay
Figure 6.3: Particle size distribution for Bangali Clay

Figure 6.4: Particle size distribution for Black Cotton Clay
Figure 6.5 summary particle size distribution for Kaolin, London Clay, Bangali Clay and Black Cotton Clay.
Chapter 7

Sample preparation and test procedures

7.1 SAMPLE PREPARATION

As mentioned in chapter 6 four soils have been studied as part of this research project, Speswhite Kaolin, London Clay, and two Kenyan clay soils, Bangali Clay and a Black Cotton Clay. The origins, mineralogy and index properties are presented in chapter 6.

All soils were prepared in the same way for testing. Each soil was air dried before being mixed with deionised water and reconstituted into a slurry. Intrinsic properties were determined from the slurry and samples prepared by one dimensional consolidation for further study.

7.1.1 Initial preparation

The London, Bangali and Black Cotton Clays were obtained as disturbed, bagged samples at natural moisture content. In order to reconstitute them it was necessary to break the soil down into a small particle size. These three soils were ground in a mechanical grinder and passed through a 425 \( \mu m \) sieve prior to reconstituting.

To facilitate crushing, all visible stones were removed and the soils allowed to air dry to a residual moisture content. Oven drying was avoided as it would increase the time for hydration, particularly for soils containing montmorillonite, Grim (1962).

7.1.2 Mixing

Each soil was mixed at 1.5 \( W_L \), in line with the recommendations of Burland (1990). Burland also recommends that the sample is not allowed to dry prior to reconstituting, and that the water used for mixing is of the same composition as that in the soil in its natural state. These points are important when making comparisons between the intrinsic and natural properties of a soil. However this study is investigating the influence of suction on the intrinsic behaviour, and as such the above, whilst desirable, are not essential. One should also consider that:
i) It is difficult to determine the pore fluid chemistry with confidence. The chemistry will also change during testing, due to:
   a) Consolidation, as moisture lost contains salts.
   b) Drying, as moisture lost does not contain salts.
   c) Possible diffusion of salts across the membrane boundary into the PEG solution during suction controlled testing.

ii) To ensure adequate hydration of the expansive soils they need to be broken down to expose a maximum surface area. Air drying is needed to allow crushing to be completed.

Each soil was mixed with deionised water, initially by hand, adding dry soil to the water. This was followed by several hours mechanical mixing, and finally, a further period of mixing under vacuum to remove entrapped air and assist hydration. The slurry was then transferred to an air-tight, sealed container and allowed to stand for several weeks. When required, the sample was mixed once more under vacuum, ensuring homogeneity and de-airing.

When the container lids were removed, prior to the final vacuum mixing, it was noted that the kaolin and London Clay samples both had some standing water above the slurry, probably due to sedimentation of the clay during hydration. There was no free water above the two Kenyan soils however, with the Black Cotton Clay seeming to have a dry skin.

### 7.1.3 One-dimensional consolidation

Two different sized samples were prepared by one-dimensional consolidation, 105 mm diameter, 150 mm deep and 150 mm diameter, 150 mm deep.

The 105 mm diameter samples were used for three types of test:

i) Oedometer tests with a nominal suction (i.e. 10 kPa).
ii) Determination of the soil moisture characteristic curve.
iii) Air drying tests.

The 150 mm diameter samples were used to prepare oedometer samples (100 mm diameter) with a high initial suction. These samples commenced at 150 mm diameter and shrinkage occurred during drying. However, following this period of drying, all samples were still in excess of 100 mm diameter, allowing oedometer samples to be trimmed.
Both sizes of sample were initially prepared and consolidated in the same way.

The slurry was mixed under vacuum for several hours following the initial period of hydration, to ensure adequate mixing and to remove any trapped air.

The oedometer cells have drainage from both upper and lower faces. Porous stones were boiled in distilled water to saturate them. The inner cell walls were wiped with silicon grease to reduce side friction during consolidation. The lower porous stone was placed with a filter paper on the exposed face to prevent clogging of the stone by the soil. The cell was filled with water which was allowed to drain through the base until about 30 mm of water remained in the cell, at which point the drainage taps were closed.

The slurry was placed, taking care to do so under the standing water in the base of the cell. The water level rose as the cell was filled, preventing air from being trapped in the sample as it was formed. Once filled, the top was struck flat with a straight edge, and the top porous stone placed, again with a filter paper separating the stone from the soil. The top cap was now placed.

The drainage taps were then opened and the sample allowed to consolidate overnight under the weight of the top cap. Loading was subsequently applied using an air actuator to generate the required vertical stress, fig 7.1. All samples were consolidated to a maximum vertical stress of 200 kPa. The final load was applied for several days to ensure consolidation and creep were complete. As both sizes of sample had a common drainage path length, 75 mm, the consolidation times were similar.

Once consolidation was complete all surplus water was removed, the drainage taps were closed and the sample unloaded quickly. Porous stones and filter papers were removed and the sample gently pressed from the cell. The outer faces were cleaned, the sample wrapped in several layers of cling film and finally sealed in paraffin wax.

7.2 CALIBRATION OF TEST APPARATUS

This is a process of checking the measurements of an instrument against a known standard, to confirm its accuracy, resolution and repeatability. Calibration was carried out on each instrument in isolation, and also as part of the fully assembled test apparatus, operating under simulated test conditions. The overall performance of the test apparatus is discussed in section 7.2.2.

7.2.1 Instrument calibration

All transducers require calibration. Most commercial transducer based instrumentation is supplied with a
manufacturers calibration. These should not be accepted as more than an indication that the manufacturer has confirmed the operation of the particular instrument. Independent calibration is required for all instrumentation.

Two types of transducer based instruments were used in the testing programme, strain gauged and inductive instruments.

The strain gauged instruments rely on the dependence of electrical resistance on strain. Instruments using such devices are load cells and tensiometers. Displacement transducers use inductive principles, whereby a core moving in an electrical field will alter the generated voltage.

Calibrations are carried out on each instrument on a regular basis to ensure that malfunction, either catastrophic in the form of complete breakdown or secondary in the form of a zero drift or change in the calibration gradient has not occurred. As tests ran for several weeks at a time a check on the calibration of each instrument was carried out both before and after each test.

It is important to undertake the calibration in the environment in which the instrument will be used. Transducer outputs are affected by each component in the electrical system, including wires, plugs, monitoring equipment and power supply. In addition, changes in ambient conditions will also influence the calibration. For accurate, reliable calibration, all instruments should be plugged in, power switched on, allowed to warm up and cycled through their working range prior to commencing calibration.

### 7.2.1.1 Load cells

Imperial College load cells were used throughout. Loads are measured via a strain gauged, floating diaphragm. The three legged diaphragm is contained within an oil filled chamber which allows it to measure deviator forces if subjected to a cell pressure, although for these purposes axial force only is measured. They are available in a range of capacities, those used for this work have a maximum working load of 2700 kg force. For a 100 mm diameter sample this allows vertical stresses up to 2 MPa to be applied.

Calibration is performed using a Budenburg dead load apparatus, fig 7.2. It consists of two rams connected by a hydraulic link. The diameter of both ram pistons are known, and when a load is applied to one piston, the force generated at the other can be calculated. Rotating seals are used to reduce friction in the system. The force generated is resisted by the load cell to be calibrated. Each load cell is taken through a complete loading and unloading cycle, and a single linear regression established through the combined data.
7.2.1.2 Pressure transducers - I.C. tensiometer

The I.C Tensiometer is discussed in detail in section 4.3.4.

Prior to the porous ceramic being fitted, a primary calibration is undertaken to confirm that the instrument is functional. Once the ceramic is fitted it must be fully saturated prior to the working calibration being carried out. If saturation is not complete, not only will the tensiometer not measure high suctions, but the calibration will be unrepresentative of the working instrument.

The calibration procedure is similar to that for the load cells, also using the Budenburg dead load apparatus, fig 7.2. The transducer can be connected to the hydraulic line and the force generated by the application of dead loads applied directly to the transducer via an oil-water interface. The transducer is taken through a complete cycle of loading and unloading and a linear regression fitted to the combined data. It is particularly important to ensure that the calibration is good, as it is carried out in the positive range and subsequently extrapolated to the negative range during use. This is an acceptable practice, as the working range of the diaphragm is within its linear elastic range, and hence obeys Hooke's law.

7.2.1.3 Displacement transducers

These are linear variable differential transformer (LVDT) instruments, using the principles of inductance to generate voltage changes.

A bench mounted micrometer frame is used to calibrate the instruments, fig 7.3. As with all other instruments a complete cycle through the full range of movement is undertaken.

All calibration measurements are made with the micrometer moving in the same direction. This is to eliminate any backlash in the micrometer itself. Once calibration has been completed in one direction the "unload" cycle is carried out by backing off the micrometer one half turn more than required and then re-applying the displacement.

The drawback in this approach is that it could mask any backlash in the instrument itself as well as the calibration frame. This is unlikely to be a problem with LVDT instruments considering the principles of operation. If there were any mechanical components in the instruments, such as hinges in electrolevels then an alternative method would be advisable.
7.2.2 Compliance

This can be defined as measured displacements which are due to compression or expansion of the test apparatus itself and not the soil sample. This effect is noticed in both the lever arm and suction controlled oedometers.

In order to model and correct for the effects of compliance it is necessary to perform a check under test conditions. This involves setting up the apparatus as for a test except that a steel "dummy" sample is used in place of the soil sample. The proposed test stress path is followed and the load displacement behaviour monitored.

7.2.2.1 Lever arm oedometer

As these employed LVDT's to monitor displacements a loading sequence identical to that for the determination of the intrinsic properties could be followed and the relative displacements recorded automatically.

7.2.2.2 Suction controlled oedometer

These tests are controlled by the measured suction with load being applied by either stepper motor driven base plate or air actuator. Considering the differences between each type of apparatus, separate compliance tests were performed. There are inherent differences in the resulting data.

Stepper motor apparatus

Initial plotting of the compliance data revealed a significant step in the displacement when the direction of loading was reversed, fig 7.4. This could not be classified as compliance in the purest sense but would need to be accounted for when the load was reversed during a test. Closer inspection of the base plate revealed the cause of this displacement. The base plate is raised and lowered on a screw thread rotated by the stepper motor. The base plate is prevented from rotating by two stanchions which pass through the plate. There is some clearance between the stanchions and the plate, and when the load is reversed the plate rotates slightly. The displacement transducer (fixed to the load cell) does not however rotate. The load cell and the base plate are not truly parallel to each other - ball joints are used for locating the load cell to overcome this problem - and rotation of the base plate following load reversal could cause the displacements observed.

The displacement due to load reversal is significant, and occurs at the same point during each test. As such
it is straightforward to correct the test data for this. The remaining load displacement data was separated into loading and unloading data sets, fig 7.5. Utilising the curve fitting attributes of Stanford Graphics an equation was developed to describe both the loading and unloading data. Both equations were of a polynomial form, equation 7.1.

\[ y = a_0 + a_1 x + a_2 x^2 + \ldots + a_n x^n \]  

A 6\textsuperscript{th} order polynomial was required for the loading curve and an 8\textsuperscript{th} order for the unloading curve. These corrections were applied separately to their respective raw data. As can be seen from fig 7.5 there is some non-closure of the load-unload loop. Although small (0.05\% ε) it could generate a cumulative error if a test involving a series of load-unload loops were undertaken. However, for this work, all tests were undertaken in a single load-unload cycle.

**Air actuator apparatus**

Using an air actuator to generate the required stresses is a more straightforward process. It has much in common with the apparatus used to pre-consolidate the clay slurries, fig 7.1. The key difference is that software control is used to monitor and vary the applied stresses. Once more the curve fitting abilities of Stanford Graphics were used to generate polynomial equations describing the loading and unloading data. 4\textsuperscript{th} and 6\textsuperscript{th} order expressions were required respectively. The form of the loading and unloading compliance is shown in fig 7.6.

7.3 **INTRINSIC COMPRESSION TESTS**

The intrinsic compression and swelling properties of each soil were determined in a standard lever arm oedometer apparatus.

A 3" (75 mm) diameter sample ring, 50 mm deep was used for all tests, with drainage from both upper and lower faces via coarse porous stones.

The initial load was provided by the self weight of the top cap, and initial compression determined with a depth micrometer from a reference datum. Thereafter, load was applied in the normal way with compression being monitored with a displacement transducer.

The samples were taken from the stock reconstituted material, see section 7.1. Each sample was mixed under vacuum for several hours before use (to ensure thorough mixing and remove any trapped air).
7.3.1 Test procedure

Porous stones were saturated by boiling in deionised water and the sample ring was wiped with silicon grease to reduce side friction during compression.

Prior to placing the sample, the cell was assembled, with upper and lower porous stones, two filter papers and the top cap in place. A measurement was made from a reference datum, with a depth micrometer, of the distance to the top cap in the empty cell, fig 7.7(a).

The top cap, upper porous stone and filter paper were removed and the mixed, de-aired slurry placed, taking care to avoid trapping any air as the sample was formed. The filter paper, upper porous stone and the top cap were now replaced and a subsequent measurement made with the depth micrometer. This defined, accurately, the initial depth of the sample, fig 7.7(b).

The initial load was applied by the top cap only (1 kPa). Thereafter incremental loading at vertical stresses of 3, 6, 12, 25, 50, 100, 200, 500 and 1000 kPa was applied. The decision to increase the load was taken after analysing the compression data for that particular load. Unloading was carried out at the same increments of vertical stress.

Moisture content determinations were made from the initial slurry and the final sample.

7.4 SOIL MOISTURE CHARACTERISTIC CURVES

The soil moisture characteristic curve is defined as the relationship between moisture content and suction. It is determined by controlled drying and subsequent wetting, making measurements of both suction and moisture content at each stage. There are three options for making these measurements:

i) Pressure plate

As discussed in section 4.3.3 there are a number of limitations with this method, both from equipment and fundamental viewpoints. It is not the ideal method for this work.

ii) Combination of I.C. tensiometer and electronic balance

Fig 7.8 shows the use of an I.C. tensiometer mounted on an electronic balance. This allows both suction and moisture content to be recorded simultaneously. In theory this setup should provide an ideal solution. The drawbacks are; the limitations of the tensiometer in measuring high suctions, the inability to undertake wetting up of the sample and the uncertainty that the measured
suction is an equilibrium value for the whole sample.

iii) Filter paper

Although the slowest and most labour intensive method, it has much to recommend it. It has a wide range of measurement (30 - 30,000 kPa), it is capable of defining wetting as well as drying relationships and allows the addition or removal of small amounts of moisture in a controlled way. Also the necessity for a 5 - 10 day period between measurements ensures a reasonable degree of confidence that equilibrium has been achieved throughout the sample.

7.4.1 Samples

100 mm diameter, 25 mm thick samples were trimmed from the 105 mm diameter preconsolidated samples. Two samples were prepared from each of the four soil types.

7.4.2 Test procedure

There are a number of published guidelines on the procedure for making filter paper measurements, Chandler and Gutierrez (1986), BRE (1993), Chandler et al (1992) and ASTM (1992). A detailed review of the theory of filter paper measurements is given in section 4.3.2. The filter papers used are Whatman No 42, and the procedure is based on the principles outlined in Chandler and Gutierrez (1986). A number of modifications to the procedure were made, taking account of the experiences of researchers in the Soil Mechanics Section at Imperial College. Measurements were made of matrix suction and the following method relates to matrix suction determinations only.

7.4.2.1 Drying curve

1. Filter paper is Whatman No 42, 7 cm diameter
2. One filter paper is placed in direct contact with each exposed face, and retained with a 1 cm thick, 7 cm diameter perspex disc.
3. Two layers of clingfilm enclose the sample, each fixed in place with adhesive tape. The wrapped sample is then sealed in two plastic bags, wrapped in "bubble wrap" insulation and placed in a box, fig 7.9.
4. A seven day equilibrium period is allowed, in a temperature controlled environment (+/- 1° C)
5. Prior to measurement, small resealable plastic bags for holding the filter papers are weighed, to the nearest 0.0001 g.
6. The cling film is cut away and the filter paper sealed into the plastic bag and weighed. A pair of tweezers are used for handling the filter paper and plastic bag. It is important that the filter paper is sealed in the plastic bag as quickly as possible once exposed. With practice this can be completed within 5 seconds.

7. Once weighed, the filter paper is removed and dried in an oven at 105°C for two hours. A separate filter paper is placed in the plastic bags during this period to remove any accumulated moisture.

8. The bags are weighed, empty, once more. Again using tweezers, the filter paper is removed from the oven, sealed in the plastic bag and re-weighed.

9. The resulting measurements give the moisture content of the filter paper. The calibration by Chandler et al (1992) is used to determine the equivalent suction in the sample.

10. Whilst the filter paper is drying in the oven, physical measurements of the sample mass and volume are made. Volume measurements are made using a vernier calliper, measuring the diameter and thickness at a number of predetermined points to the nearest 0.01 mm. The mass is determined to the nearest 0.01 g.

11. The sample is allowed to air dry for one to three hours depending on the rate of moisture loss. At high suctions this period often needs to be extended as the rate of evaporation reduces. A problem with cracking during drying was observed in the Bangali and Black Cotton Clays. The Bangali Clay would show signs of distress after 30-40 minutes of air drying, and the Black Cotton Clay after only 5-10 minutes. The rate of evaporation was reduced for these two samples by creating a canopy with cling film. This created a much higher humidity environment, and allowed these samples to lose moisture at a reduced rate. This prevented cracking from occurring.

12. Once sufficient drying had taken place the process from step 1. was repeated.

7.4.2.2 Wetting curve

Once the samples were air dry the wetting cycle of the soil moisture characteristic curve was investigated.

There are two options for adding moisture to the sample during filter paper measurements:

i) Wet sample and add dry filter paper

As discussed in section 4.3.2 there are fundamental drawbacks in this approach, principally due to the effects of an uncontrolled addition of moisture. Also, when one considers the differences in both permeability and suction between the filter paper and sample when the filter paper is first placed, the direction of moisture movement is not clear. The moisture may well be drawn into the filter paper initially and subsequently move into the sample. This is a significant point as the filter paper, in common with all
porous media, exhibits hysteresis depending on the direction of moisture movement. As such, if the moisture movement is from filter paper to sample, the existing calibration for the filter paper will not apply.

ii) Place initially wet filter paper

This will allow the addition of a controlled amount of moisture, which will move from filter paper to sample only. However, considering the hysteresis exhibited by porous media, the calibration used for the drying filter papers will not be appropriate. For wetting up, the calibration proposed by Ridley (1995) is applied.

The procedure for wetting up is the same as for drying out, except that once the filter paper is removed from the sample and the sample volume and mass determined, a fresh filter paper (wet) is added immediately, and the sample wrapped again.

iii) Wetting at suctions below 1000 kPa

Placing two wet filter papers onto a soil sample at 1000 kPa will add approximately 0.7 g of moisture to the sample at equilibrium. Each successive addition will increase the mass of the sample - and hence its moisture content - by a similar amount.

Noting however, that the moisture content - suction relationship forms a broadly linear trend when plotted on a logarithmic suction axis, an increasing amount of moisture will need to be added to the sample to reduce the suction by an equal amount. Thus as the suction reduces, the effect of adding wet filter papers will be to cause a progressively smaller reduction in suction. This effect will also be compounded as the suction reduces due to the filter papers retaining an increasing amount of moisture.

Noting that each filter paper measurement takes one week, and that the wetting up process for kaolin from 1000 kPa to 100 kPa involved the addition of more than 65 g of moisture, the determination of this part of the moisture content - suction relationship would have taken 2 years to complete using the existing technique.

The same phenomenon occurs during the initial drying phase, where a significant amount of moisture needs to be removed to cause a noticeable change in suction. It can be overcome in this instance however, by allowing the sample to "air dry" for 1 - 3 hours before placing the next set of dry filter papers.

The addition of moisture is not quite as straightforward. There are two main constraints to be overcome. Firstly, the controlled addition of moisture. If moisture is allowed free ingress to the sample, differential
swelling may occur, causing damage to the sample. Controlled addition of moisture will also ensure that the sample is closer to an equilibrium condition throughout the reduction in suction, and assist in ensuring that the subsequent suction measurement is representative of the true sample suction. Secondly a method of suction measurement is needed that is sufficiently quick in response, such that the accuracy of its measurement is not affected by a change in suction at the point of contact (surface) as the sample itself moves to equilibrium.

It had been noted during the initial wetting up measurements that care had to be taken to be consistent with the amount of moisture added to the filter paper prior to placing in contact with the sample. Failure to do this resulted in an increased scatter in the measured values. Thus increasing the amount of moisture added to the wet filter papers or partially wetting the sample itself would probably result in loss of both precision and accuracy in the measured values

A technique was devised to add moisture under controlled conditions.

• Addition of moisture

An oedometer cell was modified such that it contained a reservoir of free water in its base. This was separated from the sample by a layer of the semi-permeable membrane used in the suction controlled experiments, fig 7.10. The sample is placed in direct contact with the upper surface of the membrane, the top cap placed and sealed using a latex membrane seal. A small excess head of water (5-6 cm) was established across the membrane to assist flow. The sample was left in contact for 8 - 12 hours prior to making a suction measurement. The sample was turned over every hour to ensure that both faces of the sample had equal exposure to moisture. Approximately 2 - 3 g of moisture would be added during this period.

The permeability of the membrane is \(2.127 \times 10^{-13}\) m/s. At less than 1000 kPa the sample can be considered as predominantly saturated, and as such is likely to have a permeability greater than that of the membrane. This ensures that the addition of moisture occurs in a controlled fashion.

The sample was then removed, and wrapped for 24 hours prior to making a measurement.

• Suction measurement

It would be feasible at this stage to add wet filter papers to the sample to make the measurement. However as there was some uncertainty at this stage of the state of equilibrium of the sample, and the effect that this might have on the measurement, it is perhaps not the ideal technique.
The seven day equilibrium period allowed during filter paper measurements is primarily to enable the filter paper to reach equilibrium with the soil sample. If the sample suction changed whilst the sample was in contact the filter paper might not respond sufficiently to reflect this.

The level of suction at this stage was within that measured by the I.C. tensiometer, and it seemed prudent to employ this technique for suction measurement. The quick response of the instrument would also allow a check to be made on the state of equilibrium of the sample.

A latex membrane seal was formed that would cover the sample and allow access for the tensiometer, fig 7.11. This was placed and monitored for up to 24 hours for stability of measurement.

The use of this technique allowed more than two measurements to be made per week, and with each adding moisture equivalent to 3-4 filter paper measurements this phase of wetting of the sample took 10 - 12 weeks.

7.5 AIR DRYING TESTS

A small series of tests were undertaken to investigate the rate of drying of the soils and crack propagation during drying.

Samples from each of the four soils were used. All samples were 100 mm diameter, 25 mm thick, and trimmed from the 105 mm diameter pre-consolidated samples.

7.5.1 Test procedure

Each sample was placed on an electronic balance and allowed to dry freely in the laboratory atmosphere. The change in mass of the sample was monitored and photographs taken of the surface at regular intervals. A small container of water, of known surface area, was also allowed to dry during each test to allow comparison to be made between tests undertaken at different times. This also allowed the rate of free water evaporation to be defined.

The test was complete when the mass of the soil sample remained constant, i.e. the soil had reached a residual moisture content for those particular atmospheric conditions.
7.6 SUCTION CONTROLLED OEDOMETER TESTS

A number of suction controlled oedometer tests were undertaken. These were carried out in the newly designed osmotically controlled oedometer, see section 5.4. Tests were conducted under both nominal (10 kPa) and high, constant suctions.

7.6.1 Sample preparation

All samples were trimmed to 100 mm diameter, 20 mm thick from the preconsolidated samples, see section 7.1.3.

Tests carried out at nominal suction were trimmed directly from the 105 mm diameter sample. Tests undertaken at high suction were prepared from 150 mm diameter samples.

Following initial consolidation to 200 kPa the samples should have a suction close to 200 kPa if they were unloaded under undrained conditions. Whilst measures were taken to reduce uptake of moisture during unloading - surplus water drained, drainage valves closed, and unloading operation processed as quickly as possible - it is inevitable that moisture is drawn into the sample. Measurements of suctions in the samples, following a period of equalisation, commonly gave measured values of 50 - 100 kPa. Tests at nominal suctions were prepared at this initial suction. For high suction tests a further, drying phase was needed.

