CRITICAL STATE BEHAVIOUR OF AN AGRICULTURAL SOIL

A Thesis submitted to the College of Graduate Studies and Research
in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy
in the Department of Agricultural and Bioresource Engineering
University of Saskatchewan

by

Bankole A. Adams

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ABSTRACT

Soils bear natural and artificial structures and sustain agricultural and biological activities. Understanding soil behaviour is the key to appropriating the benefits derivable from this material. Engineers are usually interested in the mechanical aspect of soil behaviour. This involves the study of volume change and shear response to applied stresses. In the past, soil shear and volume change behaviour were studied independently. But many practical problems involve shearing accompanied by volume change, therefore, it is desirable to consider this interaction in modelling soil behaviour. The critical state theory (Roscoe *et al.* 1958) was developed to combine both shear and volume change behaviour of soils.

Early soil mechanics theories including the critical state theory, were developed for saturated soils. Because unsaturated soils are encountered in many situations, there is the need to adapt earlier theories or to formulate new ones for unsaturated soils. The major differences in the physical behaviour of saturated and unsaturated soils are due to an internal stress called matric suction formed at the air-water interface in the soil pores. Theories developed for unsaturated soil mechanics are based on the inclusion of the effect of matric suction on soil behaviour.

In this study, the applicability of two major frameworks (i.e., critical state theory and unsaturated soil mechanics) to the study of agricultural soil behaviour was investigated. Soil behaviour was considered within a modified critical state framework that permits the application of critical state theories to an unsaturated soil.

An experimental program involving tests in a modified state-of-the-art triaxial testing system was undertaken. Critical state soil parameters were obtained and state boundaries identified. The role of matric suction and soil structure on state parameters are presented. Changes in matric suction caused a shift (translation) in state boundaries, while changes in structure caused a rotation of the state boundaries. Results also showed that the assumption that matric suction is a constant variable depends on the stress level in the soil. Under low stress regimes, this assumption seems justifiable, however this is questionable under high stress regimes. This study also showed the importance of consideration of suction in the analysis of agricultural soil behaviour.

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DEDICATION

To God the Father Almighty without whom this would not be possible.

To my Lord Jesus Christ who taught me to say at all times; “It is well with my soul”.

To my Dad and Mum for instilling the value of education.

To my wife Adenike and my daughter Toluwa for their love, loyalty and patience.

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LIST OF SYMBOLS

c	total cohesion of soil (kPa)
c'	effective cohesion of soil (kPa)
e	void ratio
e_{pc}	void ratio at preconsolidation pressure
D_c	diameter of triaxial specimen after isotropic compression (m)
G_s	specific gravity of soil particles
I_p	plasticity index
L_c	length of triaxial specimen after isotropic compression (m)
p	mean stress (kPa)
p'	effective mean stress (saturated) or mean net stress (unsaturated soil) (kPa)
p_a	atmospheric pressure (kPa)
p_o	preconsolidation or precompression stress (kPa)
p_o^*	preconsolidation or precompression stress for saturated condition (kPa)
p^c	reference stress for which $v = N$ or $v = \Gamma$ (kPa)
p_s	intercept of the critical state line with p' - axis at matric suction s (kPa)
q	deviatoric stress (kPa)
q_o	intercept of the critical state line on q - axis (kPa)
R_s	radius of curvature of contractile skin (m)
s	($= u_a - u_w$) matric suction (kPa)
S_r	degree of saturation (%)
T_s	surface tension (kN)
u_a	pore-air pressure (kPa)
u_w	pore-water pressure (kPa)
v	($= 1 + e$) specific volume
v_w	specific water volume
v_κ	intersect of the unloading rebound line at $p' = 1$ (or at reference pressure)
V_o	original volume (m ³)