7.6.1.1 Initial drying

The Kaolin and London Clay samples were air dried, placing an I.C. tensiometer in contact with the surface to monitor the development of suction. Following trimming into the oedometer ring, the measured suction was always lower than this value, by several hundred kPa in some cases. Initially this was thought to be a sampling effect, however during trimming a thin dry crust was apparent on the surface of the soil. Measurements made once this had been removed were close to the final trimmed value. This necessitated trimming the surface of the sample each time a measurement was made.

Measurements were also made by boring a small hole (5 mm deep) in the top of the sample, placing the tensiometer and covering the top surface with cling film. Thus evaporation could only occur from the sides and base of the sample, (the sample being raised above the bench). At a suction below that required, the sample was wrapped completely, and the suction monitored until an equilibrium value was attained. The suction continued to increase for some time, and the sample was generally left overnight. With practice a final equilibrium value could be attained which would be close to the required test suction. Commonly the samples were wrapped at 80-100 kPa below the required value.
There is clearly a need to ensure that the measured suction is representative of that in the bulk sample. Whilst all samples would remain saturated at these levels of suction, a very thin skin does appear to form during air drying. It is necessary to ensure that this is not influencing the measured value.

The best approach to drying a sample out appears to be to do so in a number of stages, allowing for equilibrium at each stage. Where possible the tensiometer should be allowed to remain in contact with the sample throughout the drying process.

This method proved satisfactory for the Kaolin and London Clay samples. However, with the Bangali and Black Cotton Clay samples cracking occurred in much the same way as observed during the drying tests. The cracking originated from the corners of the samples, where stress concentrations would be centred. These are areas of maximum evaporation, with moisture loss occurring from two faces in close proximity to one another, fig 7.12.

Creating a high humidity environment and hence slowing the rate of evaporation, seemed to help but did not prevent the cracking from occurring. If the sample was subsequently wrapped the cracks closed, and in many instances could not be identified upon unwrapping. The loss of moisture from the exposed surface causes a tensile stress concentration at the corners of the sample. In order to reduce this effect the sample was wrapped such that evaporation could only occur from the central core of the sample, fig 7.13. The effect of this is to create tension in the central core and a compressive hoop stress in the periphery of the sample. This prevented cracking occurring.

Once the sample had dried to the required suction it was ready for trimming into the sample ring.

7.6.2 Test procedure

7.6.2.1 Initial sample preparation

1. The sample is initially trimmed to a 102 mm diameter right cylinder. One flat face is formed, and, using a level surface and suitable steel parallels the opposite face is trimmed such that it is parallel to the first. Finally the diameter is reduced by hand using a soil lathe.

2. The sample cutting ring is weighed, and coated lightly with silicon grease to reduce side friction.

3. Using a bench mounted drill stand in the workshop, the sample ring is carefully pressed onto the sample. The drill stand allows the ring to be driven smoothly and squarely.

4. The sample ring (100 mm diameter, 40 mm deep) is driven approximately 25 mm onto the sample, fig 7.14, the remaining material cut off, and the exposed face trimmed flat. Using a 20 mm deep, 99.8 mm diameter aluminium gauge, the sample is driven further until the gauge is
flush with the cutting ring edge, fig 7.15. This leaves 20 mm of sample in the cutting ring. The second face is trimmed flat, the plug is removed and the cutting ring containing the sample weighed. Moisture content samples are taken from the trimmings.

5. The sample is placed on the bench, top cap fitted and latex seal fixed. A tensiometer is put in contact with the sample through the top cap. A softened piece of the parent material is used to ensure good contact is made between the tensiometer and the sample. This is left overnight to reach equilibrium, fig 7.16.

Fig 5.48 is reproduced in fig 7.17 and should be viewed in conjunction with sections 7.6.2.2 and 7.6.2.3

**7.6.2.2 Initial oedometer preparation**

1. The PEG solution is mixed with deionised water. The concentration determined in accordance with the new calibration, see section 6.3.4.2. Ampicillin antibiotic is added at a concentration of 100 mg / litre. The solution can take several hours to dissolve thoroughly, and a magnetic stirrer is used to assist this process.

2. A piece of semi-permeable membrane is cut, washed thoroughly under running water, and trimmed to size.

3. A piece of woven mesh is cut and trimmed to size.

4. Peristaltic pump tubing is cut to size and fitted to the oedometer. This requires the application of hot water to soften the tubing and allow it to stretch over the fittings.

5. Water is pumped through the system initially, to flush out any trapped air. Inlet and outlet tubes are filled with water as is the recess in the base plate which locates the membrane clamping ring. The mesh and membrane are placed, taking care to avoid trapping air beneath them.

6. The membrane is sealed in place with the clamping ring, (which contains an "O" ring seal). This locates in the base plate and is retained using six 4 mm diameter cap head screws. Once the membrane is clamped, all excess water can be removed from the cell. Water is circulated to ensure there are no visible leaks and all air is flushed from beneath the membrane.

7. A small volume of the PEG solution (2 - 300 ml) is now pumped through the drainage system to displace the water. This solution is pumped to loss as it will have been diluted by the water it displaces.

8. The circulation tubes are now placed in the stock solution and suspended by a retort stand to prevent contact with the container. The stock solution container is sited on an electronic balance. Silicon oil is added and the glass culture dish placed. The pump is restarted once more to confirm the PEG system is functioning satisfactorily.
7.6.2.3 Sample set up

When the sample has reached an equilibrium suction, and the cell has been set up as above, the test can be started.

1. The cutting ring, containing sample (and tensiometer) is lowered into contact with the membrane in the base of the cell and held in place with a clamping ring retained with six 4 mm diameter cap head screws. This ensures a rigid location for the sample and provides a secondary seal for the membrane as the sample ring itself also contains an "O" ring seal.

2. The load cell is positioned on the top cap, and its upper bail joint located. The base plate of the loading frame is raised manually until the load cell is held in place without placing any load on the sample beyond the self weight of the load cell. The displacement transducer is fixed to the load cell.

3. The control software can now be set to define the test path to be followed. Load, displacement and suction are monitored by the control software (the moisture exchange measurements are monitored separately). The measured suction is used to control the testing sequence during both loading and unloading stages.

During all testing the suction is held at a constant, predetermined value during both loading and unloading stages. The effect of loading will be to reduce the suction, and unloading to cause an increase in suction. The osmotic potential of the PEG solution is used to compensate for this reaction, keeping the suction at the required value.

PEG system

The PEG system is used to control sample suctions, see section 5.2.3. Initially a solution generating a suction equivalent to that required in the sample was used. A number of problems were observed with this approach:

i) During loading, moisture transfer from the sample caused a dilution of the PEG solution and reduction in its osmotic potential, leading to a reduction in the sample suction. The opposite effect was observed during unloading, with the osmotic potential increasing due to loss of moisture from the stock solution.

ii) The pore pressure gradient across the membrane (between sample and PEG solution) determines the rate of moisture transfer - within the limits of the permeability of the membrane. If this gradient is small, as it would be if the PEG / sample suctions were close to equilibrium with one another, the rate of moisture transfer is very slow.

iii) The PEG solution is subjected to a positive pressure by the pump in order to cause the solution to circulate. This reduces the effective osmotic potential and hence suction generated across the
membrane.

In order to overcome these problems a pore pressure gradient of 200 kPa is established across the membrane boundary through use of suitable concentration PEG solutions. This creates a pressure gradient across the membrane which ensures adequate moisture transfer and is relatively unaffected by pump pressure or dilution.

**Loading**

During loading, a PEG solution generating a suction 200 kPa in excess of that required in the sample is used. The initial effect is that moisture will be drawn from the sample, causing the suction to increase. This is countered by increasing the applied load to the sample. The control software is set such that the load is applied as necessary to maintain the sample suction within a 2 kPa band, e.g. If the required suction = 750 kPa, the upper and lower boundaries will be 751 kPa and 749 kPa respectively. When the measured suction reaches 751 kPa controlled loading of the sample begins, gradually reducing the suction to 749 kPa at which point loading stops. The PEG solution continues to withdraw moisture and suction begins to rise once more. This cycle is repeated continuously during loading.

**Unloading**

The same principles are applied during unloading, except that a PEG solution is utilised which generates a suction 200 kPa less than the required sample suction. Unloading is used to increase the sample suction, the sample itself drawing moisture from the PEG solution to counteract this effect.

In order to move from loading to unloading stages it is necessary to change the solution. This is performed by stopping the circulation and using an external pump to draw off the existing solution and replace it with one of the required concentration.
Chapter 7  page 220

Figure 7.1 Sample preparation using air controlled oedometer

Figure 7.2 Principle of operation of Budenburg dead load calibration frame
Chapter 7  page 221

Figure 7.3 Micrometer calibration rig

Figure 7.4 Effect of load reversal (stepper motor apparatus)
Figure 7.5 Compliance data for stepper motor apparatus

Figure 7.6 Compliance data for air actuator apparatus
initial sample height = $h_{cell} - h_{sample}$

figure 7.7 measurement of initial sample height

figure 7.8 determination of soil moisture characteristic using balance and tensiometer
figure 7.9 typical filter paper test set up

figure 7.10 controlled addition of moisture to sample
Figure 7.11 Measurement of sample with tensiometer during wetting up

Figure 7.12 Crack propagation in drying sample
Figure 7.13 measures to prevent cracking during drying

Figure 7.14 Oedometer ring driven 25 mm onto sample
cutting ring driven onto sample

figure 7.15 aluminium gauge used to ensure sample is 20 mm thick

tensiometer latex seal top cap sample

figure 7.16 initial sample preparation
Figure 7.17 Suction Controlled Oedometer

- Fixing for tensiometer (see figure 4.44)
- Fixing for latex seal
- Clamping ring for sample ring
- Sample ring 100 dia
- Recess to retain mesh
- Clamping ring for membrane
- Machined to suit std. oedometer
- SECTION X - X
- 12 number tapped holes (6 for each clamping ring)

Outlet
Inlet
Chapter 8

Experimental results, analysis and discussion - intrinsic and soil moisture characteristics

8.1 INTRODUCTION

A wide and varied range of experimental work has been undertaken as part of this study into the determination of the intrinsic and soil moisture characteristics of a broad range of soil types. The results will be presented under two headings:

i) Intrinsic compression tests

These will define the intrinsic behaviour of the materials. This will allow the influence of suction to be evaluated, the effects of the tests conditions, in particular the new apparatus, to be studied, and provide a link between this work and more conventional soil tests

ii) Soil moisture characteristic tests

Involving unconfined drying and wetting tests, from low suctions to the limits of the measurement techniques employed. Relationships between sample volume, suction, moisture content and degree of saturation will be developed and discussed. A brief study into the rate of evaporation from different soil types, the effect of the unconfined stress state, formation and propagation of shrinkage cracks and variation of moisture content through a drying sample will also be presented.

The data will be presented as a gradual build up of the behaviour of the material. Commencing with the intrinsic properties, we will move on to the soil moisture characteristics and also the evaporative study.
8.2 INTRINSIC COMPRESSION AND SWELLING

8.2.1 Terminology

The importance of developing a reference framework against which to compare soil behaviour was highlighted once more by Burland (1990). He proposed that the behaviour of a reconstituted soil could be used as a basis for the interpretation of the same soil in its natural state.

The reconstituted soil parameters are termed intrinsic as they are inherent in the soil, and independent of its natural state. By defining the intrinsic properties, the influence of the structure, fabric and stress history could be assessed.

These same concepts can be applied to this study whereby the influence of suction is to be studied.

A number of "intrinsic" parameters were introduced. Those with relevance to this study are detailed below:

- **Intrinsic**

  A term applied to the properties of a clay that has been reconstituted at between 1.25 and 1.5 \( w_c \). Ideally the sample should not have been allowed to air or oven dry, and the fluid used for reconstituting should be identical to the initial pore fluid in the sample (see section 7.1.2).

  An asterisk, \( * \), is used to denote an intrinsic property.

  \[
  \begin{align*}
  e^{*}_{100} &= \text{Intrinsic void ratio, at } \sigma' = 100 \text{ kPa} \\
  e^{*}_{1000} &= \text{Intrinsic void ratio at } \sigma' = 1000 \text{ kPa} \\
  c^{*}_{c} &= \text{intrinsic compression index, } = e^{*}_{100} - e^{*}_{1000} \\
  c^{*}_{s} &= \text{intrinsic swelling index, } = e^{*}_{100} - e^{*}_{1000} \text{ (Note: these values are from swelling tests)}
  \end{align*}
  \]

- **Void Index**

  In order to allow comparison between a number of different soil types, the intrinsic properties can be normalised with respect to void ratio, eqn 8.1.

  Therefore when \( e = e^{*}_{100} \), \( I_v = 0 \), and when \( e = e^{*}_{1000} \), \( I_v = -1 \).
\[ \frac{e - e^*_{100}}{e^*_{100} - e^*_{1000}} = \frac{e - e^*_{100}}{C'_c} \]  

8.1

- **Intrinsic compression line**

This is an empirically derived relationship which can be described by the cubic spiral equation, 8.2, although it is preferable to measure the relationship directly.

\[ I_v = 2.45 - 1.285x + 0.015x^3 \]  

8.2

where \( x = \log \sigma^* \) (kPa)

8.2.2 **Intrinsic compression and swelling tests**

The intrinsic compression and swelling characteristics were determined for all four soils. The results are shown in fig 8.1-8.2, with the indices summarised in table 8.1

**Intrinsic compression index**

These are summarised in fig 8.1. The general trend is as one would expect, with the more active clays exhibiting greater compression, where activity is as defined by Skempton (1953), eqn 8.3.

activity = \( \frac{\text{plasticity index}}{\text{clay fraction}} \)  

8.3

Clay mineralogy can have a dominant influence, particularly in reconstituted materials where the attributes of structure, fabric and stress history are not considered. As such it will be the mineralogy, both in terms of type of clay mineral, and the extent to which each is present which will govern compressive behaviour.

All soils exhibit linear trends when plotted in \( e - \log \sigma^* \) space. The compression indices obtained by plotting a "best fit" straight line through all the data correlate well with those obtained by plotting through the void ratio at \( \sigma^*_c = 100 \) and 1000 kPa only, \( C'_c \) (as defined by Burland, 1990).

**Intrinsic swell index**

These are summarised in fig 8.2. There is good agreement in the data for each soil, with a similar trend observed as for the compression data with the more active soils displaying a greater capacity for swelling. There would be little change in the swelling indices if a best fit line were drawn through all data points.
as opposed to only through those at $\sigma_v = 100$ kPa & 1000 kPa only ($C_v^*$)

**Void index**

Normalising the compression characteristics, eqn 8.1, allows direct comparison between a range of soil types. Fig 8.3 compares the experimental results with the general ICL proposed by Burland (1990).

There is good agreement between all soils from a void index of about 0.5. There is some deviation above this value of void index, although this threshold represents a vertical stress of less than 30 kPa and may suffer from experimental difficulties at such low applied stresses. The void index relationship (eqn 8.2) is intended for use over a higher range of vertical stresses and then only as a guide, where experimental data is not available. Nevertheless, the experimental data compliments the application of this concept.

The most marked deviation is for the Bangali and Black Cotton Clays. Burland (1990) qualified the empirically derived intrinsic concepts with respect to the moisture content at the liquid limit as only being applicable to soils which lie above the "A" line on the plasticity chart. The Bangali and Black Cotton Clays both lie below the "A"line.

If we plot the data from Burland (1990) and that from Schreiner (1991), together with the data from this study, on the plasticity chart we can identify a number of soils which lie below the "A" line, fig 8.4.

Five soils from Schreiner (1991), one from Burland (1990) and two from the present study fall into the silt category. Let us consider these "silt" soils in isolation. They actually form a broad range, including some highly plastic, expansive soils through to some very silty soils. The intrinsic characteristics are given in table 8.2.

Plotting the void index relationship of these soils, fig 8.5, we note that there is a strong correlation between them, although the relationship differs from that given for the "clay" soils. The relationship can be represented by equation 8.4.

$$I_v = 2 - \log_{10}(x)$$

where $(x) = \sigma_v \text{ (kPa)}$

The intrinsic swelling data can be presented, as void index (swelling), in fig 8.6. This requires the use of the intrinsic swelling index $C_v^*$ in place of the intrinsic compression index $C_v^*$ in equation 8.1. The relationship described by equation 8.4 represents the data presented quite well. There is some deviation at low vertical stresses, which requires further attention, however the general trend is common to these soils. The void index (swelling) is very sensitive to small changes in void ratio, and considering the form of the swelling curves at low applied stresses for the expansive soils (fig 8.2), it is clear that they may deviate from the trend predicted by Burland (1990).
Intrinsic properties and void ratio at the liquid limit

Burland (1990) also made comparisons between the void ratio at the liquid limit and both \( C^* \) and \( e_{100}^* \). He developed empirical equations to describe these relationships, eqn 8.5, 8.6.

\[
C^* = 0.256e_L - 0.04
\]  
\[
e_{100}^* = 0.109 + 0.679e_L - 0.089e_L^2 + 0.016e_L^3
\]  

The data from Burland (1990) used to define this relationship is reproduced in fig 8.7, together with data from Schreiner (1991) and that from this study. The additional data compliments the relationship.

If we now add the data from those soils which lie below the plasticity "A" line, fig 8.8 we see that the overall relationship is now not as definitive. Viewing these additional data in isolation however, fig 8.9, we see that although the data set is limited, the trend forms a line parallel to that for the clay "soils", but displaced upwards. The relationship can be described by an amended form of equation 8.6:

\[
e_{100}^* = 0.409 + 0.679e_L - 0.089e_L^2 + 0.016e_L^3
\]

8.2.3 Discussion

The use of the intrinsic properties of a soil as a basis for the interpretation of soil behaviour allows the influence of a particular soil parameter to be isolated for study.

The data from both the literature and this study fit the concepts proposed by Burland (1990) well. The limitations suggested, in terms of the plasticity chart, are well founded. It has been shown that soils which fall below the "A" line behave in a subtly different way to those above. By applying this distinction, it has been shown that these isolated soils can be treated together, and behave in a similar fashion.

The experimental data defined in this study can be used to assess the influence of suction on the overall behaviour of the soil

8.3 SOIL MOISTURE CHARACTERISTICS

The relationship between soil moisture, suction, degree of saturation and volume change is a fundamental one for both fully and partly saturated soils
We will examine the controlled drying and wetting of the four clay samples. The preparation techniques are outlined in section 7.4. Generally the filter paper method was used throughout, although a refined technique, involving the IC tensiometer was applied during wetting at low suctions (see section 7.4.2.2).

Two additional, brief studies are also presented:

i) The rate of moisture loss through free evaporation from a soil surface, and the instigation of cracking.

ii) The moisture content variation through a sample during air drying.

8.3.1 Filter paper calibration

As discussed in section 4.3.2 there are a number of calibrations for the filter paper. The filter papers used are Whatman number 42. The calibration used whilst drying the samples is that published by Chandler & Gutierrez (1986). Drying is defined as placing a dry filter paper which absorbs moisture from the sample. For wetting up of the samples the data published by Ridley (1995) has been used together with data from this research to establish a revised calibration. Wetting up being defined as placing an initially wet filter paper which subsequently loses moisture to the sample. The calibrations used are shown in fig 8.10. The calibration equations used are given in table 8.3

A comparison was made between dry filter paper and suction probe measurements on a sample of London Clay. These results are presented in fig 8.11. Dry filter papers were placed top and bottom of the sample and allowed seven days to equilibrate. The papers were removed and the equivalent suction determined. Suction probe measurements were then made on both faces of the sample. The agreement between both methods of measurements is good. Also presented in figure 8.11 is a single measurement made during wetting up, allowing comparison to be made of this calibration.

It can be concluded that the methods used for measurement of suction (filter paper and suction probe) are in general agreement with one other. Also, it is noted that the calibration for the filter paper is dependent on whether the filter paper is absorbing or losing moisture as it moves towards equilibrium with the sample.

8.3.2 Drying and wetting behaviour

In this section we will consider the behaviour of the four soils undergoing drying, and in some cases subsequent wetting cycles. The soils will be considered on an individual basis, and results presented as
such. General conclusions will be drawn at end of this section.

8.3.2.1 Kaolin

The relationship between soil moisture and the drying and wetting behaviour is shown as a summary plot in figure 8.12. This gives a global view of the interaction between void ratio, degree of saturation, moisture content and suction.

The interrelation between individual parameters will be discussed with reference to their respective individual plots.

Void ratio - Moisture content (fig 8.13)

The initial drying follows the saturated drying line until a moisture content of approximately 31% is reached, at a void ratio of 0.8, from which point desaturation commences. The drying path departs significantly from the saturated equilibrium line reaching the shrinkage limit of the soil at a moisture content of approximately 29% and void ratio of 0.76.

The shrinkage limit is a back analysis from a dry state assuming a perfectly rigid media. The actual moisture content at which shrinkage ceases is 26.5%.

The drying process was continued to an air dry condition, at a moisture content of 1.5%.

Upon initial wetting, the behaviour retraces that followed during the latter stages of the drying quite closely, although the actual wetting path lies at a slightly lower moisture content for a comparable void ratio.

The approach to the saturated equilibrium line is more gradual on wetting, with apparent full saturation not occurring until the moisture content reaches 34%, at a void ratio of 0.85.

The separation between wetting and drying is indicative of the hysteresis associated with moisture movement in a porous media.

Void ratio - Suction (fig 8.14)

There is noticeable scatter during the initial drying phase, and the trend chosen is influenced more by the considerable amount of drying and wetting data at suctions in excess of 1500 kPa than by the isolated data...
between 600 - 1500 kPa. Several distinct phases of behaviour can be seen in this relationship.

The initial drying phase is as one would expect from an over-consolidated soil undergoing mechanical compression - noting that the sample preparation involved consolidation from a slurry to a vertical stress of 200 kPa. An apparent yield at a suction of about 250 kPa separates the relatively stiff over-consolidated response from the subsequent, normally consolidated behaviour. The relationship tends towards a linear one at suctions above 500 kPa.

From a suction of approximately 900 kPa the rate of change in void ratio with increasing suction begins to decrease, with a significant reduction in gradient above a suction of 1350 kPa. This coincides with the point of desaturation of the Kaolin sample.

The void ratio changes little from this level of suction, and the shrinkage limit is quickly approached, indeed the overall change in void ratio between 1350 kPa and the maximum measured suction, at 35 MPa, is minimal.

As one might expect the wetting phase follows the drying path quite closely until the shrinkage limit is transgressed once more. Thereafter, there is a gradual swelling of the sample as the suction is reduced. There is a significant non-closure between the initial and final void ratios at low suctions. This is to be expected as the sample is now in an overconsolidated state, whereas during a significant proportion of the drying phase the sample was normally consolidated. This hysteresis reflects the irreversible structural and fabric changes that will have occurred during the initial drying phase.

Adding the intrinsic compression line allows us to compare mechanical compression with that induced by drying. Following the initial overconsolidated behaviour we can see that the drying line approaches but does not reach the ICL. It can be interpreted that both are parallel between an applied stress of 400 - 1100 kPa, after which point the sample undergoing drying desaturates.

*Degree of saturation - Moisture content (fig 8.15)*

The drying and wetting paths follow very similar paths in this plot. During drying, an initial period of full saturation is followed by a brief transition until the degree of saturation falls below 90% (31% - 27% moisture content), thereafter the relationship is practically linear.

Noting that the sample quickly reaches its shrinkage limit following desaturation (fig 8.13), it would be expected that the relationship between degree of saturation and moisture content to be linear. This is confirmed by considering equation 8.8 where
\[ S \cdot e = W \cdot G_s \quad (8.8) \]

\( S \) = degree of saturation  
\( e \) = void ratio  
\( W \) = moisture content  
\( G_s \) = specific gravity

When the sample reaches its shrinkage limit the void ratio becomes a constant, thus equation 8.8 can be rewritten:

\[ S \propto W \quad (8.9) \]

During wetting up the degree of saturation - moisture content relationship remains constant until the degree of saturation reaches 85%, at a suction of about 800 kPa. From this point the sample swells and the relationship becomes non-linear. The sample approaches full saturation at a moisture content of 34%, equivalent to a suction of 100 kPa.

The hysteresis exhibited between the drying and wetting phases of this relationship at degrees of saturation below 80% is slight, and indicative of only minimal structural and fabric changes to the solid phase of the sample once desaturation has occurred. The separation can be better attributed to the effects of causing the moisture to flow into rather than out of the sample.

**Degree of saturation - Suction (fig 8.16)**

The relationship between degree of saturation and suction shown in fig 8.16 gives a good experimental demonstration of the drying / wetting behaviour of porous media as discussed in chapter 2.

Drying from a fully saturated state, we see the ability of the sample to hold moisture, remaining fully saturated to quite a high level of suction, 1300 kPa. At this point the volumetric strain is 13.5% and the moisture content 30.6%. As the sample dries it also shrinks, ensuring it remains fully saturated. The point of desaturation is quite distinct, and marks the stage where the pore structure can no longer support a pore water meniscus of the required radius.

The degree of saturation decreases very rapidly from this point. At a suction of 2000 kPa, \( S_r = 66\% \) and at a suction of 5000 kPa the degree of saturation has reduced to 22%. This quite dramatic reduction in degree of saturation for a relatively small change in suction is influenced by two important factors:
particle size distribution

The particle size distribution for Kaolin is given in chapter 6, fig 6.1. It shows a uniform grading with 25% of the sample falling within the fine silt category. If we include the material between 0.001 - 0.002 mm (classed as "non-colloidal" on the ASTM grading classification) this proportion now accounts for 50% of the sample. With half the sample falling into quite a narrow grading, one would expect that the pore structure would also be of a uniform nature, with a small range of pore sizes accounting for a significant percentage of the total pore space.

In chapter 2 we discussed the mechanisms of desaturation, and noted that the pore size would have a dominant role to play. Thus if a narrow range of pore sizes exists, one would expect that moisture would be lost from these pores over a small suction range.

Clay mineralogy

The dominant clay mineral present is kaolinite. It is the least active of the clay minerals and as such displays a comparatively small amount of shrinkage during drying. As the point of desaturation is approached we see that the sample is already approaching its shrinkage limit. Thus with little further shrinkage occurring, the sample can be viewed as a rigid porous medium. This rigidity of the structure means that the pore openings will not change in size with increase in suction (beyond the shrinkage limit), thus allowing desaturation to occur readily.

As the degree of saturation reduces below 40% a more noticeable increase in suction is required to remove the remaining moisture. This is probably indicative of the removal of moisture that is retained by the clay particles and as such more strongly held. At the conclusion of drying, the degree of saturation is approximately 7%, and the volumetric strain 17%.

The wetting up path shows a distinctly hysteretic behaviour with respect to the drying phase. This experimental data confirms the discussion in chapter 2 on the drying and wetting of a porous media. The separation between the drying and wetting paths can be largely accounted for by reference to the pore fluid contact angles, pore constriction and geometry (see section 2.6)

Additionally there will be some effect due to the structural and fabric changes that occur as a result of the first time drying curve. The drying is of a largely normally consolidated soil whereas the wetting is on an over-consolidated material. The extent of the structural changes is clearly demonstrated in fig 8.14, where we can see a final void ratio (following wetting) which lies some way below the initial void ratio at the commencement of drying. We can also note from fig 8.14 that there is not much indication of volumetric hysteresis (with respect to level of suction) until the suction reduces below 1500 kPa. In terms
of degree of saturation however, (fig 8.15), the separation between drying and wetting paths is at its greatest at this point. This is indicative of the effects of hysteresis in moisture flow.

Moisture content - Suction (fig 8.17)

The relationship between moisture content and suction displays trends in its behaviour which relate to both fig 8.14 and fig 8.16.

The drying behaviour displays an initially over-consolidated type response to an increase in applied stress (suction). Upon desaturation, there is a significant reduction in the moisture content for a relatively small change in suction. This effect reduces below a moisture content of 15%.

Upon wetting, an element of hysteresis develops from the outset, due primarily to pore fluid and pore structure influences. Once the suction reduces below about 800 kPa (at which point the degree of saturation is above 80 %), the shape of the wetting curve matches that of the drying curve quite well. There is a clear separation between the two which is caused by permanent structural and fabric changes as a result of shrinkage during drying.

Marinho (1994) defined the linear phase of the drying for a normally consolidated soil at suctions below the point of desaturation as the suction capacity, eqn. 8.10

\[
C = \frac{\Delta W}{\Delta \log(suction)}
\]

8.10

This parameter has been defined by a number of researchers (Marinho, 1994) and gives an indication of the shrinkage abilities of a soil. It is unique to a particular soil and stress history, and can be compared to the intrinsic compression indices defined in terms of moisture content. The Kaolin did not display a distinct, linear phase during drying. The equivalent value from the intrinsic compression line is 15.4.

8.3.2.2 London Clay

A summary plot of the relationship between soil moisture and the drying and wetting behaviour is shown in fig 8.18. This gives a global view of the interaction between void ratio, degree of saturation, moisture content and suction.

The interrelation between individual parameters will be discussed with reference to their respective individual plots.
Void ratio - Moisture content (fig 8.19)

The initial drying follows the saturated equilibrium line until the moisture content reduces to 22.5%, at a void ratio of 0.57 and volumetric strain ($\varepsilon_v$) of 28.4%. A gradual departure from the fully saturated state occurs from this point (desaturation). At the maximum suction which can be measured by the filter paper ($\approx 35$ MPa) the shrinkage limit of the soil had not been reached. It has been estimated to be at a moisture content of 17%, void ratio of 0.42. The actual condition under which the sample ceases to shrink has also been estimated, at a moisture content of 6.1%, void ratio = 0.42, suction $> 35$ MPa and $\varepsilon_v = 35\%$. This represents the air dry condition, the general trend suggests that further shrinkage is unlikely beyond this point.

The wetting path separates from the drying path from the outset. The wetting path lies at a lower moisture content for an equivalent void ratio and moves gradually towards the fully saturated line but does not reach it. From a moisture content of about 27% the wetting path moves parallel but adjacent to the saturated line.

Void ratio - Suction (fig 8.20)

The first measurements of suction during drying were at about 200 kPa. At this level of suction the overconsolidated behaviour (due to sample preparation) is not seen. An isotropic stress of 200 kPa is equivalent (in normally consolidated London Clay) to the application of a vertical stress of 250 kPa. The maximum applied vertical stress during sample preparation was 200 kPa.

The path is fairly linear up to a suction of 1000 kPa, beyond this a gradual curve develops, with a reduction in the rate of change of void ratio. This trend continues to the maximum measured suction.

The wetting phase of the test follows a predictable path. From the air dry state, little increase in void ratio is seen as the suction is initially reduced. As the suction is reduced further the void ratio - suction gradient increases initially, reaches a point of inflection at about 2000 kPa and then decreases thereafter.

Upon completion of the wetting phase, at a suction of 80 kPa, it is apparent that there is still some swelling potential in the sample.

A comparison between mechanical (ICL) and drying induced compression shows good agreement up to a suction of about 1000 kPa. Up to this point the ICL is determined experimentally, beyond this value it is calculated using the general expression given by Burland (1990).
Degree of saturation - Moisture content (fig 8.21)

The drying and wetting phases follow similar paths, although there is greater separation than was seen for the Kaolin sample.

The London Clay remains saturated over a considerable change in moisture content, from an initial value of 42% until desaturation occurs at 22.5%. A transition phase follows desaturation, with the trend becoming broadly linear for moisture contents below 16.5% (a degree of saturation below 90%). This behaviour can be attributed to the fact that only a small change in void ratio occurs over the final stages of drying, thus allowing a linear relationship to be established, (see equation 8.9).

During wetting up, a linear relationship between degree of saturation and moisture content is seen in the initial stages until the moisture content reaches 15% (a degree of saturation of 80%). A much greater transition phase follows than was seen in the drying phase, with a constant degree of saturation not reached until the moisture content increases to 27%. The sample did not reach full saturation but stabilised at 97%.

Degree of saturation - Suction (fig 8.22)

This figure gives a good indication of the extent of hysteresis which can occur between wetting and drying cycles. It should be remembered however that there is some influence due to the permanent deformation which occurs as a result of the first time drying.

During drying we see that the sample remains fully saturated up to a suction of 3 MPa. Following a transition, the degree of saturation - suction relationship is linear between suctions of 12 MPa and 35 MPa. If we look at the particle size distribution for the London Clay, fig 6.2, we see a well graded material with a clay content of about 70 % and the remaining material spread across all silt categories equally. This, coupled with a more active clay mineral than the Kaolin ensures that:

i) The point of desaturation will be quite high (clay content)

ii) There will be no significant change in degree of saturation at any particular suction (Broad spread of particles sizes , hence pore sizes)

iii) The degree of saturation will reduce on a gentle, continuous gradient to very high suctions (high and active clay content)

The wetting up relationship deviates from the drying path significantly, from the onset of wetting. This separation may be accentuated by the fact that it was not possible to continue the drying phase to a low, residual degree of saturation. As the drying gradient is quite steep at the commencement of wetting, any
Hysteresis at this point is magnified.

This is supported when one considers the same relationship for Kaolin where it was possible to dry the sample to a residual degree of saturation. We saw little increase in degree of saturation following a reduction in the suction from 35 MPa to 20 MPa, however as the curve is very flat at this point, the separation between drying and wetting curves it is not as obvious.

During the latter stages of wetting the path moves towards but doesn't reach full saturation. The maximum degree of saturation is 97% which is reached at a suction of 500 kPa.

**Moisture content - Suction (fig 8.23)**

Measurements of the drying phase of this test commenced at a suction of 200 kPa, as such the drying behaviour presented is of a normally consolidated material.

The suction capacity "C" can be estimated from the drying phase between a suction of 300 kPa and 2000 kPa, giving a value of 17.5. The equivalent value from intrinsic, one-dimensional compression is 19.8. The drying trend is one of a gentle curve. From the available data it is tending towards zero moisture content, although it is not certain that this trend would continue. The final measured moisture content is 7%.

Wetting up displays the same form of hysteresis seen in fig 8.22, with a distinct separation occurring between drying and wetting paths from the outset of wetting up.

The gradient of the wetting curve is broadly parallel to the drying curve once the suctions have reduced below 4000 kPa (degree of saturation > 80%). From 1000 kPa, the rate of change of moisture content reduces, tending towards a final moisture content of about 31%. This is considerably less than the initial moisture content, at 200 kPa (m/c_s = 42%, m/c_f = 27%) and this difference can be attributed to the permanent shrinkage which would have occurred during the initial drying phase.

### 8.3.2.3 Bangali Clay

Two samples of Bangali Clay were used for the determination of the soil moisture characteristics. The expansive nature of the material led to a number of handling difficulties.

During the drying phase, the samples would show signs of distress, and begin to crack if allowed to dry in the open laboratory atmosphere. As it was necessary to allow the sample to dry between measurements in order to cause a reasonable increase in suction at each stage, a sealed environment was created which
reduced the rate of evaporation sufficiently such that cracking did not occur (see section 7.4)

During wetting however it was not possible to exert the same control over the addition of moisture. Placing two wet filter papers on each sample caused an initial, uncontrolled addition of moisture. This resulted in differential swelling as the wetting front advanced through the sample, causing extensive cracking.

Although two samples were used - trimmed from the same initial sample - in an attempt to ensure a complete data set could be obtained, both samples broke up upon wetting, and it was not possible to make volumetric measurements during the wetting up phase.

A summary plot for each sample is given fig 8.24 (BA1) and fig 8.25 (BA2). It should be noted that there is excellent agreement between the results from both samples and therefore they can be considered as a single data set.

*Void ratio - Moisture content (fig 8.26)*

The saturated drying phase of this material is extensive. From an initial moisture content of 61%, the sample remained saturated until the moisture content had reduced to 23%, at a void ratio of 0.62 and volumetric shrinkage ($e_{vol} = 39\%$). Drying continued to a suction of about 25 MPa, at which point the moisture content had reduced to 16%, void ratio 0.51 and $e_{vol}$ 42%. The laboratory air dry moisture content is 9.3%. This was determined from a separate test (it required fours days continuous exposure to the laboratory atmosphere to reduce the moisture content from 16% to 9.3%).

Although both partly saturated and under the influence of a significant suction, the soil had not reached the limit of shrinkage at the conclusion of drying. The shrinkage limit is estimated to be at a moisture content of 18% and a void ratio of 0.5, however there is little data to support this estimation.

Both samples were wetted up using the filter paper technique following the conclusion of the drying phase. It was not possible to make volumetric measurements during the wetting up phase. At maximum suction during drying the sample was still close to the saturated equilibrium line and it is likely that it would move back towards, if not onto this line as the moisture content increased, probably before the moisture content reached 30%

*Void ratio - Suction (fig 8.27)*

Suction measurements commenced at 150 kPa and continued up to in excess of 25 MPa.
There is some evidence of over-consolidated behaviour in the early stages of drying. Between 400 kPa and 5000 kPa suction the relationship is almost linear. Above 5000 kPa the gradient begins to reduce and more pronounced curvature develops, although at the maximum suction the void ratio is still reducing. On the evidence presented here, if the asymptotic tendency were to persist, shrinkage might continue up to 100 MPa.

The intrinsic compression line can be plotted on this graph. This was determined experimentally between 100 and 1000 kPa vertical stress, but can be extended using the expression given by Burland (1990). Up to 1000 kPa suction the drying and intrinsic compression lines do not agree well. This can probably be attributed to the effects of over consolidation in the drying samples. Between 1000 kPa and 10,000 kPa the agreement is very good.

At the conclusion of drying the sample was already showing some signs of distress. Cracks were beginning to form, both laterally and radially in both samples. These may have influenced the determination of the void ratio at these very high suctions, although the cracks were visible but not open at this stage.

The wetting phase caused a significant increase in both the number and severity of the cracks, ultimately causing the samples to break into a number of separate pieces, thus precluding the determination of the void ratio.

Whilst the moisture content and suction were monitored, and at the conclusion the degree of saturation was still very high (80%), it would be unwise to attempt to infer a void ratio from the moisture content data. We have seen from the London Clay and Kaolin samples that the degree of saturation can continue to change for a considerable period during wetting up and this would introduce a systematic and varying error to all estimations of void ratio from moisture content and degree of saturation.

\textit{Degree of saturation - Moisture content (fig 8.28)}

We can see very clearly the effect a high and active clay content has on the ability of a soil to retain soil moisture during shrinkage.

From an initial moisture content of 61%, the sample reduces to 23% before desaturation occurs. At this stage the volumetric strain is 39%. The soil is still moving through a transition zone at the conclusion of drying. The final condition of the sample is a moisture content of 15%, suction of 25 MPa and a degree of saturation of 81%.

The ability of the soil to retain moisture at such high suctions is remarkable. The main influence is the
presence of montmorillonite clay minerals. The particle size distribution for Bangali Clay is much more well graded than either the London Clay or Kaolin samples. With 10% sand and 25% silt present one would expect a gradual loss of moisture over a wider range of suction (reflecting the wider range of pore sizes). However the presence and dominance of montmorillonite clay minerals allowed the sample to remain fully saturated over a wide range of suctions and precluded the investigation of this effect.

*Degree of saturation - Suction (fig 8.29)*

The measurement from both samples give an excellent degree of correlation. The Bangali Clay remains fully saturated until the suction reaches 9 MPa, at which point desaturation is seen. At the maximum measured suction, 25 MPa, the sample remains over 80% saturated. At this stage the soil appears to be reaching a linear trend, although there is insufficient experimental data to be conclusive.

*Moisture content - Suction (fig 8.30)*

We can see some evidence of overconsolidation in the initial stages of drying in fig 8.30.

An estimation of the suction capacity can be made between 350 - 1500 kPa, giving a value of 23.5. This compares with a value of 30.4 from the intrinsic compression data.

A gradual reduction in gradient is seen as the suction reaches higher values (>4000 kPa). The moisture content is 15% at the conclusion of the drying phase but is still reducing. The likely residual moisture content is not clear from this plot, although the air dry moisture content is 9.3%.

The wetting up data forms a trend similar to that already seen for the London Clay. There is some similarity between the drying and wetting data. The shape of the wetting curve is analogous with the drying curve, although the gradients are reduced. Following a flat gradient upon initial reduction in suction, the gradient increases to a peak at about 3000 kPa and thereafter reduces towards a stable moisture content. At the conclusion of the wetting phase the suction was about 200 kPa and moisture content 38%.

8.3.2.4 Black Cotton Clay

Two samples of Black Cotton Clay were trimmed from a single, larger sample which had been consolidated one-dimensionally from a reconstituted state.
As one might expect for an expansive clay, significant shrinkage occurred during drying, which highlighted two problems.

Firstly allowing the samples to dry in the laboratory atmosphere for more than ten minutes caused cracking to occur. This necessitated a higher humidity environment to be established that would reduce the rate of evaporation sufficiently to allow the samples to dry without cracking (see section 7.4). Secondly, due to the high moisture content of the sample, the drying process took a considerable period of time (40 weeks).

At the conclusion of the drying phase both samples remained intact, although one (BCC1) had a large, open crack radially across the sample. Two unresolved problems prevented the continuation of the test to generate wetting data. Firstly, the length of time required to wet the sample would exceed that for drying, and has been estimated to take a further 18 months, placing its conclusion beyond the limits of this study. Secondly, a modified method of wetting up would need to be established if the sample were not to break up in the same way the Bangali Clay samples did. For these reasons it was decided not to continue with the wetting up process of this sample.

Summary plots of the behaviour of the two samples are given, fig 8.31 (BCC1) and fig 8.32 (BCC2). The radial crack began to develop significantly in sample BCC1 at about 2000 kPa and precluded the continuation of volume measurements above this suction. Again there is excellent agreement between the two samples and they can be considered as one data set.

void ratio - Moisture content (fig 8.33)

The initial moisture content, following sample preparation, was 78%. The soil remained fully saturated until it reduced to a moisture content of 23% and void ratio of 0.603, at a volumetric strain of 47%. Drying continued to the limit of the filter paper method (≤35 MPa), at which point the sample had a moisture content of 16.5%, void ratio of 0.5 and had undergone volumetric straining of 50%.

At the conclusion of drying the sample remains close to the saturated equilibrium line and is still shrinking considerably. It is not possible to estimate the shrinkage limit from the data available.

In a separate drying test the moisture content of a Black Cotton Clay sample was measured at 11.2% after 8 days of continuous drying in the laboratory atmosphere. During this test it took three days to reduce the moisture content from 16.5% (filter paper limit) to 11.2%. At this stage the sample still appears to be losing moisture very slowly and although tending to a constant value it had not reached a stable, residual moisture content.
Void ratio - Suction (fig 8.34)

Suction measurements were made from 180 kPa to 35 MPa during the drying process in a continuous test.

The initial suction data gives no indication of over-consolidation due to sample preparation, which would be expected at the low suctions.

There is a gradual reduction in gradient as the suction increases, giving a gentle curvature to the void ratio - suction relationship.

The intrinsic compression line can also be plotted on fig 8.34, allowing a comparison between mechanical compression and shrinkage induced by drying. Agreement is best at suctions below 1000 kPa, becoming poor at higher suctions. Although of a similar shape, the intrinsic compression line suggests a more compressible material throughout.

Degree of saturation - Moisture content (fig 8.35)

The Black Cotton Clay remained fully saturated during a significant reduction in moisture content. The sample moisture content reduces by 55% before the sample desaturates. Continued drying, to a moisture content of 16.5% only reduces the degree of saturation to 86%.

At this stage the sample is still going through a transition stage and the air in the sample is likely to exist as either discrete bubbles within the sample or at the surface of the sample as the soil moisture menisci begin to retreat.

Degree of saturation - Suction (fig 8.36)

The point of desaturation is seen at 13 MPa. At the conclusion the suction in the sample is 35 MPa and degree of saturation is 86%. The sample is moving between behaving as a fully saturated soil and one with a continuous air phase and interparticle menisci.

Moisture content - Suction (fig 8.37)

As one would expect, the moisture content suction relationship, fig 8.37, is very similar to that for the void ratio - suction (fig 8.34) as the sample remains almost fully saturated throughout the drying process.
The suction capacity of the soil can be estimated, between 1,000 kPa and 10,000 kPa suction, at 33. The value from the intrinsic compression line is 40. One factor that would influence the ICL data to give a higher "suction capacity" is that the interpretation is made at a much higher stress level on the drying curve, and hence on a lower relative gradient. The ICL value is still however a noticeably larger value even when due allowance is made for this factor.

8.3. Discussion

We have looked experimentally at the behaviour of a range of soils under drying and wetting cycles. The form of an idealised drying curve for a porous rigid media, shown in figure 8.38 has three distinct phases:

Stage 1 Increasing suction under full saturation. A feature of the pore structure and clay mineralogy, the sample will remain fully saturated until the pore structure can no longer support a pore water menisci of the required radius (see section 2.4). Desaturation will occur if the suction is increased beyond this point.

Stage 2 Reduction in moisture content / degree of saturation with increasing suction. Whilst the pore size controls the point of desaturation the range of pore sizes will influence the gradient of this phase of the soil moisture characteristic curve. A shallow gradient is indicative of a wide range of pore sizes with a significant increase in suction required to reduce the moisture content substantially. If a narrow range of pore sizes predominate, moisture loss will occur for a much smaller increase in suction.

Stage 3 Residual moisture content / degree of saturation. This is generally defined as the stable, constant moisture content / degree of saturation for a soil which has been allowed to dry fully under atmospheric conditions.

The experimental data, whilst displaying the above general tendencies, are often considerably different from this idealisation. A number of factors influence the form of the soil moisture - suction relationships. The two most influential factors are related to the presence of clay minerals. A clay soil will undergo volumetric changes whilst remaining fully saturated, and have a direct attraction for moisture in addition to the effects of capillarity. This results in the following, observed behavioural patterns:

- Moisture loss and shrinkage under full saturation. Shrinkage under full saturation is a feature of all clay soils. The extent to which this occurs is controlled by stress history, grading and clay mineralogy. From the experimental data, in which all samples had the same initial stress history, we can see that the dominant influence appears
to be clay mineralogy. The compression indices ($C_*$) from mechanical compression range from 0.396 for Kaolin to 1.074 for the Black Cotton Clay. This can be generally compared to the shrinkage as a result of drying whilst the sample remains saturated. The extent of fully saturated shrinkage is also enhanced by the increased air entry value for the more expansive clays.

• **Transition at air entry value and reduction in moisture content / degree of saturation with increasing suction**

The pore structure and clay mineralogy complement each other in influencing desaturation and continued moisture loss during drying. We can infer a pore size distribution from the particle size distribution (PSD). Kaolin excepted, the samples can all be classified as well graded, which one could expect to yield a range of pore sizes. Thus moisture will be lost over a wide range of increasing suction. The Kaolin PSD shows a more uniform grading and this leads to a more pronounced point of desaturation and loss of moisture over a much smaller suction range.

The more gradual nature of moisture loss observed in the London, Bangali and Black Cotton Clays is enhanced by continued shrinkage with increasing suction. This will have the effect of reducing the effective pore size as the suction increases, thus allowing the sample to both retain moisture to a higher level of suction, and restrict the rate of loss of moisture.

• **Continued shrinkage at low moisture content.**

We have seen that the more expansive soils have a greater ability to retain moisture under high suctions. These samples continue to reduce in volume over a considerable suction range. The Bangali and Black Cotton Clays were still undergoing significant shrinkage at the conclusion of the drying phase, at a suction of 35 MPa. At this point it is unclear how much further these samples would continue to shrink. This behaviour is far removed from that depicted in the idealised case.

A schematic comparison between real and idealised behaviour is shown in fig 8.39. We have seen that although most of the soil moisture characteristic curves have a similar generic form there can be significant differences in the particular relationship.