ΔV	change in total volume (m^3)
ΔV_a	change in volume of air (m^3)
ΔV_w	change in volume of water (m^3)
w	gravimetric water content (dry basis) (%)
w_L	liquid limit (%)
χ	material parameter related to degree of saturation
ε_v	volumetric strain (%)
ϕ	angle of internal friction (degrees)
ϕ'	effective angle of internal friction (degrees)
ϕ^b	parameter quantifying the variation of shear strength with matric suction (degrees)
ϕ'_{cs}	soil angle of internal friction at critical state (degrees)
Γ	intercept of the critical state line at $p' = 1$ or at reference pressure
Γ_{aw}	specific volume when $(\sigma - u_a)$ and $(u_a - u_w)$ equal unity
κ	slope of the unloading-reloading line on $v - \ln p'$ plane (kPa^{-1})
λ	slope of the normal compression line on $v - \ln p'$ plane (kPa^{-1})
λ_{cs}	slope of the critical state line on $v - \ln p'$ plane (kPa^{-1})
λ^*	($= \lambda_{cs}$) slope of the critical state line on $v - \ln p'$ plane (kPa^{-1})
λ_a	slope of normal compression line with respect to total stress (kPa^{-1})
λ_w	slope of normal compression line with respect to suction (kPa^{-1})
$\mu(s)$	($= q_o$) intercept of the critical state line on q - axis (kPa)
M	slope of the critical state line on $q - p'$ plane
M_a	total stress ratio
M_w	suction ratio
N	intersect of the normal compression line at $p' = 1$ (or at reference pressure)
θ	($= w$) gravimetric water content (%)
σ	total normal stress (kPa)
σ'	effective normal stress (kPa)

σ_n	total normal stress on failure plane (kPa)
σ'_n	effective normal stress on failure plane (kPa)
$\sigma_x, \sigma_y, \sigma_z$	normal stress along x, y, z axis (kPa)
$\sigma_1, \sigma_2, \sigma_3$	principal normal stresses (kPa)
ρ	bulk density (Mg/m^3)
τ	shear stress along failure plane (kPa)
$\tau_{xy}, \tau_{yx}, \tau_{yz}$	shear stresses on the faces of a cubical soil element (kPa)

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CRITICAL STATE BEHAVIOUR OF AN AGRICULTURAL SOIL

Soils bear natural and artificial structures and sustain agricultural and biological activities. Understanding soil behaviour is the key to appropriating the benefits derivable from this entity. Of the many aspects of soil behaviour, engineers are usually interested in the mechanical response. This involves the study of volume change and shear response to applied stresses. In the past, soil shear and volume change behaviour were studied independently. But many practical problems involve shearing accompanied by volume change hence it is desirable to consider this interaction in modelling soil behaviour. A conceptual model (the critical state theory, Roscoe *et al.* 1958) was developed to combine both shear and volume change behaviour of saturated soils.

The earliest soil mechanics theories were developed for saturated soils. Because unsaturated soils are encountered in many situations, there is the need to adapt earlier theories or to formulate new ones for unsaturated soils. The major differences in the physical behaviour of saturated and unsaturated soils is due to an internal stress called matric suction formed at the air-water interface in the soil pores. Theories developed for unsaturated soil mechanics are based on the inclusion of the effect of matric suction on soil behaviour.

In this study, the applicability of two major frameworks (i.e., critical state theory and unsaturated soil mechanics) to the study of agricultural soil behaviour was investigated. Soil behaviour was considered within a modified critical state framework that permits the application of critical state theories to an unsaturated soil.

An experimental program involving tests in a modified state-of-the-art triaxial testing system was undertaken. Critical state soil parameters were obtained and state boundaries identified. The role of matric suction and soil structure on state parameters are presented. Change in matric suction caused a shift (translation) in state boundaries while change in structure caused a rotation of state boundaries. Results also show that the assumption of matric suction as a constant

variable (i.e., using total stress analysis) depends on the stress level in the soil. Under low stress regimes, total stress analysis is justifiable, however this approach is questionable under high stress regimes.

CHAPTER ONE

INTRODUCTION

The dependence of humans directly or indirectly on soils has made the quest for knowledge on this material important. Soils bear natural and artificial structures and sustain crop growth. The topsoil is a part of the earth used for the cultivation of arable lands. The topsoil is referred to as agricultural soil in this study. The proximity to the surface renders some agricultural soils unsaturated. This factor must be taken into consideration in any attempt to study its behaviour. Apart from the presence of a large quantity of air in the pores, agricultural soils are also generally characterized by low initial bulk densities and large deformations when subjected to mechanical disturbances by traction members or tillage tools.

The inevitable interactions between foreign bodies and the soil medium leads to some problems because of the difference in properties and behaviour of the soil and the foreign body. For example, during the passage of an agricultural vehicle on the soil surface the tires or tracks load the soil and the soil responds by deforming. This deformation alters the pore space in the soil and has a direct impact on the physical fertility of the soil. Hence, it becomes important to investigate the implication of the presence of foreign

bodies on arable lands. Agricultural engineers have studied soil-machine interactions such as the effect of compaction on agricultural soils and draft requirements of tillage tools.

It is essential to acquire more knowledge of soil behaviour to procure solutions to these problems. Attempts at this have involved laboratory and in-situ tests. Several empirical models have been proposed. With the advent of the computer age, numerical analysis has served as a powerful tool in the simulation of soil behaviour and the formulation of constitutive relations.

Several simple and complex soil models have emerged from the study of soil behaviour. These models are all based on the same fundamental theories of soil behaviour. The mechanical behaviour (i.e., strength and volume change) of soils is of particular interest to soil scientists and engineers.

1.1 Mechanical Behaviour of Soils

Soil is a mixture of several materials and while some aspects of its behaviour are intriguing and simple, in many ways soil behaviour is complex. Machines, plants and the environment all impart some stress on the soil. The soil pore space and skeleton in turn respond to these external stresses such that a state of equilibrium is re-established in the medium.

Numerous studies have been directed to the behaviour of soil under stress. Significant findings on soil behaviour date as far back as the early works of Coulomb in 1776 on the shear strength of soils, Rankine on earth pressure theories in 1860, and Terzaghi's effective stress concept in 1936. These early works set the foundation for

subsequent studies of soils in the diverse fields of soil science, agricultural engineering and civil engineering. Though the basic soil mechanics principles are similar, the applications are not necessarily the same in these disciplines. For example, while civil engineers are generally interested in long time response of sub-layered-soils under stress, the agricultural engineer focuses more on the behaviour of arable topsoil under short term loading by tillage tools or traction members (e.g., tires, tracks and wheels).

The mechanical response of soil to external stress has been likened to the elastic and plastic behaviour of metals. For example, some soils under a structure or temporary load (within the elastic limit) may be modeled after elastic behaviour, while most agricultural soils (characterized by large deformations under field conditions) tend to exhibit predominantly plastic behaviour, but a combined elasto-plastic behaviour is also possible.

A soil body under stress generally undergoes volume change by deforming in response to the applied stress. The immediate result is compaction or consolidation, or even dilation, depending on the nature of the soil and loading. When soil is loaded beyond its strength, failure occurs. It is now known that at some point during the loading process the soil reaches a state at which it continues to deform without further increases in stress. This behavioural pattern forms the basis of the critical state theory.

1.2 The Critical State Theory

A series of events takes place as the soil is loaded to failure. The external stress could cause compression and shear failure with resulting volumetric and shear strains. Shear failure is often accompanied by compaction or dilation of the soil. Stress and volume changes occur concurrently in many practical field problems in response to an external stress.

While soil compression and shear behaviour have been studied separately in classical soil mechanics, the critical state theory permits the study of both under a unified framework. The theory emerged from the soil mechanics group at Cambridge University under the pioneering work of Roscoe *et al.* (1958) and was originally formulated from the observed behaviour of saturated clay soil under loading.

This approach, which is characterized by simplified stress-strain (constitutive) relationships, provides a better insight into soil behaviour than classical theories. For example, the complete theory of stress-strain analysis of soil requires a relationship between six components of strain and associated stress (i.e., normal stresses and strains in three dimensions) while the critical state approach has reduced these stresses to just two by making certain simplifying assumptions and by choice of state variables. These two stresses, called spherical and deviatoric stresses, cause volumetric and distortion strains respectively. Hence, the basic critical state relationships for a soil now involve only the deviatoric stress, spherical stress and the volumetric strain. For unsaturated soils, soil suction becomes an additional variable.