It is clear from the data presented that the definition of the air entry value and the shrinkage limit require some consideration in respect of these highly plastic soils. The AEV is defined in fig 8.40 and the shrinkage limit in fig 8.41. On the respective figures the actual point at which air enters the sample, and at which shrinkage ceases has also been indicated. We can see that the accepted definitions have little physical meaning, and perhaps need revising to the actual and not extrapolated values. Ridley (1993) suggested that the physical limit of shrinkage be termed the volumetric limit, this has some merit. The extrapolated AEV may have uses in the development of conceptual and predictive models but it should be identified as the idealised and not actual point of air entry.
Modelling soil moisture characteristics

A number of models depicting the behaviour of drying and wetting soils have been proposed. One of the main difficulties lies in accounting for the both fully and partly saturated behaviour in compressible soils, particularly the transition between these two aspects. Each aspect can be modelled individually but it has not been possible to compute the full relationship. Toll (1988, 1995) proposed a simple conceptual model which combined both saturated and partly saturated behaviour under drying and wetting paths.

He enhanced the saturated soil mechanics parameters with an additional void ratio term, the equivalent void ratio, \( e_w \), which would represent the partly saturated behaviour, equation 8.11

\[
e_w = \frac{\text{volume of water}}{\text{volume of soil particles}}
\]

\[
e_w = \frac{\theta_w}{1 + e}
\]

\[
e_w = w G_s
\]

\[
e_w = e S_x
\]

The drying and wetting concepts are shown in fig 8.42. The saturated drying line, termed the Virgin Consolidation Line (VCL) is equivalent to mechanical compression of the sample. Whilst the sample remains saturated \( e \) and \( e_w \) follow the VCL. From the point of desaturation both void ratio parameters diverge. The void ratio \( (e) \) - suction path reduces in gradient as the sample moves towards its shrinkage limit. Conversely the equivalent void ratio \( (e_w) \) - suction path increases in gradient as the degree of saturation term enhances the reduction in \( e_w \). As the degree of saturation tends towards a constant, residual value, \( e_w \) also tends towards a constant value. Toll also made allowances for an initially over consolidated soil, and for both wetting and redrying paths in his model.

The drying and wetting data from the Kaolin and London Clay samples can be used to assess the validity of this concept. Fig 8.43 shows the Kaolin data and fig 8.44 the London Clay.

Both soils exhibit the predicted form of behaviour. The separation between \( e \) and \( e_w \) marks the point of desaturation quite clearly. The Kaolin has reduced to a low residual degree of saturation at the conclusion of drying, indicated by the extent of separation between \( e \) and \( e_w \) and the constant value of \( e_w \). The void ratio and degree of saturation are both still reducing for the London Clay at the conclusion of drying. This is indicated by the gradient of \( e \) and the limited separation between \( e \) and \( e_w \).

This model is useful in visualising the general behavioural changes which occur as a result of drying and wetting.
Prediction of soil moisture characteristics

The physical determination of the soil moisture characteristic curve is a very time consuming process. The measurements made for this study took 18 months of continuous recording, and in the case of the Black Cotton Clay are as yet incomplete. An ability to predict the form of these relationships could prove to be a very useful tool. There are two distinct requirements, firstly to derive an expression which can describe the empirical behaviour of a soil undergoing drying, and secondly to predict this behaviour from a limited data set.

A number of research workers have published expressions describing experimental data. Fredlund and Xing (1994) reviewed a number of these. They concluded that most were empirically based and generally limited to a particular group of soils. They also presented a new form of expression which they propose could model the characteristics from a wide range of soil types, eqn 8.12

\[
\theta(\psi, a, n, m) = C(\psi) \frac{\Theta_s}{(\ln[e + (\psi/\Theta_s)^n])^m}
\]

8.12

\[
C(\psi) = \frac{\ln(1 + \psi/\psi_t)}{\ln[1 + (1000000/\psi_t)]}
\]

where:

- \(a\) is a variable related to the air entry value of the soil
- \(n\) is a variable related to the slope of the partly saturated phase of the soil
- \(m\) is a variable related to the air dry moisture content of the soil
- \(\Theta_s\) = initial saturated moisture content
- \(\psi\) = matrix suction
- \(\psi_t\) = matrix suction at the air dry moisture content

There are two methods for determining the constants \(a, n, \) and \(m\). Take the air entry value, slope of the drying line, and air dry moisture content as \(a, n,\) and \(m\) respectively as a starting point and by a process of manual iteration determine the constants required for the best fit line. Alternatively, Fredlund and Xing include some non-linear curve fitting algorithms which allow the most suitable constants to be calculated. This does however make the expression significantly more complex.

The definition of the initial values of the constants and the effect that they have on the shape of the curve is shown in fig 8.45. The above expression can be used to represent the soil moisture characteristic in terms of degree of saturation, volumetric or gravimetric moisture content. They present data for a number of soils, although all are principally granular materials (sands, silts and silty loams). Equation 8.12 was applied to the data from the Kaolin and London Clay to investigate its applicability in describing clay soil data, and also to consider its possible use in predicting the soil moisture characteristics.
If we consider the degree of saturation - suction relationship, fig 8.46 and 8.47 we see that it is possible to obtain quite a good agreement if the full data set is accessed in each case. The limitations of the curve fitting appear to be in accounting for the sharp deviation in the Kaolin data at the point of desaturation.

As the full drying curve data are available we can use a limited part of the data set and explore the ability to predict the full curve using equation 8.12. Two partial data sets were used, relating to a degree of saturation >80%, and to a degree of saturation >70% respectively. In each case the best fit to the data was determined by manual iteration. The predicted relationship is for both the London Clay and Kaolin are plotted on figs 8.46 and 8.47.

The Kaolin gives a better fit at the point of desaturation for both limited data sets, but this is at the expense of accuracy in the extrapolated, higher suction range.

The London Clay prediction also loses accuracy in the high suction range when the limited data sets are used, this is particularly evident with the smaller data set (Sr>80%).

The equations proposed by Fredlund and Xing (1994), whilst providing a useful fit to experimental data, appear limited in their ability to extrapolate a relationship beyond the available data. The data set needs to be sufficient such that it is sensitive to changes in any of the variables. For a reasonable prediction it would seem necessary to have data which included at least part of each of the three phases of drying behaviour outlined in fig 8.38. This would allow the influence of each of the variables to be assessed with both a degree of sensitivity, and a level of confidence. However this would require obtaining the majority of the data experimentally before a reasonable prediction could be made.

The gravimetric moisture content - suction relationship was also assessed for the London Clay, fig 8.48 - 8.49. The experimental data set contains compressible saturated, transitional and partly saturated phases. The form of the data is similar to that depicted by the equivalent void ratio, e_v (Toll, 1995).

A distinct change in trend is seen at the point of desaturation, and it is not possible to fit a single curve to describe the overall behaviour accurately using eqn 8.12. This deviation is not apparent when the data is viewed in terms of degree of saturation, as saturated compressibility is not explicit in this form. If we treat the saturated and partly saturated phases separately, we can develop an expression for each, fig 8.48 - 8.49, although the partly saturated data set has limited sensitivity to the variables in the equation.

Summary

We have looked at the measured relationships between volume, moisture content and suction for a number of soils under drying and wetting cycles.
The ability of a soil to retain moisture is directly related to the size, shape and distribution of pores within the soil. It is also related to the clay mineralogy, and the inherent attraction the clay particles have for moisture.

As discussed in section 2.5, the accurate measurement of the pore size distribution is a complex and involved process, beyond the scope and main aims of this project.

From simple theory we can say that a soil containing coarse grains will have a more open pore structure than one containing a finer grading. Equally, a well graded soil - with a broad range of particle sizes - will have a wider range of pore sizes than a uniformly graded soil.

Clearly the soil moisture characteristics will be affected by the pore structure. A soil containing a broad range of pore sizes will display a gradual transition during desaturation, and also a more gentle gradient of moisture loss during increase in suction. A soil with a more uniform pore structure will display a sharp point of desaturation and more significant moisture loss over the same suction range.

These concepts have a sound theoretical basis, and some published experimental data for confirmation.

The data from this study however shows the dominant effect of clay mineralogy. The London, Bangali and Black Cotton Clays all have similar clay contents (65 - 75%) and a well graded particle size distribution (PSD). The suction capacity varies considerably and the ability to retain moisture is marked in the expansive clays with desaturation occurring at 3, 9 and 13 MPa respectively. At the conclusion of drying (at 35 MPa suction) the degree of saturation ranged from 42% for the London Clay to 86% for the Black Cotton Clay.

Marinho (1994) performed a series of drying tests on reconstituted London Clay mixed with varying quantities of quartz silt. The air entry point of the soil did not vary greatly until the quantity of silt exceeded 30%. The shape of the grading curves were similar but the clay content (<0.002 mm) reduced from 65% for the "pure" London Clay to 45% at 30% added quartz silt.

It is not clear whether the effect of the clay minerals is to reduce the actual pore size, change its shape, or retain moisture through surface attractions, but it can be seen that the effect is considerable.

During wetting up the main observations were of the cracking of the expansive clays. This is likely to be related to a combination of low permeability, coupled with a high swelling potential and the advance of a free wetting front.
8.3.3 Evaporation, shrinkage and cracking

The initial suctions required for undertaking the suction controlled testing involved allowing the samples to air dry. Some of the samples developed cracks during this drying phase. This was also seen during the drying phase of the soil moisture characteristic determinations.

It was also noted that the more expansive the soil, the quicker the cracks formed. The Kaolin and London Clay samples showed no signs of distress throughout the drying process. The Bangali and Black Cotton Clays began to crack within 30 minutes when allowed to dry freely.

There has been much research in the fields of hydrology and agronomy on evaporation. A number of expressions have been published from which the rate of evaporation can be predicted from an equation. Most are of the form shown in eqn 8.13

\[ E = f(u) (e_s - e_a) \]  \hspace{1cm} 8.13

where:
- \( e_s \) = vapour pressure of the soil surface
- \( e_a \) = vapour pressure of the atmosphere
- \( f(u) \) = a function dependent on a number of factors relating to both the soil and the atmosphere (wind speed, turbulence, surface roughness)

The aim of this series of tests is not to assess the predictive abilities of the published expressions, but to observe the drying characteristics and develop some hypothesis as to the cause and possible prevention of cracking.

A series of tests were undertaken to investigate the air drying characteristics of the four soils from an initially high moisture content (low suction) condition through to residual moisture content.

Samples of each soil were prepared, of the same size and shape as those used for the definition of the drying / wetting behaviour.

They were each placed on an electronic balance and allowed to dry freely. The change in mass and hence moisture content was monitored continuously throughout the each test. Free water evaporation was also monitored, from a glass dish of water on a similar balance.

A comparison was made between the free water evaporation and that from the sample. Utilising the moisture content - volume relationships determined earlier from the filter paper tests a correction could be made to the surface area of the soil sample as shrinkage occurred during drying.
The results are presented in the form of cumulative specific evaporation (g/cm\(^2\)) for both the sample and the water.

The soil moisture characteristics could also be employed to identify the key phases in the sample that had been noted during the filter paper tests during drying, namely:

- An initial period of saturated moisture loss
- Desaturation
- Partly saturated moisture loss and continued shrinkage
- Limit of shrinkage
- Steady state, residual moisture content
- Sample cracking

The effect of these stages on the rate of evaporation could then be assessed.

The tests were carried out in two separate phases. The Kaolin and London Clay were dried together, followed by the Bangali and Black Cotton Clays. Photographs were taken at regular interval throughout each test.

8.3.3.1 Kaolin

The calculated specific rate of evaporation for the Kaolin, fig 8.50, exceeds that measured for the free water surface. This is clearly not possible as the maximum rate of evaporation would be that associated with a free water surface. It was noted that from the outset the kaolin sample did not shrink as a right cylinder but distorted as it dried. One effect of this distortion would be to allow some evaporation to occur from the lower face of the sample. This was not accounted for in the subsequent calculation. It was assumed that evaporation occurred from the upper and peripheral surfaces of the sample only, thus the calculated rate of evaporation would be lower than that actually occurring.

The rate of evaporation is constant until the moisture content reduces to 12.2%, at a suction of about 2500 kPa and degree of saturation of 40%. From this point the soil goes through a sharp transition, moving rapidly away from a linear rate of evaporation. Once the moisture content has reduced to 5.5% the approach to residual conditions (moisture content 1.2%, degree of saturation 4%) is quite prolonged. It takes less than three days for the moisture content to reduce from 45% to 5.5% and a further 3 days to reduce from 5.5% to 1.2%, fig 8.51.

Oven drying of the sample did not produce any significant shrinkage, and the sample remained intact on completion.
8.3.3.2 London Clay

During drying the London Clay sample, fig 8.52, remained essentially a right cylinder. As such evaporation is unlikely to have occurred to any great extent from the lower face (which was in contact with the weighing pan of the balance).

The rate of drying matches that from a free water surface almost exactly for over two days of continuous drying.

From an initial moisture content of 41% the sample passes the point of desaturation (moisture content 22.5%, suction 3000 kPa) and continues until the sample reaches a moisture content of 18% (suction 5200 kPa, degree of saturation 94%). This point coincides quite well with the shrinkage limit moisture content (17%).

It should be remembered however, that the shrinkage limit is a back analysis of the moisture content at which the sample stops shrinking and assumes that it deforms as a rigid, porous medium, with no further shrinkage occurring after desaturation. A plastic soil such as the London Clay does not behave in this way.

From a moisture content of 18%, the rate of evaporation begins to reduce. If we look at the moisture content - time plot, fig 8.53, we can see a more distinct reduction in the rate of change of moisture content after 60 hours, (moisture content 12%, degree of saturation 94%, suction 15 MPa).

At the conclusion of the 6 day drying test the residual moisture content was 6.1%. Oven drying did not cause any cracking to occur in the sample.

8.3.3.3 Bangali Clay

A very good correlation between the rate of evaporation from a free water surface, and that from the Bangali Clay is seen in fig 8.54.

There is a gradual reduction in the specific rate of evaporation from the sample below a moisture content of about 24% (still fully saturated, at a suction of 7 MPa). The deviation is slight until the moisture content reduces to 18% (S = 20 MPa, Sr = 92%). From this point there is an increasing separation between the soil and free water rates of evaporation, although the reduction is gradual throughout the drying process, fig 8.55.

It is interesting to note that this definite trend towards a residual moisture content occurs at about the shrinkage limit moisture content of the soil (18%) as was seen for the London Clay earlier.
One main difference between the Bangali Clay and the Kaolin and London Clay is that it cracked considerably as it dried. A series of photographs were taken as the Bangali Clay dried, to monitor the development and propagation of the cracks.

Cracking began after 30 minutes drying. The initial crack formed at the edge of the sample through its full depth, fig 8.56. After 1 hour the crack was 10 mm long, fig 8.57, and 30 mm after 2 hours, fig 8.58. Additional cracks developed at 4 hours, fig 8.59.

After 24 hours continuous drying two significant cracks had opened, forming a similar pattern, fig 8.60. The cracks opened about 5 mm in width and had a roughened surface, fig 8.61. The maximum extent of cracking was reached after 48 hours, and they remained in this form for a significant period. As the rate of evaporation began to reduce, the cracks began to close once more, and at 96 hours they had closed completely, fig 8.62. Once the sample reached a residual moisture content some of the cracks had opened again, but the sample was essentially dry at this stage. Oven drying caused the sample to break up completely, both radially and laterally into a number of discrete irregular pieces.

**8.3.3.4 Black Cotton Clay**

The relationship between the evaporation from the soil and that from the free water surface, fig 8.63, is not as close as seen for the previous two clays. The soil evaporation exceeds that from the free water surface suggesting an error in the estimation of the soil surface area. There are two effects which could have caused this apparent increase in evaporation:

- The Black Cotton Clay distorted as it lost moisture in much the same way as seen for the Kaolin. Thus it is likely some evaporation occurred from the underside of the sample, which has not been accounted for in the reduction of the data.

- In addition to the distorted shape, the sample suffered substantial cracking during drying. Each crack would cause an increase in exposed surface area.

It is not clear the extent to which these two factors influence the overall surface area and hence the evaporation from the soil. However it is likely to be the distortion of the sample that had the greatest influence as:

- The Kaolin and Black Cotton Clay distorted upon drying. Both lost moisture at an enhanced rate.

- The Bangali and Black Cotton Clays both suffered significant cracking, but the Bangali Clay did not display an enhanced rate of evaporation.
Despite being significant, it is possible that the confined nature of the cracks created a zone of high relative humidity preventing any increase in the overall evaporation.

It is difficult to correct the raw data for this phenomenon without making an assumption concerning the actual rate of evaporation. It is perhaps better therefore to plot the data without any further correction, make observations of any general trends and noting any possible factors which affect the absolute values.

The rate of evaporation from the soil is reasonably linear up to the point of desaturation, at a suction of 13 MPa. It took about 90 hours continuous evaporation to reach this stage. There is a gradual reduction in the rate of evaporation after desaturation, although the change in gradient is slight at the outset.

A stable air dry moisture content has not been reached after 8 days continuous drying, although the measured value of 11.2% at the conclusion of the test is close to the air dry condition. It can be seen from fig 8.64 that the change in moisture content is still asymptotic and the sample may have continued to lose moisture for a considerable period of time.

The Black Cotton Clay suffered cracking in the same way as the Bangali Clay. The cracking started after only 15 minutes exposure, fig 8.65, and was more extensive. At 30 minutes four peripheral cracks had formed to the full depth of the sample, fig 8.66. After 2 hours the cracks had extended to about 30 mm in length and were of a common shape, fig 8.67. At 6 hours the cracking appeared to have reached a maximum, with a crack width of up to 5 mm. As the rate of evaporation began to reduce the cracks began to close up once more. At the conclusion of air drying, the sample had lost some of its structural integrity, but the general trend was that the cracks had closed once more, fig 8.68

8.3.3.5 Additional drying tests

Two additional drying tests were carried out enable some preliminary conclusions to be drawn with respect to air drying and evaporation from soil samples.

- A study of possible causes of crack formation and propagation
- A study of moisture content variation through a sample during drying

Crack formation and propagation

As discussed earlier, (section 7.6), it was felt that the crack formation and propagation at the edge of the samples was influenced by two distinct factors:

- Evaporation occurs from two faces adjacent to one another at the corner of the sample.
• Hoop stresses are set up as the sample dries, these are at a maximum on the periphery of the sample.

A simple test was devised using a sample of Bangali Clay that would give some indication if the above factors were influencing cracking during drying.

A sample was trimmed from the same initial sample used for the earlier drying tests. The periphery was wrapped in cling film and fixed with adhesive tape, fig 8.69.

As the sample was placed on a balance and allowed to dry freely. A dish of water was also allowed to evaporate on a separate balance. The results are shown in figs 8.70 & 8.71.

The drying behaviour is similar to that seen for the earlier Bangali Clay sample. The initial drying rate is the same as for the free water sample until the point of desaturation is approached. After desaturation the rate of evaporation decreases as the sample moves towards a residual moisture content.

The cumulative evaporation for this test is greater than that seen for the earlier Bangali sample as, whilst the sample size is similar, the exposed surface area is reduced by wrapping. The covered area is not accounted for in the calculation but it does "feed" moisture to the exposed part of the soil.

There was no evidence of cracks on the exposed area as the sample dried out. Unwrapping the sample once air dry revealed some evidence of small cracks at the periphery, fig 8.72, but nothing of the scale that had been seen in the uncovered sample.

Moisture variation during drying

There is little experimental data on the possible variation in moisture content through a sample undergoing air drying.

There are a number of factors that might influence the moisture content through a sample including the type of soil and its stress history, the rate and extent of evaporation, sample size and shape, the extent of exposed surface and the environment under which the drying takes place. To undertake a rigorous study would require a considerable number of carefully controlled tests.

The intention of this study is to make a preliminary investigation of possible moisture content variation during drying. It was decided to use a Kaolin sample, to avoid the influence of shrinkage cracks occurring.

Three identical 50 mm cube samples were prepared. They were placed on separate electronic balances
(suspended above the surface of the balance such that evaporation could occur from all sides of the sample during drying)

At two separate stages one of the samples was removed and the moisture content variation throughout determined. The third sample was allowed to dry completely to determine the overall drying characteristics.

When a sample was removed, each of the six exposed faces was trimmed, approximately 5 mm thick, into separate moisture content tins. This gave six independent measurements of the moisture content at each distance from the original exposed surface. This process was continued until the whole sample had been trimmed. Four "layers" were cut from each sample, yielding a total of 24 moisture content measurements.

The samples were trimmed at overall moisture contents of 18.8% and 2.6%. The third sample reduced to a moisture content of 1.2% at the conclusion of drying, fig 8.73.

The results of the measurements are shown in fig 8.74. They are plotted as moisture content against distance from the original exposed surface.

Sample K1, trimmed at an average moisture content of 18.8%, shows little variation with depth through the sample. Although the sample is at a degree of saturation of 64% and a suction of about 2000 kPa the moisture content is constant. The variation within each layer (six measurements) was also very small, less than +/- 0.5%. At this point the rate of evaporation is still linear fig 8.73.

Sample K2, trimmed at an average moisture content of 2.6%, shows a distinct moisture content variation with distance from the original exposed surface. The trend is almost linear with an outer edge moisture content of 1.71% increasing to 5.68% at the centre. This leads to a variation of suction between 6 MPa at the centre increasing to 20 MPa close to the exposed surface. The degree of saturation varies between 19.5% and 8%. There was also a noticeable increase in scatter between individual measurements on each layer, often by +/-1%.

A number of general observations can be made from these two series of drying tests:

**Rate of drying**

- The rate of evaporation from all soils is closely related to that from a free water surface for a significant period of drying.
The rate of moisture loss eventually reduces with time.

The point of desaturation has no noticeable effect on evaporation.

The shrinkage limit has little effect on evaporation.

The sample suction does not control the evaporative losses. The suction at which the evaporation began to reduce varied from 3 MPa to >25 MPa.

The degree of saturation does not control evaporation. This varied between 35% - 92% when the rate reduced.

Cracking / moisture content variation

- Cracking was limited to the Bangali and Black Cotton Clays.

- Cracking commenced at the edges of the samples, and reached a maximum extent during the linear rate of evaporation.

- The crack pattern is similar on both samples.

- When the rate of evaporation reduced the cracks closed.

- Restricting the evaporation from the outer edges proved successful in reducing cracking.

- During the significant part of air drying there is no noticeable moisture content variation through the Kaolin sample.

8.3.6 Discussion

Rate of drying

There doesn't appear to be a single soil parameter which controls the rate of evaporation. Each soil type lost moisture at the same rate as that from a free water surface initially. The deviation from this rate of evaporation did not occur at a consistent point for all samples.

The rate of evaporation from the Kaolin reduces when the degree of saturation reduces to 35%, at a suction of 3MPa. The moisture content is 9% at this stage, and it may be this limited availability of moisture, or perhaps the development of a moisture content gradient that is the cause of the reduced evaporation.
The rate of evaporation for the London, Bangali and Black Cotton Clay began to reduce when the degree of saturation fell to 90%. The suction at this point was 6, 20 and >25 MPa respectively, and hence it is probably not the suction gradient that is causing the reduction in evaporation.

The void ratio at this stage is approximately 0.45 in all three soils. This gives some indication of the structure of the samples and perhaps suggests that the permeability has a significant role to play in this behaviour. The more expansive clay soils are likely to have a smaller average pore size, but these soils are also at a higher degree of saturation. The moisture content is likely to remain fairly consistent at this high degree of saturation.

**Cracking**

The formation and propagation of cracks during drying was limited to the Bangali and Black Cotton Clays. In both cases cracking commenced within 30 mins of exposure. The cracks reached a maximum extent and aperture whilst both were undergoing a linear rate of evaporation, and were of a similar pattern.

Cracking occurred when the tensile stresses in the sample exceeded the shear strength of the material. The geometry of the sample is such that this would always occur at the periphery of the sample.

Cracks will form as the sample shrinks, and will tend to be concentrated furthest from the centre of the sample. This stress concentration is enhanced by an element of differential shrinkage as the outer edges have a greater exposed surface area for a unit volume of soil.

There are a number of possible causes for cracking to occur, particularly in the expansive soils:

i) Varied pore pressure regime

With evaporation occurring from one face, moisture may move through the sample to the exposed surface. However whilst the sample remains saturated the volume reduction associated with moisture loss will ensure that the exposed surface, in effect retreats into the mass of the soil. This will prevent moisture movement.

There is however a large difference between the suction in the sample and that generated by the atmosphere surrounding the sample. It is possible that a suction gradient exists between the centre of the sample and the surface

If a suction gradient exists, there will be a higher suction at the boundary of the soil. Whilst the
soil remains fully saturated, the effect of this would be to introduce an additional compressive stress component at the boundary (as the sample has a high degree of saturation), which would reduce the likelihood of tensile failure in the form of cracking.