The critical state theory has been applied in various areas of soil research and experimental results are still emerging. This theory has evolved since its formulation by Roscoe *et al.* (1958). The earliest critical state model described by Roscoe *et al.* (1958) was the Cam clay model which has since been modified. A recent and major modification of the critical state theory is the formulation of models for unsaturated soils (Toll 1990). The modified Cam clay model has been further modified such that it is appropriate for unsaturated soils (Toll 1990; Wheeler and Sivakumar 1993; Petersen 1994).

In recent years, several laboratory studies have been undertaken to study the critical state behaviour of saturated and unsaturated soils within the original and modified frameworks. (Toll 1990; Wheeler and Sivakumar 1993; Petersen 1994). The critical state models involve a series of parameters most of which are experimentally determined in triaxial tests. There also exist empirical models to predict some of these parameters from standard soil consistency limits.

1.3 Effective Stress Concept and Unsaturated Soil Stress State Variables

While saturated and unsaturated soils may respond in a similar manner to external stress in some situations, the behaviour could also be significantly different. It is generally not appropriate to equate the behaviour of saturated soils to that of unsaturated soils. Terzaghi (1936) observed the behaviour of a saturated soil experiencing an external stress. If the soil is loaded such that water cannot escape from its pore space, the pore-water pressure (u_w), rises as it first supports the load. The total stress, σ , imposed on this soil is

borne by the soil skeleton and pore-water. The effective stress, σ' , is the component of stress acting on the soil skeleton and is expressed as:

$$\sigma' = \sigma - u_w \quad (1.1)$$

This expression has been described as the stress state variable for saturated soils (Fredlund and Rahardjo 1993a).

Unsaturated soils are characterized by the existence of negative pore-water pressure and three phases, namely solid, air and water. The presence of a fourth phase, the air-water interphase, called the “contractile skin” was postulated by Fredlund and Morgenstern (1977).

The presence of air and air-water interfaces has always posed major challenges in the study of unsaturated soils, one of which is the determination of stress state variables. Several pioneering studies, such as that of Bishop (1959), were directed at quantifying a single effective stress in an unsaturated soil. The concept of two independent stress state variables was proposed by Fredlund and Morgenstern (1977). These independent stress state variables are becoming increasingly accepted in the solution of practical problems.

Many researchers have used a total stress analysis in the application of the critical state theory to unsaturated agricultural soils. One reason often given is that this approach is convenient from an experimental viewpoint. But this approach does not address the independent nature of the effects of total stress and soil suction on unsaturated soil behaviour. Based on fundamental principles of unsaturated soil mechanics and experimental evidence (Fredlund and Morgenstern 1977; Fredlund and Rahardjo 1993a), it is no longer appropriate to define a single effective stress for unsaturated soils. In this study, independent stress state variables are used to quantify critical state variables.

1.4 Research Objectives

As an overview of this research effort, it is purposed to apply recently developed theories in unsaturated soil mechanics and the Cambridge critical state theory to the study of an agricultural soil. Hence, the major objective of this research is to investigate the applicability of the critical state theory to the mechanical behaviour of agricultural soils within an unsaturated soil mechanics framework. This research focuses mainly on the development of a model for agricultural soil compaction. The use of critical state concepts for agricultural soils is a further attempt at finding appropriate stress-strain models for agricultural problems.

Though many agricultural problems are dynamic in nature, modelling of soil behaviour, especially using triaxial tests is carried out under near-static conditions in order to allow the equalization of stresses within a test specimen. The approach in this study is to test the specimens at practical field drainage conditions, (i.e., constant gravimetric water content and constant suction), while using test speeds slow enough to allow the dissipation or equalization of stresses in the specimens.

In line with the major objective, the following are the targeted aims:

- (i) to establish an unsaturated soil testing program to investigate the critical state behaviour of agricultural soils using a modified triaxial apparatus,
- (ii) to obtain critical state parameters of the soil from triaxial isotropic compression and shear tests for saturated and unsaturated specimens of an agricultural soil,
- (iii) to experimentally establish critical state boundaries for an agricultural soil,

- (iv) to observe the effect of matric suction and soil micro-structure on critical state parameters, and
- (v) to investigate the uniqueness of the critical state line for agricultural loading situations.