If however, the sample became partly saturated, then the effect of this compressive "hoop stress" would be reduced. Furthermore were micro-cracks to develop in the sample surface the shrinkage associated with drying would cause these cracks to expand. It is feasible that such a condition could exist at the surface of most soils.

The expansive soils have a much lower permeability, and lend themselves more readily to a distributed pore pressure regime. The closure of cracks once the rate of evaporation had reduced is an indication of the possibility re-distribution of soil moisture.

ii) Clay mineralogy

The dominant clay mineral in both the Bangali and Black Cotton Clays is montmorillonite. This is the most active clay mineral and also combines to produce soils of very low permeability. The shrinkage will cause the particles to move closer to one another. This may occur during air drying without sufficient time for particle re-orientation. The repulsive forces may contribute to the cracking that results. The closure of the cracks at the conclusion of the test may be indicative of particle reorientation, although it is difficult to separate this effect from a distributed pore pressure / moisture content.

iii) Increase in suction with moisture loss.

If we consider the intrinsic compression indices, fig 8.1, for the four soils we can see that for a unit decrease in void ratio, which can be compared to a unit loss of moisture, the more expansive a soil, the smaller the increase in applied stress required to cause such a change in void ratio. Under fully saturated conditions, this can be equated that for a unit loss of moisture, and hence unit decrease in volume, the more expansive soils will experience a smaller increase in suction. When we consider that the effect of suction is to act as a compressive stress, whilst the sample is saturated, offsetting the tensile stresses in the sample during shrinkage we can see that the tensile stresses will be highest in the expansive soils.

iv) Shear strength

The shear strength characteristics of the soils are not known. We do know however that the shear
strength decreases with increasing plasticity, and we also have some knowledge of $\phi'$ from the literature allowing further relative comparisons to be made of shear strength. It can be said that the more expansive soils are likely to have lower shear strengths under comparable conditions.

It is difficult to separate the possible causes of cracking from one another. It is likely to be a combination of factors that cause the tensile strength of the sample to be exceeded.

Further confirmation that it is a tensile stress that causes the cracking was seen when a similar sample was allowed to dry from a restricted surface area that did not include the edges of the sample. In this situation, with no evaporation allowed from the outer edges a compressive stress component would be generated at the periphery as the central section underwent shrinkage.

The final test undertaken investigated the possibility of a moisture content variation through the sample during drying. There was a uniform moisture content in the Kaolin sample during drying, until the overall moisture content reached a very low value. Ideally this test should be repeated on the other soil types, although the cracking of the expansive soils may prevent meaningful results from being obtained.
<table>
<thead>
<tr>
<th>soil</th>
<th>activity</th>
<th>intrinsic compression index</th>
<th>intrinsic swell index</th>
<th>predominant clay mineral present</th>
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</thead>
<tbody>
<tr>
<td>Black Cotton Clay</td>
<td>0.866</td>
<td>1.074</td>
<td>0.207</td>
<td>Montmorillonite</td>
</tr>
<tr>
<td>Bangali Clay</td>
<td>0.794</td>
<td>0.839</td>
<td>0.147</td>
<td>Montmorillonite</td>
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<td>London Clay</td>
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<td>0.522</td>
<td>0.121</td>
<td>Illite</td>
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<td>Kaolin</td>
<td>0.416</td>
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<td>0.098</td>
<td>Kaolin</td>
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*Table 8.1 some properties of soils used in this study*

<table>
<thead>
<tr>
<th>soil</th>
<th>TRL1</th>
<th>TRL6</th>
<th>TRL9</th>
<th>TRL10</th>
<th>TRL13</th>
<th>Bangali</th>
<th>BCC</th>
<th>Wa</th>
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<tr>
<td><em>W</em>&lt;sub&gt;L&lt;/sub&gt;</td>
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<td>28</td>
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<td>67.55</td>
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<td>0.054</td>
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<td>0.082</td>
<td>0.147</td>
<td>0.207</td>
<td>___</td>
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<td>e*&lt;sub&gt;100&lt;/sub&gt;</td>
<td>2.25</td>
<td>1.47</td>
<td>1.39</td>
<td>1.59</td>
<td>1.49</td>
<td>1.926</td>
<td>2.282</td>
<td>2.44</td>
</tr>
</tbody>
</table>

*Table 8.2 intrinsic properties of soils below plasticity "A" line*

NOTE:

Soils TRL 1 - TRL 13 are from Schreiner (1991)
Bangali and BCC (Black Cotton Clay) are from this study
Wa (Whangamarino Clay) is from Burland (1990)
<table>
<thead>
<tr>
<th>drying or wetting</th>
<th>moisture content range (%)</th>
<th>log₁₀ suction (kPa)</th>
<th>reference</th>
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</thead>
<tbody>
<tr>
<td>drying</td>
<td>≤47</td>
<td>4.84 - 0.0622 (m/c)</td>
<td>Chandler et al (1991)</td>
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<td></td>
<td>&gt;47</td>
<td>6.05 - 2.48 log (m/c)</td>
<td></td>
</tr>
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<td>wetting</td>
<td>≤15.47</td>
<td>4.842 - 0.0622 (m/c)</td>
<td>Chandler et al (1991)</td>
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<td></td>
<td>&gt;15.47 ≤57.2</td>
<td>4.573 - 0.0449 (m/c)</td>
<td>Data from Ridley (1995)</td>
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<td></td>
<td>&gt;57.2</td>
<td>2.904 - 0.0158 (m/c)</td>
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</tr>
</tbody>
</table>

Table 8.3 filter paper calibrations used in analysis of wetting and drying behaviour
Black Cotton Clay
Bangali Clay
Kaolin
London Clay

$C_c^*$
BCC = 1.074
BA = 0.839
LC = 0.522
KA = 0.396

vertical effective stress (kPa)

void ratio

Figure 8.1 Intrinsic compression data

Black Cotton Clay
Bangali Clay
Kaolin
London Clay

$C_s^*$
BCC = 0.207
BA = 0.147
LC = 0.121
KA = 0.096

vertical effective stress (kPa)

void ratio

Figure 8.2 Intrinsic swelling data
figure 8.3 normalised intrinsic compression curve - void index

figure 8.4 data from Burland (1990), Schreiner (1991) and this study represented on a plasticity chart
figure 8.5 void index - vertical stress relationship for intrinsic compression of soils below plasticity "A" line

figure 8.6 void index - vertical stress relationship for intrinsic swelling of soils below plasticity "A" line
Figure 8.7: Comparison between $e_L$ and $e_{100}^*$ (after Burland, 1990)

Figure 8.8: Comparison between $e_L$ and $e_{100}^*$ (after Burland, 1990)
Soils falling below plasticity "A" line added.

- Burland (1990)
- Kaolin
- London Clay
- Schreiner (1991)
- Black Cotton Clay
- Bangali Clay
- "A" line

Soils falling below plasticity "A" line added.
Chapter 8  page 271

Burland (1990) # Black Cotton Clay

relationship for soils below "A" line

\[ e^{*}_{100} = 0.409 + 0.679 e_L - 0.089 e_L^2 + 0.016 e_L^3 \]

figure 8.9 comparison between \( e_L \) and \( e^{*}_{100} \) for soils below "A" line

Ridley (1995) This study

figure 8.10 comparison of filter paper calibrations used for wetting and drying tests
Figure 8.11: Comparison between suction probe and filter paper measurements.
Figure 8.12: Summary of soil moisture characteristics for Kaolin sample KA1.
Figure 8.13: Void ratio - moisture content relationship for kaolin undergoing drying - wetting cycle.
Figure 8.14 void ratio - suction relationship for Kaolin undergoing drying - wetting cycle
Figure 8.15: Degree of saturation - moisture content relationship for Kaolin undergoing drying - wetting cycle.
Figure 8.16 Degree of saturation - suction relationship for Kaolin undergoing drying - wetting cycle.
Figure 8.17 moisture content - suction relationship for kaolin undergoing drying - wetting cycle.
Figure 8.18 Summary of soil moisture characteristics for London Clay
Figure 8.10 void ratio - moisture content relationship for London Clay undergoing a drying-wetting cycle.
Figure 8.20 Void ratio - suction relationship for London Clay undergoing a drying - wetting cycle.
Figure 8.21 degree of saturation - moisture content relationship for London Clay undergoing a drying - wetting cycle.
Figure 8.22 Degree of saturation - suction relationship for London Clay undergoing a drying - wetting cycle.
Figure 8.23 moisture content - suction relationship for London Clay undergoing a drying-wetting cycle.
Figure 8.24 summary of soil moisture characteristics for Bangali Clay sample BA1
Figure 8.25 summarizes the soil moisture characteristics for Bangali Clay sample BA2.
Figure 8.26: Void ratio - moisture content relationship for Bangail Clay undergoing drying.
Figure 8.27 void ratio - suction relationship for Bangali Clay undergoing drying.
Figure 8.28 Degree of saturation - moisture content relationship for Bangai Clay undergoing drying.
figure 8.29 degree of saturation - suction relationship for Bangari lay undergoing drying
Figure 8.30 moisture content - suction relationship for Bangui Clay undergoing drying.

- BA1
- BA2

suction capacity $C = 23.5$

Moisture content (%) vs. suction (kPa)
Figure 8.31 summary of soil moisture characteristics for Black Cotton Clay sample BCC1 undergoing drying only.
Figure 8.32 summary of soil moisture characteristics for Black Cotton Clay sample BCC2 undergoing drying only.
Figure 8.33 Void Ratio - Moisture Content Relationship for Black Cotton Clay undergoing drying.
Figure 8.34 void ratio - suction relationship for Black Cotton Clay undergoing drying.
Figure 8.35: Degree of saturation - moisture content relationship for Black Cotton Clay undergoing drying.

- BCC1
- BCC2

Degree of saturation (%)

Moisture content (%)

0 10 20 30 40 50 60 70 80

0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1

desaturation
m/c = 23%
Figure 8.36 Degree of Saturation - Suction Relationship for Black Cotton Clay undergoing drying.
Figure 8.37 moisture content - suction relationship for Black Cotton Clay undergoing drying.
Figure 8.38 idealised soil moisture characteristic for a porous rigid media.

Figure 8.39 comparison between idealised and real soil moisture characteristics.
Figure 8.40 comparison between air entry value (AEV) and actual point of air entry

Figure 8.41 comparison between shrinkage limit and limit of shrinkage
void ratio

log (suction)

figure 8.42 conceptual model of drying and wetting (after Toll, 1995)

figure 8.43 comparison between $e$ and $e_w$ for Kaolin
Figure 8.44 comparison between $e$ and $e_w$ for London Clay
Figure 8.45 Definition of constants $a$, $n$ (m/c) and $m$ (m/c) and the effect of increasing parameters $a$, $n$, $m$ in equation 8.12 (after Fredlund and Xing 1995)
Chapter 8  page 304

**figure 8.46** prediction of degree of saturation - suction relationship from limited data - Kaolin

**figure 8.47** prediction of degree of saturation - suction relationship from limited data - London Clay
Chapter 8  page 305

**Figure 8.48** Moisture content - suction relationship, London Clay - complete data set utilised

**Figure 8.49** Moisture content - suction relationship, London Clay - partly saturated data only utilised

Constants used for best fit:
- $a = 1000$
- $n = 0.6$
- $m = 2$

Constants used for best fit:
- $a = 2400$
- $n = 1$
- $m = 2$

Suction (kPa)

Gravimetric water content (%)
Figure 8.50: Specific evaporation for Kaolin undergoing air drying.
figure 8.51 moisture content - time relationship for Kaolin undergoing air drying
Figure 8.52: Specific evaporation for London Clay undergoing air drying.
Figure 8.53 moisture content - time relationship for London Clay undergoing air drying
Figure 8.54 specific evaporation for Bangali Clay undergoing air drying.
Figure 8.55 moisture content - time relationship for Bangali Clay undergoing air drying.
**Figure 8.56**: Crack formation in Bangali Clay during air drying. 30 minutes exposure.

*Figure 8.57*: Bangali Clay after 1 hour exposure.
figure 8.58 Bangali Clay after 2 hours exposure

figure 8.59 Bangali Clay after 4 hours exposure
figure 8.60 Bangali Clay after 24 hours exposure

figure 8.61 Bangali Clay after 48 hours exposure
(crack width at maximum)
Figure 8.62 Bangali Clay after 96 hours exposure
(cracks have closed up)
**Figure 8.63**: Specific evaporation for Black Cotton Clay undergoing air drying

**Legend**
- 25 MPa - sample suction
- SR 88% - degree of saturation
- mc 26.5% - sample moisture content

**Note**
- No area correction for sample shrinkage from mc 16.5%

**Graph Details**
- Cumulative specific moisture loss (g/cm²)
- Elapsed time (hours)

Key Points:
- 800 KPa: Sr 100% mc 56%
- 1400 KPa: Sr 100% mc 46%
- 8 MPa: Sr 100% mc 46%
- 20 MPa: Sr 97% mc 31%
- 26 MPa: Sr 90% mc 21%
- 33 MPa: mc 16.5%

**Notations**
- Point of desaturation: 13 MPa mc 23%
- Free water surface evaporation
- Evaporation from soil sample
- Air dry mc: 11.2%
figure 8.65 crack formation in Black Cotton Clay during air drying 15 minutes exposure

figure 8.66 Black Cotton Clay after 30 minutes exposure
figure 8.67 Black Cotton Clay after 2 hours exposure

figure 8.68 Black Cotton Clay at residual moisture content
figure 8.69 Bangali Clay, partially covered to prevent crack propagation
Figure 8.70 Specific evaporation for Bangali Clay
(constant surface area maintained during drying)
Figure 8.71 moisture content - time relationship for Bangali Clay (constant surface area maintained during drying)
figure 8.72 Bangali Clay after 300 hours exposure, with partially covered surface
Figure 8.74 Moisture content distribution through Kaolin samples during drying

- Sample K1: average mc = 18.77%
- Sample K2: average mc = 2.6%
Chapter 9

Experimental results, analysis and discussion - suction controlled oedometer tests

9.1 INTRODUCTION

The intrinsic and soil moisture characteristics have been presented in chapter 8. In this chapter we will consider the compressibility and swelling characteristics these same soils, under controlled suction conditions. The suction controlled oedometer developed during this study will be used to carry out the testing.

During the development and commissioning of the suction controlled oedometer a number of interim tests were undertaken. In addition to their development potential some of these tests produced useful data on the behaviour of the soil under test. Where relevant, these are presented and discussed.

The main experimental programme is concentrated on Kaolin. A few tests have been carried out on London clay, and the ability of the equipment to test a more expansive clay is demonstrated.

9.2 DEVELOPMENT TESTS

During the development of the experimental apparatus a number of tests were undertaken to confirm the operation of particular aspects of the equipment. Further tests were undertaken to calibrate the system prior to undertaking suction controlled tests.

The development tests can be classified into two sections:

- Confirmation of the applicability of the osmotic technique to suction controlled oedometer testing.
- Calibration of the osmotic potential of PEG.
9.2.1 Use of the osmotic potential of PEG to control matrix suction.

The background to this series of tests is discussed in section 5.3.2. They were undertaken principally to establish the applicability of the osmotic system for controlling suction, however we can gain some useful information from the test data itself.

All tests were carried out using a kaolin slurry, mixed at 1.5 W_L. The use of a soil such as Kaolin, whose behaviour is well researched would allow us to concentrate on the performance of the technique as opposed to the behaviour of the soil. Using a reconstituted material further assists in this definition.

The sample preparation and experimental set-up is discussed in section 7.6.

Suction induced compression under a range of applied suctions was undertaken, initially at PEG concentrations of 200, 250, 300 and 350 g/litre. The mass of moisture passing from sample to stock solution, and the compression of the top cap were monitored until equilibrium conditions were achieved.

On average each test took six days to reach equilibrium.

Fig 9.1 shows a summary of the results of all four tests. The axes of "compression of the top cap" and "moisture loss from the sample" have been scaled such that they are directly comparable, i.e. 6 mm compression is equivalent to a sample volume change of 48.6 cm³.

These tests were performed primarily to assess the function of the semi-permeable membrane, although a number of useful observations can also be made.

- The same general pattern of behaviour is observed in all tests.

- Initially the compression is greater than that predicted by the moisture loss. This can be explained by visualising the top cap sinking slightly in the slurry when first placed. Also separation of the slurry due to sedimentation may occur, this would leave some standing water on the top surface of the clay.

- When the excess water has been drawn off the compression, due to the applied suction at the base, is one dimensional.

- After a sufficient gain in strength has occurred in the Kaolin, lateral shrinkage occurs away from the wall of the oedometer. This is indicated by the separation of the compression and moisture loss data.
Increasing the applied suction at the base of the sample increases the rate of compression, confirming that the function of the membrane can be compared to that of a porous material, in that a change of pore pressure on one side of the membrane is seen on the other side.

Let us consider the test carried out with a PEG concentration of 300 g/litre in a little more detail, fig 9.2.

- The effects of placing the top cap on the slurry, and subsequent sedimentation following placing, was to cause the top cap to sink by a maximum of approximately 2 mm.

- At a moisture content of 83.3% the excess moisture had been drawn from the sample, which now has sufficient strength to support the top cap. The suction at this point can be inferred from the virgin compression curve for the reconstituted material, fig 8.1. It is not exact but will give us an estimate for these purposes. We see that the suction is likely to be less than 1 kPa at this stage.

- Between moisture contents of 88.3% and 66.8% the suction induced compression is one-dimensional.

- At a moisture content of 66.8% the moisture loss and compression become uncoupled. This is indicative that the sample has sufficient strength to support the self weight of the soil and top cap loading without need for the lateral restraint provide by the oedometer ring. Lateral shrinkage begins to occur from this point. The sample suction can be estimated to be approximately 10 kPa at this stage.

- The rate of vertical compression reduces noticeably at a moisture content of 46.76%. The suction is estimated to be approximately 150 kPa. Suction induced compression is at 97% of its final value.

- The moisture loss from the sample, although reducing does not experience the same degree of change in gradient as the vertical compression. The moisture loss is at 90% of its final value.

- Steady state conditions in respect of both compression and moisture loss occur at about the same time period.

To conclude it can be seen from these tests that the osmotic technique can be applied to control matrix suctions in soils, but as discussed in chapter 5 there was much work to be undertaken in refining the technique.
Permeability of the semi-permeable membrane

One further aspect of the this series of tests was that the permeability of the membrane could be measured by analysing the early part of each test.

During the initial phase of each test, free water was drawn through the membrane under a known suction. This could be compared to a constant head permeability test, with measurement of the rate of flow across a known area under a defined head loss (applied suction in this instance). The drainage path length is the thickness of the membrane. Thus the permeability could be determined under a number of constant head conditions, equation 9.1:

$$k = \frac{q}{Ah} \quad \text{9.1}$$

where

- $k$ = permeability (m/s)
- $q$ = flow (m$^3$/s)
- $l$ = drainage path length (m)
- $A$ = cross sectional area of flow (m$^2$)
- $h$ = head loss (m, of water)

Fig 9.3 shows the first 20 hours of each of the four tests. During this period the flow rate was linear and could be taken as constant for the purpose of determination of permeability. Table 9.1 gives the details of each test. There is a gradual reduction in permeability with increasing applied suction, from $2.903 \times 10^{-13}$ m/s at 200 kPa reducing to $1.545 \times 10^{-13}$ m/s at an applied suction of 350 kPa. The cause of the reduction in permeability is unclear, it may be influenced by compression of the membrane at higher suctions - the membrane when wet is quite pliable - alternatively, the PEG molecules could be clogging the membrane structure. This effect is not critical to the testing programme, but should be borne in mind for future experimental procedures.

The membrane permeability is very low and therefore will have a significant effect on the observed soil behaviour. With the possible exception of the clay soils containing montmorillonite, the permeability of the membrane is likely to be the controlling factor in the time for equilibrium of all tests.

9.2.2 Calibration of the osmotic potential of PEG.

A series of tests were carried out to determine the relationship between the concentration of PEG and the suction generated across a semi-permeable membrane.
All tests were carried out on kaolin samples which had been preconsolidated one-dimensionally to 200 kPa from a slurry in a lever arm oedometer.

Samples were trimmed and set up in the osmotic oedometer, as outlined in section 5.4.3.2.

Six tests were performed, using PEG concentrations of 150, 200, 250, 290, 330 and 350 g/litre.

Firstly a multi stage test was carried out, to determine the suctions generated by PEG solutions of 200, 250, 290 and 330 g/litre. During each stage, vertical compression, moisture loss from the sample and matrix suction at the top of the sample were monitored. Two separate single stage tests were also undertaken at a PEG concentrations of 150 and 350 g/litre.

During the multi-stage test, all parameters were monitored until a stable equilibrium condition was reached. The PEG solution was then changed for one of increased concentration. The same sample remained in the cell throughout the tests, only the solution was changed. Each stage commonly took five days to complete.

We will look a little more closely at some of the individual results, at PEG concentrations of 200, 290 and 330 g/litre. Fig 9.4 shows the results from this series of tests. Consider the data from the test where a PEG solution of 200 g/l was used. The compression and moisture loss axes are scaled such that the data can be directly compared.

We can make the following comments on this result:

- The moisture loss and suction curves are of a common shape indicating the direct relationship between moisture content and suction. Also they show no discernable lag between response of the two parameters.

- Initially the sample compresses one-dimensionally, this can be seen from the coincidence of the moisture loss and compression plots at the beginning of the test.

- At about 150 kPa the sample also begins to shrink laterally, indicated by the uncoupling of the moisture loss and compression paths. This uncoupling occurs at about the same suction that a change in the rate of uncoupling was observed in these two parameters in tests on kaolin slurry, fig 9.2.

There were a number of instances when tension breakdown occurred in the IC tensiometer. When this happened, the test was allowed to continue until it reached equilibrium, when the equilibrium suction was measured.
Fig 9.5a shows the measurement of the suction in the sample under the influence of a PEG solution of 290 g/litre. This measurement was made when the vertical compression and moisture exchange had both reached stable values. We can see that the tensiometer response is quick and the final measured suction of 735 kPa was monitored for approximately 3 hours to ensure that the sample was indeed in equilibrium.

Fig 9.5b shows the results from an additional single test using a PEG solution of 350 g/litre. This test demonstrates the need for good contact between tensiometer and sample. We can see that as the sample dried, contact was lost after about 30 hours. This is indicated by a sudden increase in apparent suction, leading to tension breakdown. The tensiometer was replaced using a soft piece of the parent material to ensure good contact was maintained for the remainder of the test. Following this test, soft parent material was used for all measurements and this resulted in a much improved ability to make long term continuous measurements over a large range of suctions.

9.3 INTRODUCTION TO STRESS PATHS AND TEST DETAILS

A series of tests was undertaken to investigate the effects of suction on compressibility and swelling characteristics.

All samples were prepared initially from a reconstituted state (1.5 W). One-dimensional consolidation of the slurry was undertaken to a maximum vertical stress of 200 kPa.

From this point the samples are sufficiently stiff to allow them to be handled. Ideally one would want to unload and remove the sample without allowing any uptake of moisture. In practice this is not possible, and some swelling of the sample will occur as it draws moisture from the porous stones. This initial sample preparation is shown diagrammatically in figure 9.6, as a three dimensional plot of void ratio, vertical applied stress and suction.

All tests shared this common initial preparation. The required suction was achieved by allowing the sample to air dry, before trimming into the cutting ring of the suction controlled oedometer. Holding the suction constant using the PEG system, a load-unload cycle was undertaken. This is shown in figure 9.7. a number of samples at different (constant) suctions were subjected to the same load-unload cycle.

9.3.1 Stress states, stress paths and presentation

A number of soil parameters were measured during the suction controlled oedometer tests. The principle measurements were of applied vertical load, vertical compression, matrix suction and moisture loss from the sample. This allows us to derive the following parameters directly:
i) Applied vertical total stress, $\sigma_v$

ii) Matrix suction, $S$

iii) Void ratio, $e$

iv) Moisture content, m/c

It is important to note that the measured parameters show that the internal and external stresses are not compatible.