A strong base for the application of the unsaturated and critical state soil mechanics framework to the study of agricultural soils will be established upon achievement of these aims. This base will assist in formulating appropriate constitutive or empirical relationships for agricultural soils.

CHAPTER 2

LITERATURE REVIEW

Overview

Fundamental theories of soil mechanics have been used in the past to explain or predict soil behaviour. The improvement and expansion of these theories over the years has resulted in more advanced models that can be applied to a broader range of soil problems.

In this chapter, the fundamentals of soil behaviour are reviewed with emphasis on unsaturated soils. The shear strength behaviour and the importance of soil-water characteristics of unsaturated soils are highlighted.

The essence of the critical state theory is also described, along with the development of constitutive relations. A formulation of the critical state theory within an unsaturated soil mechanics framework is presented. The relevance of the critical state theory to agricultural soils is discussed based on a review of earlier studies.

Since a substantial part of this study was conducted using a modified triaxial apparatus, the remainder of the chapter is devoted to a review of the development, improvement and modifications of the triaxial apparatus and testing procedures for unsaturated soils.

2.1 Fundamentals of Unsaturated Soil Behaviour

2.1.1 *State of Stress in Soil*

A conventional way of presenting the complete state of stress in the soil is to analyze the stresses acting on a cubical element as shown in Figure 2.1. This is an infinitesimal cube which represents a mere point when the whole soil body is considered. The stresses existing on the six sides of the cubical element can be analyzed within a three-dimensional orthogonal coordinate system.

Body and surface forces act on each side of the cube. All the forces acting per unit area of a side can be resolved into a normal stress perpendicular to the side and two shear components tangential to the side.

Several sides as the one analyzed exist planes throughout the soil body. Hence the complete stress state of the soil can be represented as a tensor of nine components, six of which represent shear stresses, and three which represent normal stresses.

$$\begin{pmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{xz} & \sigma_z \end{pmatrix}$$

As a condition of equilibrium, components of the shear and normal stresses on opposite faces of the cube are equal in magnitude but act in opposite directions. The summation of moments on the element must be equal to zero. Therefore,

$$\tau_{xy} = \tau_{yx} \quad \tau_{yz} = \tau_{zy} \quad \tau_{xz} = \tau_{zx} \quad (2.1)$$

Figure 2.1 Three-dimensional normal and shear stresses acting on a cubical soil element.

This means that the number of components of the stress tensor is reduced from nine to six. Planes exist in the soil body for which there are no shear stresses. The stresses acting normal to these planes are called principal normal stresses. The stress tensor can thus be reduced further to a three-component tensor made up of three mutually perpendicular stresses termed, major, intermediate and minor principal stresses (σ_1 , σ_2 and σ_3 respectively):

$$\begin{pmatrix} \sigma_1 & 0 & 0 \\ 0 & \sigma_2 & 0 \\ 0 & 0 & \sigma_3 \end{pmatrix}$$

2.1.2 Air-Water Interface in Unsaturated Soils

Unsaturated soils are encountered in many engineering problems. The presence of air in the pores of unsaturated soils leads to the formation of air-water interfaces. The action of the interface is responsible to a large extent for the difference in the behaviour of saturated and unsaturated soils. The interface exerts a tensile pull characteristic of the surface tension phenomenon in a capillary tube. Also termed the contractile skin (or air-water interphase), its elastic action tends to pull soil particles closer together as the soil dries and pore-water pressure goes more negative.

In unsaturated soils, the contractile skin feels an air pressure u_a greater than pore-water pressure u_w . The pressure difference, $(u_a - u_w)$, is termed matric suction. This suction is related to surface tension by Kelvin's model:

$$(u_a - u_w) = \frac{2T_s}{R_s} \quad (2.2)$$

where,

T_s = surface tension

R_s = radius of curvature of contractile skin

u_a = pore-air pressure

u_w = pore-water pressure

More details of the contractile skin and the four-phase nature of unsaturated soil are discussed by Fredlund and Rahardjo (1993a). The existence of a matric suction introduces an additional stress in an unsaturated soil. This affects the mechanical behaviour as well as the agronomic quality of the soil.