The internal stress (matrix suction) is an isotropic stress state, acting equally in all directions whereas the external stress is anisotropic and comprises two different stress components, $\sigma_v$ and $\sigma_h$, of which only $\sigma_v$ is measured. The relationship between the vertical and horizontal stresses, under confined lateral straining, can generally be represented by eqn 9.2

$$K_0 = \frac{\sigma_h'}{\sigma_v'}$$

where:

$\sigma_h'$ = horizontal effective stress

$\sigma_v'$ = vertical effective stress

There are several empirical and theoretical relationships describing the $K_0$ relationship under both normally and overconsolidated loading and unloading.

Mayne and Kulhawy (1982) undertook an extensive study of these relationships, and presented a database of over 170 soils where the $K_0$ - OCR relationship had been investigated. They concluded that the following expressions, eqn 9.3 and 9.4 best described the behaviour of soils under normally consolidated and overconsolidated stress paths respectively:

$$K_{oc} = 1 - \sin\phi'$$

$$K_{oc} = (1 - \sin\phi')\text{OCR}^\sin\phi$$

where:

$K_{oc}$ = $K_0$ normally consolidated

$K_{oc}$ = $K_0$ overconsolidated

$\phi'$ = effective angle of shearing resistance

It should be noted that due account must be made of the suction when calculating OCR, eqn 9.5
9.3.2 The application of $K_0$ to suction controlled testing

It is important to note that $K_0$, (eqn 9.2), relates the effective vertical and horizontal stresses. During suction controlled testing, measurement is made of the total vertical stress and the pore water pressure. For analysis purposes the total horizontal stress is required, allowing interpretation of the test data using mean total stress and suction as separate stress variables. We can explore the effect of the presence of a negative pore pressure on the estimation of the horizontal total stress from this relationship.

\[
K_0 = \frac{\sigma_h}{\sigma_v} = \frac{\sigma_v - u_w}{\sigma_v - u_w}
\]

\[
\sigma_h - u_w = K_0 (\sigma_v - u_w)
\]

\[
\sigma_h = K_0 (\sigma_v - u_w) + u_w
\]

\[
\sigma_h = K_0 \sigma_v + u_w (1 - K_0)
\]

Thus, provided $K_0$ is known, it is possible to use eqn 9.6 to determine the total horizontal stress in a saturated sample with a negative pore pressure.

In practice there is one limitation on the application of this derivation. When the vertical stress is low and the sample suction high, the "$K_0 \sigma_v$" term is small and the "$u_w (1 - K_0)$" is both large and negative - suctions being negative pore pressures. This results in a calculated negative horizontal stress, which is clearly inadmissible in this particular situation.

Consider the stress history of a sample consolidated from a slurry. We will present the stress path followed on a plot of $\sigma_v$, $\sigma_h$, and $S$. Initially it is consolidated one-dimensionally to a vertical stress of 200 kPa, this plots on the $\sigma_v$, $\sigma_h$ plane, fig 9.8. The path follows the $K_0$ line for the particular material.

The path followed during unloading is influenced by both the non-linear variation in $K_0$ during unloading, and the intake of moisture and subsequent swelling of the sample. In this phase, the sample will have no external applied stresses, but will have a suction (which acts isotropically). This is depicted in fig 9.9.

Following removal from the oedometer the sample is carefully allowed to dry to the required suction, fig
9.10. After trimming into the suction controlled oedometer the sample is loaded such that its stress path lies on a plane of constant suction, 9.11.

To summarise we can say that:

- The measured stresses are total vertical stress and negative pore water pressures. These are one-dimensional and isotropic stresses respectively.
- The application of $K_0$ theory is related to the effective stresses. It is not directly applicable to total stresses.
- The extension of $K_0$ theory to account for negative pore pressures, allowing estimation of the total stress state may be limited at low vertical stresses.
- The sample stress history is complex. It has undergone loading, partially drained unloading, and isotropic drying prior to testing.

There are three important points to note however, which will allow us to estimate the horizontal total stress of the sample.

i) Whilst the definition of $K_0$ requires and presents effective stress states, for a saturated sample where the pore pressure is constant, a change in total stress is equivalent to a change in effective stress.

ii) Providing the sample suction exceeds the previous stress history, the sample can be considered to behave as a normally consolidated material.

iii) Upper and lower bound stress relationships can now be hypothesised, fig 9.12. Following air drying the sample will lie at the origin of the total stress axis, with just the suction acting. Loading from this point will be one-dimensional.

The actual path followed is unclear, however we can define two limiting conditions, $K_{01}$ and $K_{02}$, fig 9.12. The path may follow $K_{01}$, as it is a normally consolidated material, undergoing one-dimensional compression. However this does not account for the increased stiffness of the sample due to the presence of the suction. An upper bound could be defined from the origin of an "effective stress" axis, following a $K_0$ normally-consolidated path, $K_{02}$, this would lie above but parallel to $K_{01}$. The actual path followed is not clear, however the most probable is that, from the total stress origin the $K_0$ path would move up to meet $K_{02}$ once the effects of suction had been overcome. This path is described by $K_{02}$ on fig 9.12. From here normally-consolidated behaviour is likely.

The path from the origin to $K_{02}$ would need to determined experimentally. For the purposes of estimating the horizontal stress, $\sigma_h$, we can take a Poisson's ratio of 0.2, (from elasticity theory), allowing the
relationship between vertical and horizontal stress to be defined, eqn 9.7.

\[
\frac{\Delta \sigma_h}{\Delta \sigma_v} = \frac{\mu}{1 - \mu} \tag{9.7}
\]

This relationship can then be applied until the path reaches \( K_{o2} \) (fig 9.12) and will define a bi-linear trend for \( K_o \) which is sufficient for these purposes.

### 9.3.3 Stress parameters used for presentation

Initially presentation of the analysis will be undertaken using the measured stress parameters, i.e vertical total stress, suction and void ratio. This will allow us to examine the effects of suction on one-dimensional compression and swelling.

When combining the stresses to consider the total stress states we have two options:

i) Combine the measured stresses (i.e \( \sigma_v - u_r \)). This allows us to use known stresses but as it involves a combination of stress types (one-dimensional and isotropic) it may lead us to an invalid stress state.

ii) Use the isotropic component of the total stress, where mean total stress = \( (\sigma_v + 2 \sigma_h) / 3 \). This involves making some assumptions with respect to the value of the horizontal stress, however this has a good theoretical backing and allows us to present the data in a more meaningful stress state.

This will allow us to examine soil behaviour based on both measured and inferred stress states.

### 9.4 TESTS UNDERTAKEN ON KAOLIN

#### 9.4.1 Test details

Six suction controlled tests are presented on Kaolin. They all involved one-dimensional loading and unloading under constant values of suction. In addition some samples were allowed to swell by reducing the suction at a given point in the test.

All samples were prepared as outlined in section 7.6. Following one-dimensional compression from a slurry to a vertical stress of 200 kPa, they were unloaded in a single stage and removed as quickly as possible to prevent intake of moisture, reduction in suction and associated swelling. It is not possible however to prevent some intake of moisture.
If the sample was unloaded undrained one would expect the suction in the sample to be equal to the isotropic stress on the sample prior to unloading. At 200 kPa and assuming a $K_0$ of 0.61 for Kaolin (from Mayne and Kulhawy, 1982), the mean total stress would be 148 kPa, hence the suction in the sample would also be 148 kPa following undrained unloading. Typical measurements of the suction following removal of the sample were of the order of 50 - 100 kPa. Details of the tests undertaken are given in table 9.2, and a schematic representations are given in fig 9.13.

9.4.2 Comparison between suction controlled and lever arm oedometer tests

Test KA1 allows an important comparison to be made between a conventional lever arm oedometer and one in the suction controlled oedometer. One would expect that initially the sample would behave as an overconsolidated soil - considering the preparation stress history - but tend towards normally consolidated behaviour at higher stresses. The most direct comparison here would be with the intrinsic compression data from the reconstituted tests.

Firstly let us consider the intrinsic compression data once more. A number of researchers have determined experimentally the intrinsic compression and swelling characteristics of Kaolin.

Fig 9.14 combines ICL data from this study with that of Ridley (1993), Martins (1983) and Mahmoud (1983). The data from Mahmoud is a summary of a number of authors data on Speswhite Kaolin.

There are considerable differences in the absolute void ratio values between authors. At a vertical stress of both 100 kPa and 1000 kPa the range of void ratio values is 0.15

The gradient of the compression curves are however quite close to one another and the range of results reflect the absolute values and not the gradient of the compression curves. An average gradient from the data is taken.

The same approach is applied to the intrinsic swelling index, although there is less data available. These results are produced in fig 9.15.

The results of both intrinsic compression and swelling are summarised in table 9.3.

The variability in the results reflects the difficulty in determining absolute values of void ratio, although a greater degree of confidence can be achieved with respect to relative values. Thus the gradients from the respective studies agree quite well.

Considering the compression data from test KA1, in figure 9.16 and superimposing the average ICL from
fig 9.14 we can see that the sample behaves much as we would expect. Although the compression path does not reach the ICL it does move parallel to it at higher applied stresses.

A similar exercise can be carried out for the data from test KA1 during unloading. This can be compared to the intrinsic swelling line (ISL) for the reconstituted material.

The ISL is determined during unloading from a maximum applied vertical stress of 1000 kPa. Test KA1 underwent loading to 2000 kPa, hence the absolute void ratios between the two tests will not be directly comparable. We can however compare gradients by translating the ISL such that it has the same void ratio as KA1 at 1000 kPa. There is a very good agreement between the unloading data from the ISL and that from tests KA1.

It can be concluded that the suction controlled oedometer gives results comparable to those obtained in more conventional apparatus, under similar conditions. This is important should reference be required to additional conventional test data.

9.4.3 Effect of suction and applied stress on compression

Fig 9.17 - 9.20 shows the results of one-dimensional compression on kaolin samples at suctions of 15, 200, 300, 750 kPa (KA1 - KA4). The yield stress of each sample has been estimated using a Casagrande construction.

The result of applying an initial suction to the sample is to increase the yield stress. This is as expected, with the suction acting as an isotropic stress. A yield locus can be generated from the yield points of each compression plot, figure 9.21, and the yield stress values are given in table 9.4.

Here we see more clearly that with increasing suction the yield stress increases. It is not clear what path the yield curve would take if the suction were increased further as at this point the samples are still saturated. The shape of the curve is comparable to that proposed for the loading collapse (LC) yield curve for partly saturated soil models, (Alonso, Gens and Hight, 1987). If however the yield curve is approaching a maximum value then the LC curve associated with many partly saturated soil models will not be seen. Alternatively, there may be a discontinuity as the sample desaturates.

Fig 9.22 shows a summary of the compression paths for tests KA1 - KA4. All tests tend towards a compression gradient similar to that of the intrinsic compression line. At higher suctions the stress required to achieve this linear gradient increases. The test at a suction of 750 kPa is still behaving as an over-consolidated material, at the maximum applied stress of 2 MPa.

Fig 9.23 shows the effect suction has on one-dimensional compressibility over a range of applied vertical
stresses. The measured void ratio is plotted at a given vertical stress from each test under a range of
suctions. The data is taken from the compression stage of tests KA1 - KA4. A best fit line is drawn
through the individual data points. We can see that as the vertical stress is increased the effect of
increasing suction is reduced, fig 9.24. The shrinkage associated with drying is suppressed with increasing
applied vertical stress.

It should be noted that the relationship shown is inferred from a number of discrete points obtained from
a series of tests and does not take into account any path dependency. To confirm this relationship it would
be necessary to carry out a parametric study whereby a sample was loaded to a given vertical stress and
then subjected to an increase in suction, thus following the inferred drying path. This is not possible in
an oedometer as the lateral shrinkage associated with drying can not be measured in an oedometer.

We can however compare the inferred relationship at a nominal vertical stress (50 kPa) with the drying
data over the same suction range from the filter paper drying tests, fig 9.25. The agreement is good, with
both showing the same trend. The scatter in the drying test data can be attributed to the method of
determining the void ratio, i.e physical measurement of the sample at a number of points and subsequent
estimation of the volume.

We can therefore assume that the inferred relationship has some merit, although it requires further
confirmation, particularly under higher vertical stresses.

One further point we can note from this series of figures is that the void ratios at high vertical stresses
- above 500 kPa - obtained from test KA2 (at a suction of 200 kPa) are consistently above the overall
trend, and those from test KA3 (at a suction of 300 kPa) are consistently below the trend. This would
suggest that the absolute values could be normalised to correct for this deviation. This would harmonise
the data in figure 9.22 by moving the data from test KA2 down and test KA3 upwards on the figure.
Considering both the small magnitude of the correction required, and that it is based on an inferred
relationship it was decided not to perform this correction for analysis of the data, but to note its possible
influence.

As the suction is increased, the compression characteristics change considerably, and we can draw a
number of summary conclusions from these figures:

- At vertical stresses below the applied suction, the sample behaves as an overconsolidated soil.
- The behaviour is over-consolidated up to the yield point, and thereafter normally. This is classical
  overconsolidated material exceeding its pre-consolidation stress, and behaving as a normally
  consolidated soil.
• The ultimate compression gradient tends towards, and is comparable to, the intrinsic compression line in all cases.

• The yield stress increases with increasing suction.

• The yield stress appears to be tending to a maximum as the sample moves towards the point of desaturation.

• The yield point is more distinct at higher suction values. At nominal suction yield is not clear. As the suction is increased, however, the yield point becomes more easily determined.

9.4.3.1 Void ratio - suction - vertical stress relationship

We have looked individually at the influence of suction on compressibility, and of vertical stress on shrinkage. Now we will present a summary plot of the three measured components, void ratio, suction and vertical stress.

Fig 9.26 shows a loading drying surface developed from tests KA1 - KA4. The individual stress paths are highlighted and the yield curve is also plotted. The same data is presented on a linear plot in fig 9.27.

The apparent effect of increasing the applied vertical stress is to suppress the effects of shrinkage resulting from increasing the suction. At low vertical stresses the shrinkage is quite significant, whereas beyond a vertical stress of about 1000 kPa the shrinkage is minimal.

9.4.4 Representation using mean total stress

As discussed earlier two approaches can be taken to the analysis of the data from the suction controlled tests. The first approach, discussed in section 9.4.3, makes use of the actual measured values of vertical stress, void ratio and suction. The second approach, described in the following section, uses the mean total stress (the external isotropic stress) calculated from the applied vertical stress using the $K_e$ relationships proposed by Mayne and Kulhawy (1982). This will allow us to make some tentative comparison between effects of an isotropic stress applied externally and one generated by a suction.

It can be seen from fig 9.22 that the normally consolidated portions of the compression curves for the higher suction values lie below those for the lower suction. Coincidence of these curves would not be anticipated since each is at a different stress ratio $q/p'$. The tests carried out at the higher suction have lower values of $q/p'$ due to the higher component of suction (isotropic stress). As mentioned previously,
the determination of the resultant horizontal values of q and p' require the measurement of total horizontal stress σ, which was not possible in this programme. However, by estimating expected values of K₀ during the tests, it is possible to plot the compression curves in terms of estimated p' against e. Fig 9.12 shows the idealised path in (σ, u) versus (σ, u) space for a fully saturated sample which is first dried to a given suction and then subjected to one-dimensional compression at constant suction.

Figs 9.28 – 9.31 show the relationships between void ratio and p' determined from the idealised stress path in fig 9.12. In fig 9.32 these idealised compression curves are superimposed. It can be seen that although plotted in terms of p' the compression curves for the tests at higher suctions still lie beneath those at lower suctions. It might be expected that they would be coincident or perhaps lie outside the lower suction tests as at a given void ratio, p' should be greater. However the mean total stress path followed is not one of constant ratio of q/p', but of increasing q/p' as can be inferred from fig 9.12.

The yield stress points from this series of tests are plotted on fig 9.33. It can be seen that the curves are of a similar shape, with that plotted in terms of total mean stress lying inside that plotted in terms of vertical stress.

A loading - drying surface has been developed in terms of mean total stress, suction and void ratio from the compression characteristics of tests KA1 - KA4. The relationship is plotted on a logarithmic mean total stress axis fig 9.34, and with the mean total stress on a linear axis in fig 9.35.

We see that all tests tend towards a linear compressive gradient. At increasing suction a greater applied stress is required to exceed the yield point and allow the sample to move towards a gradient similar to that of the intrinsic compression line defined earlier.

**Comparison between compression induced by suction and mean total stress**

Having calculated the mean total stress applied to the sample, we can make a tentative comparison between compression induced by an applied external isotropic stress (p), and that induced by suction (S).

Using the relationship derived to allow the surface in fig 9.34 to be plotted we can calculate the compression due to increasing suction over a range of constant, applied mean total stresses. Equally we can calculate the compression due to increasing mean total stress for a range of constant suctions. Figures 9.36 - 9.39 compare compression at a range of stresses between 75 kPa and 630 kPa.

The agreement is good across the range of stresses. There are some differences in void ratio, particularly at low constant stresses, (75 kPa), although the trend is comparable. Also at high suctions there is a tendency for over consolidated behaviour, although there may be some influence of the expanding yield curve with increasing suction. These comparisons are based on the calculation of mean total stress from
an inferred $K_c$ relationship and should be explored experimentally before detailed discussion can be made.

9.4.5 Effect of suction and applied stress on swelling

We can compare the unloading data from tests KA1 - KA4 with the intrinsic swelling line (ISL) to investigate the effect of suction on unloading.

Fig 9.40 compares the unloading data from test KA1 (nominal suction) with the ISL (normalised at a vertical stress of 1000 kPa). The agreement is very good, and confirms that the suction controlled oedometer is comparable to more conventional oedometer tests, performed under similar conditions.

Figs 9.41 - 9.43 compares the ISL (normalised at the maximum vertical stress) with the unload data at constant suctions of 200, 300 and 750 kPa (KA2, KA4). We can see that the effect of the suction in the sample is to suppress the swelling path. The unloading paths for the suction controlled tests all tend to asymptotic values of void ratio. If we plot these final void ratio values on an ISL, which has been translated at the maximum vertical stress for each test, the equivalent vertical stress can be defined. It is of the same order as the suction in the sample, although slightly higher in all cases. This probably reflects the difference between one-dimensional and isotropic stresses.

We can also make a comparison between unloading in terms of applied stress and a reduction in suction. Fig 9.44 shows data from the swelling (reduction in suction) stages of tests KA2 ($s = 200$ kPa), KA4 ($s = 750$ kPa), together with the filter paper wetting data, the equivalent unloading from tests KAl - KA4. These data represent confined unloading, confined and unconfined swelling, and intrinsic unloading respectively.

The comparison is very good between all data. The gradients are very similar, and the absolute values are also comparable. We can summarise from this that at low vertical stresses, mechanical unloading and swelling through a reduction in suction are comparable. Despite being subjected to a suction of 35 MPa, the filter paper wetting data fits well although this is influenced by the fact that the sample stops shrinking during drying at about 2 MPa - equivalent to the maximum applied vertical stress in the suction controlled tests.

An unloading surface is shown in fig 9.45 (linear mean total stress axis), and 9.46 (logarithmic mean total stress axis). The data from fig 9.44 suggest that the swelling (reduction in suction) stress paths would also fall on this surface, in the same way as the drying stress paths fall on the loading surface in figs 9.26 & 9.27. The loading / drying surface is a monotonic surface whereas the unloading surface is not. If we consider the data at a vertical stress of 1.5 MPa, we can add the data from the swelling under constant load from test KA6. This will allow us to investigate the path dependency with respect to swelling.
Fig 9.47 shows the data from the unloading surface at a mean total stress of 1500 kPa. The swelling data from this stage of test KA6 is also plotted. The swelling path is at a much flatter gradient, and cannot readily be compared to the unloading surface. The discrepancy at the initial suction (750 kPa) is because the unloading surface has been defined from 2 MPa, whereas the maximum load for test KA6 was only 1.5 MPa, thus the swelling path lies above the unloading surface at this point.

We can also make a comparison between swelling and unloading at low confining stresses. We can compare the interpolated unloading surface at a vertical stress of 50 kPa, with the swelling stages from tests KA2 (s = 200 kPa), KA4 (s = 750 kPa) and KA5 (s = 750 kPa). These are shown in fig 9.48. We see a remarkably good fit, taking due account of the initial high void ratio in test KA5 where the maximum vertical stress is 50 kPA (sample preparation excepted).

9.4.5.1 Discussion

We can see that at low vertical stresses, the swelling paths are similar to those predicted from the unloading surface. The actual swelling path lies slightly below the unloading surface by an increased amount with swelling from higher suctions, although the general predictive abilities are quite good from the unloading surface.

At higher stresses the unloading surface cannot be used to predict the swelling path. The actual swelling path is at a much flatter gradient than that predicted from the unloading path.

Let us consider the differences between a reduction in loading and a reduction in suction on the sample stress state.

A reduction in vertical stress causes a reduction in horizontal stress. The reduction in horizontal stress is related to the K₀ conditions of the soil, which are influenced by stress level, path and history. The reduction in horizontal stress is lower than the vertical stress.

A reduction in suction causes an increase in isotropic internal stress. The external vertical stress will not change, however the horizontal stress will increase. It is difficult to predict the actual stress state under these conditions, but we can assume that for an equal reduction in vertical stress or suction the overall change in stress states will generally not be equivalent.

It is likely that the soil stiffness, stress history, and initial stress state will all have an influence on the resulting changes in stress state and hence behaviour of the soil. The complete stress state would ideally need to be measured.
9.5 TESTS ON LONDON, BANGALI AND BLACK COTTON CLAYS

Initially it was envisaged that the series of tests undertaken on the Kaolin would be repeated for the other, more plastic soils. The extensive development, coupled with comprehensive characterisation of the soils reduced considerably the time available for suction controlled testing. In addition the duration of the tests increased dramatically with the more expansive soils.

It was decided that some tests should be performed on all soils to prove the capabilities of the apparatus for testing a wide range of soil types.

The limited test data is not sufficient to allow loading and unloading surfaces to be developed. As such all presentations are of the actual raw test data, without any curve fitting exercise undertaken.

9.5.1 Tests undertaken

The background to the sample preparation, test stress paths and identification is the same as outlined earlier for Kaolin (section 9.4.1). The prefix to the test in this section reflects the soil type, therefore tests KA1 will be given the notation LCI (London Clay), BA1 (Bangali Clay) or BCC1 (Black Cotton Clay) respectively. The tests undertaken and the test duration are given in table 9.5.

9.5.2 London Clay tests

Three tests were performed on London Clay, LCI, LC4 and LC5. Details of the stress paths are given in table 9.2 (KA1, KA4 & KA5) and the test durations are given in table 9.5. The London Clay has a significantly lower permeability than the Kaolin. This increased the test duration considerably.

Test LCI allows direct comparison with the intrinsic compression and swelling indices determined in a conventional oedometer. As can be seen from fig 9.49, the agreement is very good. The absolute void ratio values are offset slightly (≈ 0.05 e), but the general trend, and compression indices are comparable. The ISL requires normalising (at a vertical stress of 1000 kPa) for comparison. The swelling indices are identical in both oedometer tests.

The yield stress, of 180 kPa, was determined using a Casagrande construction and is comparable to the pre-consolidation pressure (200 kPa).

Fig 9.50 details the load - unload data from tests LCI and LC4. As seen earlier with the Kaolin tests there is a considerable reduction in void ratio during the drying for sample preparation. The initial void ratio reduces by ≈0.3. We can also see the influence the sample suction has on the unloading path, suppressing
swelling at low values of vertical stress.

Fig 9.51 compares the ISL (normalised at 1000 kPa) with filter paper wetting test data from the soil moisture characteristic tests and the swelling stages of tests LC4 and LC5. The swelling gradients are all closely matched, with those of tests LC4 and LC5 being just slightly shallower than the ISL and wetting test data. Test LC4 plots above the wetting data, as this sample was only loaded to 1 MPa. Equally test LC5 plots higher still as its maximum applied vertical stress during testing is only 50 kPa.

This data adds to the concept that at low confining stresses unloading and swelling data are broadly comparable.

9.5.3 Bangali Clay tests

Two tests were performed on Bangali Clay, BA1 and BA4. Details of the stress paths are given in table 9.3 (KA1 & KA4) and the test durations are given in table 9.5.

The Bangali Clay is both less permeable and far more compressible than the London Clay. This increased the test duration particularly for test BA4, which involved swelling from a high suction following compression. The overall test duration for the Bangali Clay, is the highest for all the materials. The two tests undertaken provide useful but limited data.

Test BA1, fig 9.52 compliments the intrinsic compression indices well. The offset between the two tests is <0.05 e, but the gradient and general trend is very good. There is some divergence between the unloading data, particularly at very low stresses. This trend was also observed during the determination of the ISL, fig 8.2, and the actual data points are added to fig 9.52. The yield stress, at 210 kPa, is also comparable to the preconsolidation stress of the sample (200 kPa).

The shrinkage during the drying phase of sample preparation for test BA4 is considerable, 0.55 e. This is shown in fig 9.53 where the loading and unloading stages of both tests are shown. The compression curve for BA4 crosses that of BA1 and appears to be tending towards a similar compression index, although at 2 MPa it is still on a gradient of a much stiffer soil.

The unloading stage of BA4 shows the same form of suppressed swelling as is seen in both the London Clay and Kaolin previously. A general trend towards a constant void ratio value is apparent at low applied stresses.

The final swelling stage of BA4 (s = 750 kPa) is compared to the ISL in fig 9.54. The swelling is broadly comparable although a tendency towards a constant void ratio is more apparent in the data from BA4.
9.5.4 Black Cotton Clay tests

The Black Cotton Clay is a very expansive soil of low permeability. It is the least permeable and most compressive of the soils tested. This results in tests durations that are very lengthy. Only one test was undertaken on this material. Careful consideration would need to be made as to the tests to be undertaken, as some of those outlined in the original testing programme could take several months to complete.

Only test BCC1 of the original tests programme was undertaken. This is useful as a commissioning test for the newly developed equipment, allowing comparison with conventional oedometer data.

Fig 9.55 shows the load / unload data from BCC1 together with the ICL / ISL data from the reconstituted tests carried out in a lever arm oedometer. The suction controlled test data matches the ICL very well. There is some scatter in the data and this is a reflection of the low permeability of the material slowing the response of the material, and allowing some hunting of the software controlled apparatus. This could be easily reduced by incorporating a wait stage into the control routine. The yield stress of the sample is 180 kPa and is comparable to the preconsolidation pressure.

The unloading data does not fit the ISL as well as for the other materials. If we add the actual ISL data, the comparison improves.

The unloading stage was concluded at 500 kPa as the time for unloading was excessive. This was due to a combination of both a low permeability, and the high swelling characteristics of the soil. The swelling characteristics were particularly influential in extending the unloading stage. Whilst the situation would be improved for some of the other tests as there would be a greater suction gradient operating across the membrane, all test durations would be considerable.

9.6 SUMMARY

In this chapter three main parts of the research project have been presented and discussed:

i) Experimental aspects of the development of the suction controlled oedometer.

ii) Stress paths and the presentation of test data.

iii) Suction controlled oedometer testing.

Development

A great number of tests were performed during the development of the techniques and apparatus of which
a few are presented.

One of the key aspects of the development has been the definition of the relationship between the concentration of a PEG solution and the subsequent matrix suction generated across a semi-permeable membrane. The test data from this calibration exercise have been presented and give a useful insight into the behaviour of Kaolin undergoing drying. The re-analysis of the tests on slurry have allowed the permeability of the membrane to be defined.

**Stress paths**

Discussion on stress states, stress paths and presentation is given. The application of saturated soil mechanics principles appears well founded in these situations, however there is a need for care when extrapolating the test data beyond the actual measured values.

**Suction controlled tests**

The capability of the newly designed apparatus for carrying out oedometer tests where the sample suction is controlled and measured continuously and independently is clearly demonstrated.

The tests undertaken at nominal suction are directly comparable to those performed in a conventional oedometer. From the tests undertaken on Kaolin we can make a number of comments:

- Increasing the sample suction increases the value of the yield stress. The post yield behaviour tends towards that of the intrinsic compression line in all tests.

- A void ratio - suction - vertical stress relationship can be developed for conditions of increasing stress. This allows the effects of drying under load to be inferred. The data from the filter paper drying tests agree well with this relationship at low applied loads. There are no experimental data to assess the predictive abilities of this surface at higher applied loads however.

- The mean total stress can be inferred using the $K_0$ relationships proposed by Mayne and Kulhawy (1982). This allows a comparison to be made between compression induced by an external (mean total stress) and internal (suction) isotropic stress. There appears to be quite good agreement between both compression indices.

- The individual unloading stages are not comparable with the ISL - except for unloading at nominal suction. The effect of the suction in the sample is to suppress the swelling during unloading. If the final void ratios following unloading are plotted on the ISL, the applied stress from the ISL is approximately equivalent to the sample suction.
The swelling stages from the suction controlled tests (at nominal applied load) compare well with the ISL and the filter paper wetting tests.

A void ratio - suction - vertical stress relationship can be developed from the unloading data. Whilst this surface allows a relationship for swelling under load to be inferred, the limited experimental data for swelling under load does not agree well with the inferred surface.

The prediction is good for unloading but not for swelling. This indicates the path dependency of the relationship.

A limited number of tests were carried out on the London, Bangali and Black Cotton Clays. They serve to confirm the capabilities of the suction controlled oedometer. An indication of the extended test durations is seen for the more expansive clay soils. The data from the London Clay tests add further confirmation that at low applied stress unloading and swelling are broadly comparable.
<table>
<thead>
<tr>
<th>PEG conc. (g/l)</th>
<th>suction (kPa)</th>
<th>equiv. head (m)</th>
<th>flow (m³/s x 10^-10)</th>
<th>path length (m x 10^-4)</th>
<th>area (m² x 10^-3)</th>
<th>k (m/s x 10^-13)</th>
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<td>200</td>
<td>360</td>
<td>36</td>
<td>6.065</td>
<td>1.397</td>
<td>8.1073</td>
<td>2.903</td>
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<td>570</td>
<td>57</td>
<td>7.565</td>
<td>1.397</td>
<td>8.1073</td>
<td>2.287</td>
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<td>300</td>
<td>800</td>
<td>80</td>
<td>8.226</td>
<td>1.397</td>
<td>8.1073</td>
<td>1.772</td>
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<td>1100</td>
<td>111</td>
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<td></td>
<td>Average</td>
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Table 9.1 permeability measurements of Visking membrane
<table>
<thead>
<tr>
<th>TEST</th>
<th>DESCRIPTION</th>
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</thead>
<tbody>
<tr>
<td>NOTE</td>
<td>All samples consolidated one-dimensionally from a slurry to a maximum vertical stress of 200 kPa. A suction of 50 - 100 kPa remained following undrained unloading. Load-unload cycle between 50 kPa and 2 MPa unless otherwise stated.</td>
</tr>
<tr>
<td>KA1</td>
<td>Initial vertical stress of 50 kPa. nominal suction. Load-unload cycle undertaken with nominal suction. Test duration 7 days</td>
</tr>
<tr>
<td>KA2</td>
<td>Sample allowed to dry to a suction of 200 kPa before trimming into oedometer ring. Load-unload cycle undertaken at constant suction of 200 kPa. Swelling to nominal suction under 50 kPa vertical stress. Preparation 1 day, test duration 10 days</td>
</tr>
<tr>
<td>KA3</td>
<td>Sample allowed to dry to a suction of 300 kPa before trimming into oedometer ring. Load-unload cycle undertaken at constant suction of 300 kPa. Preparation 2 day, test duration 5 days</td>
</tr>
<tr>
<td>KA4</td>
<td>Sample allowed to dry to a suction of 750 kPa before trimming into oedometer ring. Load-unload cycle undertaken at constant suction of 750 kPa. Swelling to nominal suction under 50 kPa vertical stress. Preparation 2 day, test duration 12 days</td>
</tr>
<tr>
<td>KA5</td>
<td>Sample allowed to dry to a suction of 750 kPa before trimming into oedometer ring. Initial vertical stress of 50 kPa. Swelling to nominal suction under 50 kPa vertical stress. Preparation 2 days, test duration 3 days</td>
</tr>
<tr>
<td>KA6</td>
<td>Sample allowed to dry to a suction of 750 kPa before trimming into oedometer ring. Load to 1.5 MPa at constant suction of 750 kPa. Swelling to nominal suction under 1.5 MPa vertical stress. Unload under nominal suction. Preparation 2 days, test duration 14 days</td>
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**Table 9.2 test details for suction controlled oedometer tests on Kaolin**

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<tr>
<th></th>
<th>e100 upper</th>
<th>e100 lower</th>
<th>e100 ave.</th>
<th>e1000 upper</th>
<th>e1000 lower</th>
<th>e1000 ave.</th>
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<td>0.439</td>
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**Table 9.3 intrinsic compression and swelling properties of kaolin**
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<th>initial vertical stress (kPa) (preparation)</th>
<th>suction (kPa)</th>
<th>$K_0$</th>
<th>equivalent maximum vertical stress (kPa)</th>
<th>yield stress (kPa)</th>
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<td>0</td>
<td>0.61</td>
<td>200</td>
<td>210</td>
</tr>
<tr>
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<td>0.61</td>
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<td>475</td>
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<tr>
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<td>800</td>
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*Table 9.4 comparison between vertical stress, suction and yield stress*

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<th>Kaolin</th>
<th>London Clay</th>
<th>Bangali Clay</th>
<th>Black Cotton Clay</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Duration</td>
<td>Test ID</td>
<td>Duration</td>
</tr>
<tr>
<td>KA1</td>
<td>7</td>
<td>LC1</td>
<td>27</td>
</tr>
<tr>
<td>KA2</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KA3</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>KA4</td>
<td>12</td>
<td>LC4</td>
<td>16</td>
</tr>
<tr>
<td>KA5</td>
<td>3</td>
<td>LC5</td>
<td>8</td>
</tr>
<tr>
<td>KA6</td>
<td>14</td>
<td></td>
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</table>

*Table 9.5 test identification and duration for London, Bangali and Black Cotton Clays*
Figure 9.1 consolidation of Kaolin slurry under a range of applied suctions.
Figure 9.2 Consolidation of kaolin slurry using osmotic technique

(PEG = 300 g/l)

NOTE
Void ratio and suction values are estimated from intrinsic compression data.

- mc = 83.26%
e = 2.173
s = <1 kPa

- mc = 66.8%
e = 1.743
s = 10 kPa

- mc = 46.76%
e = 1.22
s = 150 kPa

Diagram shows compression and moisture loss over time (hours).
Figure 9.3 permeability measurements of Visking semi-permeable membrane

Average permeability
\[ k = 2.127 \times 10^{-13} \text{ m/s} \]
Figure 9.4 Multi-stage calibration test for PEG
Figure 9.5a Individual suction measurement of sample at equilibrium (PEG = 290 g/litre)

Figure 9.5b Calibration test at PEG = 350 g/litre showing loss of contact between IC tensiometer and sample
Figure 9.6 Initial sample preparation from reconstituted state.
Figure 9.7 Oedometer compression under constant suction following air drying.
Figure 9.8 One dimensional compression from a slurry in oedometer.

Figure 9.9 "Undrained" unloading and removal of sample from oedometer. Note: some uptake of water.
Figure 9.10: Air drying to required suction

Figure 9.11: Oedometer compression under constant suction
Figure 9.12 upper and lower bounds, and possible stress paths for one-dimensional loading of a fully saturated sample with a suction.
Figure 9.13 Schematic representation of suction controlled tests
**Figure 9.14** Comparison of intrinsic compression data for Kaolin

**Figure 9.15** Comparison of intrinsic swelling data for Kaolin
Figure 9.16 Comparison between one-dimensional compression and swelling at nominal suction and intrinsic compression and swelling data (test KA1)

Figure 9.17 One-dimensional compression at nominal suction (test KA1)
figure 9.18 one-dimensional compression at 200 kPa suction  
(test KA2)

figure 9.19 one-dimensional compression at 300 kPa suction  
(test KA3)
figure 9.20 one-dimensional compression at 750 kPa suction
(test KA4)

figure 9.21 yield locus for Kaolin under one-dimensional compression at a range of suctions
Figure 9.22 summary of one-dimensional compression at a range of suctions (tests KA1 - KA4)
Figure 9.23 Predicted effect of monotonic increase in suction under a range of vertical stresses.
Figure 9.24 summary of the effect of increasing suction under a range of vertical stresses.

Figure 9.25 comparison between effects of increasing suction from oedometer ($\sigma_v = 50$ kPa) and drying tests.
Figure 9.26 Void ratio - suction - vertical stress relationship for kaolin under compression.
Figure 9.27: Void ratio - suction - vertical stress relationship for kaolin under compression.
Chapter 9  page 371

**figure 9.28** one-dimensional compression at nominal suction (test KA1)

**figure 9.29** one-dimensional compression at 200 kPa suction (test KA2)
Figure 9.30: One-dimensional compression at 300 kPa suction (test KA3)

Figure 9.31: One-dimensional compression at 750 kPa suction (test KA4)
**Figure 9.32** Summary of one-dimensional compression at a range of suctions (tests KA1 - KA4)

**Figure 9.33** Yield curves for kaolin under one-dimensional compression at a range of suctions
Figure 9.34 Void Ratio - Suction - Mean Total Stress Relationship for Kaolin Under Compression
Figure 9.35: Void ratio - suction - mean total stress relationship for kaolin under compression.
Figure 9.36: Comparison between compression induced by suction and mean total stress (75 kPa confining pressure)

Figure 9.37: Comparison between compression induced by suction and mean total stress (150 kPa confining pressure)
figure 9.38 comparison between compression induced by suction and mean total stress (300 kPa confining pressure)

figure 9.39 comparison between compression induced by suction and mean total stress (630 kPa confining pressure)
Chapter 9  page 378

**Figure 9.40 One-dimensional Unloading at Nominal Suction**

**Figure 9.41 One-dimensional Unloading at Constant Suction (200 kPa)**
Figure 9.42: One-dimensional unloading at constant suction (300 kPa)

Figure 9.43: One-dimensional unloading at constant suction (750 kPa)
Figure 9.44 comparison between confined swelling (oedometer), unconfined swelling (wetting test) and unload data at constant suction.
Figure 9.45 Void ratio - suction - vertical stress relationship for kaolin under unloading.
figure 9.46 void ratio - suction - vertical stress relationship for kaolin under unloading
Figure 9.47: Comparison between interpolated unloading from 1500 kPa and swelling under load (test KA6)

Figure 9.48: Comparison between interpolated swelling surface at $\sigma = 50$ kPa and swelling stages of tests KA2, KA4 and KA5
figure 9.49 comparison between one-dimensional compression at nominal suction and intrinsic compression line for London Clay (test LC1)

figure 9.50 effect of suction on one-dimensional loading and unloading for London Clay (tests LC1 & LC4)
Figure 9.51 comparison between confined swelling (oedometer) and unconfined swelling (wetting test) for London Clay.

Figure 9.52 comparison between one-dimensional compression and swelling at nominal suction and intrinsic data for Bangali Clay (test BA1).
Figure 9.53: Effect of suction on one-dimensional loading and unloading for Bangali Clay (test BA1 & BA4)

Figure 9.54: Comparison between confined and intrinsic swelling data for Bangali Clay (test BA4)
Figure 9.55 Comparison between one-dimensional compression at nominal suction and intrinsic data for Black Cotton Clay (test BCC1)
Chapter 10

Conclusions and recommendations for further research

10.1 INTRODUCTION

The principle conclusions and recommendations for future research are presented in this chapter. The conclusions are presented under three headings, literature review, equipment and technique development and experimental results. Finally, suggestions for future research direction are given.

10.2 LITERATURE REVIEW

The principle conclusions from the literature review are as follows:

1. Soil moisture interaction for granular and clay soils are fundamentally different.

Suction capacity, $C$, is defined as the linear phase of drying for a normally consolidated soil at suctions below the point of desaturation, eqn 10.1

$$C = \frac{\Delta w}{\Delta \log (\text{suction})}$$  \hspace{1cm} 10.1

For granular soils, $C$, reduces progressively with smaller particle sizes. The opposite is seen with clay soils, where there is an increase in suction capacity for the smaller clay particles.

Granular soils can generally be considered as incompressible, relatively inert, soil structures. Their attraction for moisture is principally limited to the capillarity relating to the pore structure. Clay soils however, have a complex attraction process for moisture. In addition to capillarity, they have an independent surface attraction for moisture. The actual mechanism operating at this level has not been universally agreed upon, although the double layer theory proposed by Bolt (1956) is generally accepted.

There are two possible mechanisms which might instigate desaturation in a soil, air being drawn into the soil mass, or air forming within the soil mass. It is postulated that for granular soils a combination of air entry and loss of adhesion at the particle surface leading to air formation best describes the mechanism of desaturation. Loss of adhesion at the clay surface is unlikely due to
the clay-water bonding which takes place, and pure cavitation is also thought unlikely. It is suggested that air entry is the most likely cause of desaturation in clay soils. There is limited experimental evidence, although the available data supports these concepts.

2. Account should be taken of hysteresis in all considerations of soil water interaction. The direction of moisture movement can have a significant influence on the interpretation of engineering behaviour. Hysteresis is not only apparent in soil but in all porous media, including measurement techniques such as the filter paper method.

3. Soil stiffness increases with increasing suction. This is applicable equally to saturated and partly saturated soils.

4. Swelling can be limited by the application of an applied stress.

5. Swelling and swell pressure determination are significantly path dependent for partly saturated soils.

6. A fundamental difference between unloading at constant suction and swelling at constant load has been seen for compacted soils.

7. Measurement of the total stress state, \( \sigma_v \) & \( \sigma_h \) may lead to a better understanding of swelling behaviour. This has been demonstrated for the special case of compacted soils, but could prove equally important for reconstituted behaviour.

8. Suction measurement has received innovative and determined thinking over many decades. A number of techniques have been developed, however many have limited application or range.

The development of the IC tensiometer (Ridley, 1993) represents a significant advance in the measurement of soil moisture suction. It offers quick, accurate, repeatable measurements of matrix suction to in excess of 1500 kPa.

The filter paper technique is one of the most widely used suction measurement techniques and has a broad range of measurement (30 - 30,000 kPa). Ridley (1995) identified hysteresis associated with the use of a wet filter paper for wetting a sample. He published data defining a calibration better suited to the use of wet filter papers. Data from this project supports Ridley's calibration.
10.3 EQUIPMENT AND TECHNIQUE DEVELOPMENT

10.3.1 Suction controlled oedometer

A principle aim at the outset of this research project was to develop and implement a system for controlling suctions under atmospheric conditions within the range of the IC tensiometer, i.e. up to 1500 kPa. This would allow laboratory testing to reflect the field conditions more closely.

The use of the osmotic potential of a salt solution (PEG) across a semi-permeable membrane to control matrix suction is an elegant solution to this problem.

A comprehensive review of existing practice highlighted the need for a complete and rigorous assessment of the technique, the principles of operation and its application in the soils laboratory.

The development centred on use of the osmotic technique in an oedometer. A number of significant advances on existing systems were made in the application of the technique, in addition to the development of a greater understanding of the principles underlying osmotic control.

1. Moisture content

In the new system the MASS not VOLUME of the moisture exchanged between the soil sample and the PEG solution is recorded. This is done by placing the stock solution on an electronic balance. Data is recorded on a computer via the RS232 port on the balance. This approach is independent of temperature change and therefore eliminates the complexities associated with the measurement of volume.

2. Evaporation

Fundamental to the determination of moisture content changes from the mass of the stock solution is the definition of the extent of evaporation from the system.

A detailed investigation into evaporation from the surface of the stock solution led to the following measures. Firstly a layer of viscous silicone oil is placed on top of the PEG solution and although not eliminating evaporation it provides a significant reduction. Secondly the specific area for evaporation is minimised by placing an upturned glass culture dish on the exposed surface of the oil. This combination results in a reduction of evaporative losses from the stock solution to a level such that it does not require any subsequent attention in the calculation of the...
moisture exchange. This study of evaporative losses also resulted in two additional revisions:

The use of PVC tubing as opposed to the more common silicone based tubing. The latter was found to give significant evaporative losses through the walls of the tubing.

A latex membrane seal between the top cap and the cell. Trials showed other seals, (i.e. cling film) to be inadequate in preventing moisture loss

3. PEG solution circulation

The underdrainage from beneath the semi-permeable membrane consists of a layer of woven nylon mesh with a single inlet and outlet diametrically opposed on the base of the cell (there are no circulation groves in the base). A layer of semi-permeable membrane is then clamped on top of the mesh, separating the PEG solution from the soil sample. A series of tests showed that this simple approach provides the most effective answer. A combination of drainage length and the natural dispersive nature of a woven mesh is the key to ensuring adequate circulation of the solution and flushing of the underside of the membrane. A peristaltic pump is used for circulating the PEG solution as this provides controllable flow rates and operates with continuous tubing, a major advantage in reducing evaporative losses.

4. Suction measurement

An IC tensiometer is incorporated into the top cap of the oedometer, allowing continuous measurements of suction to be made during testing. A fixing mechanism is incorporated which allows the tensiometer to be changed as necessary.

5. Data capture

All data is captured and recorded on a computer. This includes vertical compression measurements, moisture transfer from the base of the sample to the stock solution, and matrix suction at the top of the sample. A triaxial stress path software application is used for data capture. This has the advantage that a particular stress path can be followed based on real time stress, strain or suction measurements.

6. Calibration of the osmotic potential of PEG solution

It was found that the existing calibration between the concentration of PEG and the osmotic potential is not representative of the matrix suction generated across the membrane boundary. A two-stage calibration has been undertaken, under conditions which reflect the testing
environment as closely as possible.

An initial calibration was carried out, in which the tensiometer was placed directly in contact with the semi-permeable membrane. Matrix suction across the membrane was measured for PEG solutions of a range of concentrations. The system as a whole was then calibrated by placing a Kaolin sample in the oedometer cell, and a number of PEG solutions with increasing concentrations - based on the data from the tensiometer tests - were circulated through the drainage system until equilibrium conditions were observed.

The values from both calibrations are in excellent agreement with one another and form a well defined trend lying between those of the earlier authors. The similarity between the two calibrations is indicative of the efficiency of the oedometer design in that the full osmotic pressure developed across the membrane on its own (defined in the initial calibration) is realised throughout the soil sample in the final calibration.

10.3.2 Technique for obtaining wetting curve.

A revised technique for the wetting of samples is also presented. During the latter stages of wetting, the amount of moisture required to reduce the sample suction increases considerably.

The addition of two wet filter papers adds approximately the same amount of moisture to the sample on each occasion. At suctions below 1000 kPa this will only reduce the sample suction by a small amount. It can take several months to determine the final stages of the wetting up curve using this technique. This same phenomenon is seen during drying, although it is easily overcome by exposing the sample in the atmosphere for an increasing amount of time between measurements.

Utilising the low permeability of the semi-permeable membrane, a controlled, continuous addition of moisture could be made to the sample during wetting. Subsequent use of a specially manufactured latex membrane and an IC tensiometer allowed suction measurements to be made at intervals of suction and not moisture content during these final stages of wetting.

10.4 EXPERIMENTAL RESULTS

Four soils were studied experimentally. The experimental data has been presented in two separate chapters, "Intrinsic and soil moisture characteristics (chapter 8)", and "Suction controlled oedometer testing (chapter
9), and conclusions will be drawn on these separately.

10.4.1 Intrinsic and soil moisture characteristics

1. The measured intrinsic properties and additional data from the literature were compared with concepts proposed by Burland (1990). The experimental data and that from the literature compliment the existing concepts. The limitation of Burland's empirical relationships to soils above the plasticity "A" line appears well founded. Those soils below this line do not fit the void index relationship well. Viewed in isolation however, these data can be described by an amended form of the expression given by Burland.

2. The form of the soil moisture characteristic for clay soils differs considerably from the idealised concepts applied to porous media. In principle, clay soils exhibit fully saturated shrinkage, a gradual transition at the point of desaturation and continued shrinkage at low moisture contents.

3. The expansive soils remain fully saturated to very high suctions (13 MPa for the Black Cotton Clay). Under air dry conditions they remain at a very high degree of saturation (> 80%). This adds further evidence to the hypothesis that many natural and compacted soils consist of fully saturated clay "packets" in a partly saturated matrix.