2.1.3 Stress State Variables for Unsaturated Soil

The effective stress concept has been used to define a single stress state variable for saturated soils. The presence of air means that this concept can no longer be applied for an unsaturated soil.

Several attempts have been made to address this problem and to determine appropriate stress state variables for an unsaturated soil. These efforts have involved the use of a single valued stress variable or the isolation and use of independent stress state variables.

From Imperial College, London, Bishop (1959) proposed a single valued expression for the effective stress for unsaturated soils as:

$$\sigma' = (\sigma - u_a) + \chi (u_a - u_w) \quad (2.3)$$

where,

χ = dimensionless parameter related to degree of saturation (χ approaches 1 for a saturated soil and zero for a dry soil).

Other studies at Imperial College were carried out in an attempt to justify the use of equation (2.3). Bishop *et al.* (1960) described the variation of parameter χ with the degree of saturation. Bishop and Donald (1961) carried out a series of triaxial tests to investigate equation (2.3). These studies presented evidence to validate equation (2.3).

The use of equation (2.3) was criticized by Jennings and Burland (1962) because it did not provide satisfactory relationships for many soils. Blight (1967) also pointed out the difficulty in evaluating χ . Coleman (1962) and Burland (1965), Matyas and Radhakrishna (1968) and Barden *et al.* (1969) all suggested the use of independent stress state variables to explain the mechanical behaviour of unsaturated soils.

Fredlund (1973) experimentally tested the use of independent stress state variables and investigated the relationships between stress state and deformation state variables for unsaturated soils. Fredlund and Morgenstern (1977) formally presented a theoretical stress analysis of an unsaturated soil. The authors proposed various combinations of stress state variables $(\sigma - u_a)$, $(u_a - u_w)$ and $(\sigma - u_w)$. Three possible combinations of these stress variables are:

$$(\sigma - u_a) \text{ and } (u_a - u_w)$$

$$(\sigma - u_w) \text{ and } (u_a - u_w)$$

$$(\sigma - u_a) \text{ and } (\sigma - u_w) \quad (2.4)$$

The use of independent stress state variables have been embraced and applied in recent studies of the mechanical behaviour of unsaturated soils. The first pair of the combinations is most commonly used for practical Engineering problems (Fredlund, 1979). These two variables are called:

$$(\sigma - u_a) \quad \text{net total stress}$$

$$(u_a - u_w) \quad \text{matric suction}$$

2.1.4 Elastic and Plastic Behaviour

In some ways, the behaviour of soil under loading is similar to that shown by metals, plastics and other engineering materials. Under the influence of external stresses, a material undergoes deformation. Over a certain range, a linear stress-strain relationship is defined, and if the stress is removed within this range, the material recovers its original shape. When strains are totally recovered on the removal of the external stress, the material is said to exhibit elastic behaviour. At some point the material reaches a deformation level at which the original shape will be permanently distorted. A material under this condition is experiencing plastic deformation. Figure 2.2 shows typical stress-strain behaviour of materials. Some materials are known to behave in a perfectly elastic or perfectly plastic manner while other materials combine elastic and plastic behaviour in various forms. Soils generally tend to exhibit elasto-plastic behaviour.

Figure 2.2 Typical stress-strain behaviour of materials (a) rigid, perfect plastic (b) linear elastic, perfectly plastic (c) rigid, work hardening (d) linear elastic, work hardening (e) rigid, work hardening, perfectly plastic (f) linear elastic, work hardening, perfect plastic (Kurtay and Reece 1970).

Figure 2.3 shows typical stress-strain relationships for soils. Assume that the soil is loaded in stages from points $O \rightarrow D$. The soil behaves elastically from $O \rightarrow A$, point A being the yield point beyond which strains become only partially irrecoverable. As the soil is loaded further, it undergoes plastic deformation up to a new yield point at B. It eventually fails at C when the applied stress is greater than the soil strength.

Two patterns of failure are commonly seen in soils. Brittle soils generally fail along distinct planes (Figure 2.3a). There will be an identifiable peak in the stress-strain graph and the slip angle is measurable in a typical triaxial soil specimen. Ductile soils, on the other hand, show no peak in the stress-strain curve, nor a distinct failure plane (Figure 2.3b). Failure for this kind of soil must be carefully defined. A cylindrical specimen in this case deforms by barreling.

If the soil is continuously loaded beyond point C (Figure 2.3), an ultimate state is reached at D where no further changes take place in the soil strength. From $C \rightarrow D$, the shear strength of the brittle soil drops down to a residual value at D as it fails along the shear plane. The ductile soil goes from $C \rightarrow D$ without further change in strength, as the soil begins to “flow”. Most strains at this stage are plastic (i.e., mostly irrecoverable).

Generally, the onset of plastic strains is an indication of the development of a new yield state. For example, for the one dimensional case considered in Figure 2.3, B is a new yield point. In three dimensional cases, plastic strains are associated with a yield surface. If the state of a material under applied stress lies beneath this surface, its deformation is described as elastic. On reaching the yield surface, plastic deformation is imminent and the

(a) brittle failure

(b) ductile failure

Figure 2.3 Typical stress-strain relationships and failure patterns for soils.

material under further stress, traverses (or flows) along the surface.

The observations described above are the essence of elasto-plastic and plastic theories of soil behaviour on which the critical state concepts are based. The critical state concepts will be described in section 2.2.

2.1.5 Shear Strength Behaviour

Soil can be usually subjected to a combination of normal and shear stresses and its strength increases up to a point where it can no longer withstand shear stresses. When maximum shear stresses have been mobilized, soil fails along a distinct or multiple planes. An expression for the maximum shear strength in classical soil mechanics for saturated situation is the Mohr-Coulomb failure criterion:

$$\tau = c' + \sigma'_n \tan \phi' \quad (2.5)$$

where,

σ'_n = $\sigma - u_w$ = effective normal stress on failure plane

c' = effective cohesion

τ = shear stress along failure plane

ϕ' = effective soil angle of internal friction.

The shear strength equation for unsaturated soils was formulated by Fredlund *et al.* (1978). In a three-dimensional space, the effects of net normal stress and matric suction on shear strength were isolated to give:

$$\tau = c' + (\sigma_n - u_a)_f \tan \phi' + (u_a - u_w)_f \tan \phi^b \quad (2.6)$$

where,

$(\sigma_n - u_a)_f$ = net normal stress on failure plane

ϕ' = angle of internal friction related to the net normal stress $(\sigma_n - u_a)_f$

$(u_a - u_w)_f$ = matric suction on failure plane

ϕ^b = angle relating rate of change of shear strength with matric suction.

Equation 2.6 can also be expressed as;

$$\tau = c + (\sigma_n - u_a) \tan \phi' \quad (2.7)$$

where,

$$c = c' + (u_a - u_w)_f \tan \phi^b \quad (2.8)$$

c = total cohesion.

While a linear relationship (governed by ϕ') exists between the normal stress and shear strength, the increase in shear strength due to matric suction has been shown to be non-linear and is controlled by the parameter, ϕ^b (Fredlund *et al.* 1987). Equation (2.7) is an extension of the Mohr-Coulomb failure criterion and is graphically represented in Figure 2.4. Equation (2.7) reduces to the form in equation (2.5) as the soil becomes saturated. When u_a becomes equal to u_w at saturation, there is a smooth transition from unsaturated to saturated behaviour.

Though equation (2.6) and Figure 2.4 show a linear relationship between shear strength and matric suction, experimental evidence (Fredlund *et al.* 1987) and other

Figure 2.4 Extended Mohr-Coulomb failure envelope for unsaturated soils (Fredlund and Rahardjo 1993a)

studies have shown that shear strength of unsaturated soils increases non-linearly with an increase in matric suction (i.e., the parameter ϕ^b in equation (2.6) and (2.8) varies). With an increase in matric suction, shear strength increases at first in a linear manner up to a matric suction value above which increase in strength becomes non-linear (Figure 2.5). At higher matric suction, the strength may remain constant or even drop (Fredlund and Rahardjo 1993b).

While matric suction has been shown to affect soil strength, it is also known to affect the behaviour of unsaturated soils in failure. Cui and Delage (1993) reported that high matric suction changed the mode of failure of triaxial test specimens from ductile to brittle (Figure 2.6).