4. The current definitions of air entry value and shrinkage limit are somewhat arbitrary with respect to plastic soils. A clear definition of the actual point of air entry is needed and consideration should be given to the use of the term "limit of volumetric shrinkage" (Ridley, 1993).

5. There are a number of soil models for partly saturated soils. Most have difficulty in accounting for both saturated and partly saturated soil behaviour, and in particular the transition between the two phases. The model proposed by Toll (1988, 1995) is a useful conceptual model that accounts for saturated and partly saturated soil behaviour. It also has the ability to allow representation of overconsolidated soils and drying / wetting cycles.

6. The ability to accurately predict the soil moisture characteristic, even for a single drying curve would be a useful tool. However, whilst the curve fitting abilities of the published expressions are very good (Fredlund and Xing, 1994), their ability to predict from a restricted data set are limited. This ability is further limited if the data contains both saturated and partly saturated data.

7. Evaporation from fully saturated clay soils can be directly compared to that from a free water surface.
8. The controlling parameter over the rate of evaporation varies with soil type. There is no single parameter which exerts an overall control over the rate of moisture loss.

9. Cracking occurred in the more expansive soils during drying. The cause of cracking is postulated to be due to tensile stresses in the periphery of the sample. The increase in tensile stress is offset to an extent by the compressive stress resulting from the increase in suction associated with moisture loss. This effect is reduced in the more expansive soils due to the steeper gradient of the moisture content-suction relationship.

10. There is no apparent moisture content variation through a sample during drying until the average moisture content approaches its air dry value.

10.4.2 Suction controlled oedometer tests

The principle conclusions from this experimental stage of the research project are summarised below:

1. The ability of the osmotic technique, using a PEG solution and semi-permeable membrane, to effectively control the matrix suction in a sample has been clearly demonstrated.

2. The advantages of continuous measurement of moisture content, stress and compression during a test, allowing the incorporation of real time values into stress path oedometer testing have been shown.

3. The advances in understanding and application of suction measurement and control techniques have resulted in an improved appreciation of the importance of suction in soil mechanics.

4. The intrinsic swelling line (ISL) appears to predict the swelling paths under nominal applied stress quite well. This comparison applies to confined and unconfined swelling, under both saturated and partly saturated conditions.

5. The limitations of the application of $K_0$ principles to account for fully saturated soils with suctions have been identified.

6. The estimation of mean total stress under conditions of constant suction can be made using accepted effective stress relationships. The extension of this to account for unloading and swelling paths is less certain.

7. Under similar conditions (nominal suction) the suction controlled oedometer can be compared to conventional oedometer apparatus.
8. Increasing the sample suction increases the yield stress. A yield locus has been defined for the fully saturated state for Kaolin.

9. Loading and unloading surfaces in void ratio, applied stress and suction space have been developed. These are applicable for monotonic loading and unloading paths.

10. The prediction of drying under low applied stress from the loading surface is good. The application under higher applied stresses has not been proven experimentally.

11. The prediction of swelling from the unloading surface under nominal applied stresses is reasonable. Swelling under high applied stresses cannot be predicted with confidence from the unloading surface.

12. The effect of the low permeability of expansive soils in increasing test durations dramatically may influence future test programmes. This will be of particular importance where partly saturated soils are concerned.

10.5 FUTURE RESEARCH RECOMMENDATIONS

There are a number of concepts, techniques and experimental programmes which could form the basis of future research, leading to a better understanding of soil behaviour and improved test procedures.

1. Fabric studies

The influence of fabric is perhaps more important in partly saturated soils than in saturated soils. It is important to complete parallel studies of the fabric changes which occur as a result of both suction and applied stress changes. In particular a comprehensive analysis of pore size distribution during drying and wetting should provide important data on the influence of pore geometry on the soil moisture characteristic. This would involve porosimetry measurements, which are both destructive and difficult to perform on clay soils without disrupting the absolute pore structure.

2. Radial stress measurement

From the literature we have seen the changes in radial stresses which can occur as a result of wetting. An investigation of the effects of both suction and applied stress changes on radial stress development in both saturated and partly saturated soils is an important research objective. From the limited available data we have seen that the influence of suction on radial stress can be significant and difficult to predict. Existing predictive methods need assessment in the light of
good experimental data.

The measurement of radial stress in the oedometer has received attention by many research groups. There are a number of possible experimental approaches, and a summary is presented in appendix A. Careful thought is needed in the selection of the appropriate experimental technique such that the results can be viewed with confidence.

3. **Yield locus**

The yield locus seen for the tests on Kaolin is of a similar form to that seen in many partly saturated soil models. The extension of the definition of this locus through the transition phase to a partly saturated state, will assist in developing better links between saturated and partly saturated behaviour.

4. **Path dependency of swelling**

The literature and limited experimental data supports the hypothesis that swelling is highly path dependent. Further investigation into the extent of this path dependency is needed. This would require the measurement of the radial stress to allow adequate analysis to be undertaken.

5. **Suction / moisture content distribution during drying**

Simple experiments were carried out to investigate moisture content variations in a sample undergoing drying. Repetition of this type of test with measurement of suction as opposed to moisture content would enable a more sensitive soil parameter to be measured during drying. In addition such an investigation could be undertaken on a continuous, non-destructive basis. This should be carried out on a range of soil types and structures, to develop an understanding of moisture loss and perhaps moisture flow from soils during drying. Scale effects may need careful attention in this programme.

6. **Further equipment development**

There are a number of areas of the suction controlled oedometer which would benefit from further attention.

Under high stresses, and in particular with high PEG concentrations, circulation proved difficult on occasion. A rigid support for the semi-permeable membrane, perhaps between the membrane and the sample may be a solution.
On occasion the required suction could not be achieved or could not be maintained over a long period of time. Initially it was felt that the problem lay with the positive pressure generated in the solution during circulation. Whilst this has an influence, it is also felt that possible membrane or PEG degradation may occur, reducing the effectiveness of the osmotic technique. A further study of membrane and osmotic solution combinations may yield a more robust answer. It should be noted that the application of semi-permeable membranes in soil testing is significantly different from their intended use as a medical application, where they are subjected to low osmotic potentials, over short periods of time.

The advances in understanding of the physics of the IC tensiometer, and the causes of tension breakdown have identified possible limiting factors controlling the range of measurement. If future research is to focus on partly saturated clay soils - in particular reconstituted partly saturated clay soils - this range will need to be extended. This should be undertaken in parallel with an increase in the range of suction control.
APPENDIX A

Review of lateral stress measurement in the oedometer

A.1 INTRODUCTION

There are two distinct philosophies with respect to measurement of lateral stresses, termed passive or active systems, Benoit and Lutenegger (1991). We will look at the details of a number of variations on each concept, give a critical review of each, explore some alternative options and give a brief summary of the most suitable. A summary of the systems evaluated is given in table A1.

A1.1 Passive - Semi-Rigid Systems

Lateral strains generally occur through yielding of a strain gauged thin section of the oedometer ring. These strains are converted to equivalent lateral stresses through a separate calibration.

A1.2 Active - Compensating Systems

These systems are based on the same principles as the passive systems in many instances, but employ a separate mechanism to correct the induced strains, and create an overall condition of zero lateral strain. The lateral stress in the sample is defined as that required by the compensatory system to reduce or maintain strains close to zero.

A.2 PASSIVE - SEMI-RIGID SYSTEMS

Kornomik and Zeitien, 1965 (fig A1)

A modification of a standard oedometer ring, 112.5mm dia, 26.2mm high. A thin section 0.3 mm thick was turned from the original ring. This acts as a flexible membrane during testing. Strain gauges are mounted and calibrated against applied air pressure. These results are compared with theoretical deflections for expansion of a cylinder.

Bellotti et al, 1975 (fig A2)

An interesting alternative, using a square oedometer cell. Two types of cell were utilised, both 45mm across flats and 20mm deep. In one system the wall thickness was reduced to 0.8 mm around the complete
periphery. Strain gauges were placed on all four faces and wired to form a full bridge. The second cell utilised commercial diaphragm transducers, flush mounted in each face. The cell was 17 mm thick in both systems. Cell Performance details are not given although some comparison between both types of cell is made.

**Tan Tjiong-Kie, 1981 (fig 3)**

This oedometer consists of two symmetrical rigid half rings held together by two thin steel strips. Lateral stress forces the two halves apart, straining the steel strips. Strain gauges are mounted, two active and two passive in a full wheatstone bridge. A waterproof seal is made internally to prevent leakage through the join between the two sections. No details of calibration or performance are given.

**Dyvik et al, 1985 (fig A4)**

This system consists of a thin section oedometer (67.76mm dia, 26.67mm high) with a fluid filled chamber behind, acting to transfer lateral stresses to a pressure transducer connected to the cell.

Developed by Geotechnics International Inc. Madison, MA, USA, the membrane is a thin teflon sheet, offering low friction and good transmission of the soil stresses. The fluid chamber is filled with deaired water and a stiff diaphragm type pressure transducer is connected directly to the cell body.

The cell is submerged in a temperature controlled water bath to reduce the temperature effects on the measuring system (this is significant as it is a closed fluid system).

The pressure transducer is a Kulite 5U-60 (700 kPa) silicon diaphragm type. Calibration is carried out by filling the cell with water and applying a dead load.

It has been used to study four clays, one lightly, one heavily overconsolidated and two offshore clays.

**Schreiner, 1988 (fig A5)**

An axial load cell is incorporated in the cell wall. It consists of a moveable "piston" supported by four thin axially stressed webs. Each web is strain gauged on both faces and a similar number of strain gauges (8) are mounted on a dummy pillar located in the central core of the load cell. A drift monitoring system is used due to the long duration of the tests. As axis translation is used to control suctions in the sample, the elevated air pressure is applied to the rear face of the load cell also. As such the load cell acts as a differential pressure transducer.
Kolymbas and Bauer, 1993 (fig A6)

A flexible strain gauged floating ring oedometer. There is one active ring and one outer dummy ring both strain gauged allowing for a full bridge.

The oedometer is used in a temperature controlled room, and is primarily designed for cohesionless granular soils.

Senneset and Janbu, 1994 (fig A7)

A three section split ring oedometer. Each section consists of a rigid cell with a thin steel membrane "window". A LVDT is mounted behind each face to monitor lateral deflections. The three sections are held in an adjustable three jaw engineering lathe chuck. This allows the ring to be clamped to a prepared sample. Horizontal deflections are measured and converted to equivalent lateral stresses through application of fluid pressure in place of the sample. The cell is 54.3mm dia, 20mm deep. The LVDT's have a range of +/- 2mm.

Ting et al, 1994 (fig A8)

This consists of a 104mm by 50mm deep perspex cell with an aluminium jacket for rigidity. Local measurement of total stress at the top, base and mid-height of the sample is made together with base and mid-height pore pressure.

All measurements are made with flush mounted Druck diaphragm type pressure transducers. The mid-height probe is 3mm diameter and its surface is made flush to the cell by the addition of RTV silicon rubber sealant.

Closed loop controlled gradient tests are carried out, using servo controlled air pressure. To date kaolin slurry has been tested.

A.3 ACTIVE - COMPENSATING SYSTEMS

Escario, 1978 (fig A9)

A 3mm thick, 70 mm diameter, 20 mm deep box section cell is utilised in this system. The inner cell face (in contact with the sample) is 0.3 mm thick, and is strain gauged to monitor lateral strains. The cell is restrained in a rigid frame which both locates the cell and creates an air tight seal as axis translation is used to control suction in the sample. Air pressure is applied to negate any lateral strains and offset the
effects of the elevated sample air pressures.

**Ofer, 1981 (fig A10)**

An oedometer with a strain gauged thin section and air compensation to achieve zero lateral strains overall. No indication of the method of reducing the observed strains is given but it is assumed to be manual.

Two series of tests are reported making comparison between passive and active modes of measuring lateral stresses. Firstly lateral straining was allowed to occur, monitored and equivalent lateral stresses calculated. In the second, similar set of tests a condition of zero lateral strain was maintained by the application of air pressure to the rear face of the gauged section. The results are given in table A2.

The cell is 70mm dia, 42mm deep with a central section 0.8mm thick.

**Edil and Alanazy, 1992 (fig A11)**

An Ofer type compensated cell, 63.5mm dia, 50.8mm deep with a thin section 0.38mm thick.

A servo controlled air pressure is used to null any strains in the thin section.

Calibration is carried out by comparing the applied internal air pressure to the external pressure required to null the observed strains.

### A.4 CRITIQUE OF EXISTING SYSTEMS FOR PROPOSED WORK

The principle (ideal) requirements of a lateral stress measuring system can be summarised as follows:

- Zero radial strain.
- Stable output over a long time period.
- Unaffected by the actions of consolidation or swelling of the sample.
- Robust to the operating environment.
- Quick response time.
- Be neither a semi-rigid or compensating system! i.e. permit zero lateral strain without recourse to compensatory measures.

In practice it is impossible to achieve such a system, and an assessment must be made of the merits and drawbacks of each attribute to achieve the best solution for the proposals. A summary of the rigidity of the systems considered is given in table A1.
Komomik

A common principle, applied in many systems. Its shortcomings are that lateral strains are allowed to occur. The effect of lateral straining on the development of lateral stress may be significant.

Bellotti

An interesting alternative to the circular oedometer ring. It would solve many of the machining difficulties, and allow incorporation of commercial transducers easily. However the stress discontinuities associated with the shape would need careful assessment.

Tan Tjong-Kie

Stress discontinuities due to expansion being limited to one axis is the major consideration here. Also the effects of having a waterproof membrane between the cell and the soil sample would need attention.

Dyvik

This is a very good mechanism as it goes a long way towards satisfying the ideal requirements of such a measurement system. It is a responsive, robust and stable system. If it can be sufficiently deaired it may represent the best approach, however this may also be its drawback. To achieve very good deairing is extremely difficult and even minute quantities of air will increase the compliance of the system considerably. The effects of the deflection of the diaphragm of the pressure transducer can be reduced by making the ratio between the area of the membrane and that of the diaphragm as large as possible, in practice the physical movement of the membrane due solely to this action is likely to be negligible and can be quantified theoretically quite easily. Compressibility of the fluid and overall system compliance are of a much smaller scale of magnitude than the above and should not present any great errors.

The main drawbacks with the Dyvik system is the lack of any indication of the success of any deairing prior to testing and also of any leakage during testing, which could allow a gradual deflection of the membrane and subsequent errors in the true value of the lateral stress.

Also as the measuring system is a closed fluid filled system the effects of temperature can be dramatic. The cell is submerged in a temperature controlled water bath to ensure some consistency in the operating environment.

Schreiner

Lateral movement of the "piston" is necessary to register a strain voltage reading and this is brought about
by the lateral expansion of the sample. However this will usually be influenced by the vertical displacement of the sample, hence the displacement vector will be inclined. The load cell is very rigid when loaded in a true axial sense, however it is susceptible to bending when the load is not axial. This could result in the piston coming into contact with its running bore in the cell wall, and the resulting friction will mask the true lateral stress values. Unfortunately it is difficult to determine when such an action is taking place, particularly as there is only one load cell in operation.

Furthermore, as the load cell operation is open and unsealed, it was necessary to place a membrane between the cell and the sample. This will to some extent detract from the stiffness of the load cell (a key feature in its selection). Also the cell action effects were not assessed and again these will reduce the effective stiffness of the load cell.

Kolymbas

A flexible system which has been critically assessed by the authors and its limitations identified. As such it is intended for use on cohesionless granular materials at lateral stresses of less than 100 kPa.

Senneset

This system represents a complex approach to the problems of lateral stress determination without any real improvement. Its key advantages are the ability to clamp the ring to a prepared sample and induce a lateral stress, or to leave a pre-defined gap between the ring and the sample allowing limited initial lateral swelling. However for the proposed work neither of these are priority requirements and the remainder of the system is not suited to the proposals.

Ting

Commercially available flat faced, diaphragm transducers are mounted in the cylinder wall. A flexible silicone rubber compound is used to form a curved surface on the transducer that matches that of the cylinder. The ability of the rubber compound to transmit lateral stresses, its compliance, frictional properties and uniformity of application are unclear.

Escario

The effects on strain measurements of clamping the strain gauged ring are unclear. Considering this clamping must also provide a seal against elevated air pressures (as axis translation is used to control suctions) this could be considerable. This may be further exacerbated by the use of a complete, circumferential thin section, which would clearly affect the rigidity of the ring. The application of a compensating air pressure to reverse any measured strains is made manually, leading to a non uniform
correction of the developed lateral stress.

Ofer

A further development on the above system with compensation for the observed strains. The main drawback is the cyclic, stepped correction of these strains. This is thought to have some affect on the measured values in comparison to the true lateral stresses.

Edil

A design based on that by Ofer, but with a servo-controlled system for applying compensatory air pressure. The allowable lateral strains would also appear to be quite high.

A5 ADDITIONAL OPTIONS

Cambridge type boundary stress cell

This cell, of the type developed by Bond (1989) for use in the model pile research at Imperial College, measures radial and shear stress components, allowing the effects of shear stress to be separated from the true lateral stresses during consolidation.

The limits of the cell are its size, it is understood that its current minimum dimensions are approximately 30mm by 30mm on plan. It may be possible to reduce this and perhaps make the contact face concave to suit the oedometer cell. An alternative would be to make the oedometer cell square allowing for four flat faces for mounting the cell flush. Friction may be a problem, particularly at the boundary.

Piezo Lateral Cell

A radial stress measuring system developed at MIT by Baligh et al (1985), consisting of a sealed reservoir similar to the system employed by Dyvik although on a more compact scale. Further development by Standing (1993) led to an improved device which although not constructed provides a very compact and elegant design.

Triaxial Testing

The measurement of lateral stress in the triaxial apparatus is well documented, Benoit and Lutenegger (1991). However whilst an attractive option in terms of determination of the lateral stress, the development, and incorporation, of a PEG system of suction control into the triaxial cell is far from
straightforward. For these reasons it was decided to concentrate on the oedometer as the testing platform. Thus triaxial based concepts are not considered here.

A6 SYSTEMS FOR FURTHER CONSIDERATION

The systems most applicable for the proposed work, and worthy of shortlisting are:

Dyvik / Baligh designs

These two share many common features and both benefit from the attention to the second system by Standing (1993).

Additional features would be the addition of strain gauging to the inner face of the membrane. This would assist in quantifying the compliance of the system in operation, the success of deairing, adverse temperature effects and any leakage from the fluid system during testing. Ideally the thin section should form only part of the circumference of the cell, this would allow more than one such section to be established thus giving cross referencing of measured values. There could be some problems however associated with machining thin sections as a portion of the circumference of the cell, as opposed to a complete thin section.

The main drawbacks of this system are deairing and temperature effects. Careful design would assist greatly in the solution of the former, the temperature effects may require the use of a temperature controlled bath.

Ofer

The machining of an oedometer ring with a circumferential thin section is relatively straightforward. Subsequent strain gauging and incorporation of servo-controlled compensation systems are also routine laboratory procedures. Modern measurement and control techniques would greatly reduce the tolerance allowed in the existing compensation systems.

The main drawback of this system is the use of a compensating system for lateral stress determination. The effects of allowing the soil to deform and then correcting those deformations affects the measured values.

Bellotti

The use of a square cell has much to recommend it. The installation of strain measuring instrumentation
is far more easily achieved, whether it be commercial diaphragm transducers or strain gauged thin sections, as all machining is performed on flat sections. The machining of the basic cell is also easily achieved. It is straightforward to install a number (four) of such devices in the cell giving more confidence in the measured stresses. The use of such a cell in combination with any of the measuring systems could be achieved.

The uncertainty in this system lies in the shape of the sample and the extent of stress non-uniformities which occur as a result of the symmetry of the cell.

A6.1  A note on compensating systems

The effects of allowing the soil to strain, even to small magnitudes can have a significant effect on the value of lateral stress measured, as seen earlier in the results from Ofer (1981). Okochi and Tatsuoka (1984) looked at the effect of varying the radial strain and its effect on \( K_0 \) values for pluviated sand samples in the triaxial apparatus. They found that varying the radial strain by \( \pm 0.015\% \) resulted in \( K_0 \) varying between 0.45 and 0.6 and suggest that to limit the error in \( K_0 \) to \( \pm 0.05 \), \( \varepsilon_r < \pm 0.01\% \) (3.75 \( \mu \text{m} \) for a 75mm diameter sample).

Although direct comparisons between inelastic granular materials and clay soils should not be drawn, if the strain levels observed allow the soils to move beyond its \( Y_1 \) elastic boundary (a difficult boundary to define experimentally) some of the effects seen on granular material will be experienced. Additionally the stresses to return a sample to a condition of \( \varepsilon = 0 \) are not necessarily the same as those mobilised in causing the initial strain, and neither are equal to the true lateral stress that exists under conditions of \( \varepsilon_r = 0 \). Furthermore the effects of changing soil stiffness with radial strain and the hysteretic behaviour due to recompression may need consideration.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Type</th>
<th>Range (kPa)</th>
<th>Max. Deflections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Komornik</td>
<td>Semi-Rigid</td>
<td>150</td>
<td>$\varepsilon &lt; 0.1% @ 100$ kPa</td>
</tr>
<tr>
<td>Bellotti</td>
<td>Semi-Rigid</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tan</td>
<td>Semi-Rigid</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dyvik</td>
<td>Semi-Rigid</td>
<td>500</td>
<td>$\varepsilon &lt; \text{Transducer Defl}$</td>
</tr>
<tr>
<td>Schreiner</td>
<td>Semi-Rigid</td>
<td>500</td>
<td>$\varepsilon = 0.0009%$ (Theoretical)</td>
</tr>
<tr>
<td>Kolymbas</td>
<td>Semi-Rigid</td>
<td>200</td>
<td>$\varepsilon = 0.16%$</td>
</tr>
<tr>
<td>Senneset</td>
<td>Semi-Rigid</td>
<td>800</td>
<td>$\varepsilon = 0.147%$</td>
</tr>
<tr>
<td>Ting</td>
<td>Semi-Rigid</td>
<td>200</td>
<td>$\varepsilon = 0.02%$</td>
</tr>
<tr>
<td>Escario</td>
<td>Compensating</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ofer</td>
<td>Compensating</td>
<td>600</td>
<td>$\varepsilon = 0.3%$</td>
</tr>
<tr>
<td>Edil</td>
<td>Compensating</td>
<td>50</td>
<td>$\varepsilon = 0.6% (0.19\text{mm})$</td>
</tr>
</tbody>
</table>

*Table A1 summary of lateral stress measuring systems*

<table>
<thead>
<tr>
<th>Vertical stress (kPa)</th>
<th>Horizontal stress (kPa) (Active, $\varepsilon_r = 0$)</th>
<th>Horizontal stress (kPa) (Passive, $\varepsilon_r \neq 0$)</th>
<th>Radial strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>95</td>
<td>250</td>
<td>100</td>
<td>0.054</td>
</tr>
<tr>
<td>190</td>
<td>362</td>
<td>220</td>
<td>0.229</td>
</tr>
</tbody>
</table>

*Table A2 comparison of radial stresses for passive and active modes, after Ofer (1981)*
figure A1 Kormornik and Zeitlen (1965)

figure A2 Bellotti, Formigoni & Jamiolkowski
Appendix A page 408

**figure A3 Tjong-Kie (1981)**

**figure A4 Dyvik, Lacasse & Martin (1985)**
Figure A5 Schreiner (1988)

Figure A6 Kolymbas & Bauer (1983)
Steel membrane

--- Top cap
--- Filter
--- Soil specimen
--- Filter
--- Water-filled chamber
--- Pore pressure transducer
--- Cable

**figure A7** Senneset (1989)

--- Piston total stress transducer
--- Bleed valve
--- Upper drainage port
--- Horizontal total stress transducers
--- 104 mm
--- 50 mm
--- Sample
--- Mid-plane pore pressure transducer
--- Base total stress transducer
--- Base pore pressure total and differential transducers

**figure A8** Ting, Sills & Wijeyeskera (1994)
figure A9 Escario (1978)

1. Thin wall odometer ring
2. Casing ring
3. Strain gauge
4. Electrical wires
5. Air pressure inlet

figure A10 Ofer (1981)
1. Main ring.  2. Casing ring.  3. Strain gages  

*figure A11 Edil & Alanazy (1992)*
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