2.1.6 Soil-Water Characteristic Curve

The soil-water characteristic or water-retention curve (to be used interchangeably) relates matric suction (or water potential) of an unsaturated soil with the water content or degree of saturation. This relationship is strongly influenced by some factors which are dependent on the soil macro- or micropores. The water-retention curve is one of the significant physical properties of unsaturated soil and is influenced by the type of minerals in the soil, the void-size distribution and structure (Warkentin 1971). Rendig and Taylor (1989) described how soil texture also affects the soil-water characteristic. These studies show that clay soils tend to retain more water in comparison to sandy soils.

Figure 2.5 Non-linear variation of shear strength with matric suction (Fredlund and Rahardjo 1993b).

Figure 2.6 Effect of matric suction on failure pattern of an unsaturated soil (Cui and Delage 1993).

Some external factors are known to affect the soil-water characteristics. Stresses imposed by the environment and external mechanical members alter soil properties and behaviour. While environmental factors (e.g., wetting and drying) have agronomic implications, loading by external bodies have a direct bearing on the mechanical behaviour of the soil.

The wetting and drying water characteristic curves are usually different due to hysteresis. Hence the water content of a field soil at a particular matric suction will also depend on the history of wetting and drying. Figure 2.7 shows typical soil-water characteristics and the hysteresis phenomenon.

Soil compaction is another external factor that has been shown to affect the soil-water characteristic relations in agricultural soils. Taylor and Box (1961), Box and Taylor (1962), Larson and Gupta (1980), Reicosky *et al.* (1981), and Horton *et al.* (1994) amongst others, observed the effect of soil compaction on the soil-water characteristic. From these studies, it is generally accepted that compaction sets off a chronology of events, the first of which is the increase in the bulk density. A dense soil tends to retain more water in comparison to a loose one because of more closely packed pore spaces. Increase in bulk density alters the structure of the soil, and hence properties that depend on the soil structure, such as hydraulic conductivity and permeability, are in turn affected.

From compaction tests on some agricultural soils, Larson and Gupta (1980) showed that as applied stress increases, pore-water pressure became more negative down to a minimum value after which it increased. Figure 2.8 shows that drier soils tend to approach this minimum value at a greater rate than the wetter ones. The negative pore-water pressure in the wetter specimens remained fairly constant with applied stress and

Figure 2.7 Water characteristics curves showing hysteresis (adapted from Rendig and Taylor 1989).

Figure 2.8 Variation of bulk density and pore-water pressure with applied stresses for two agricultural soils (adapted from Larson and Gupta 1980).

then increased. This phenomenon has been attributed to structural changes in the soil. It was also suggested that the stress corresponding to the minimum value of pore-water pressure should be the maximum that should be applied to a soil during cultivation or passage by traction members.

Water-retention curves have been used traditionally in agronomy to monitor the availability of free water for plant roots and to estimate pore size distribution. Numerous works have shown that soil-water characteristics can also be used to estimate other soil properties or relationships. Soil hydraulic conductivity has also been established from the soil-water characteristic curve by many researchers (e.g., Reicosky *et al.* 1981; Fredlund and Rahardjo 1993a). Fredlund *et al.* (1995) described the use of the soil-water characteristic curve in the prediction of a shear strength function for unsaturated soils.

2.2 Critical State Concepts for Saturated Soils

The development of critical state concept and its fundamental theories for saturated soils are discussed in this section. To apply the critical state theory to unsaturated soils, some modifications must be made, these will be discussed in later sections.

2.2.1 *History and Development*

A conceptual model of soil behaviour assumes that soil under stress ultimately reaches a state of plastic behaviour characterized by continuous deformation without further increases in stress. This concept is the underlying principle of the critical state theory. The critical state framework also idealizes a state boundary surface which defines the limit of possible states at which a soil can exist.

The critical state theory was introduced by Roscoe *et al.* (1958) as a powerful framework for explaining shear and volume change behaviour of saturated soils under external loading. The theory provided for the first time, soil models that combine stresses and the accompanying deformation in a three-dimensional space.

The concepts of yielding, hardening, normality and plastic potentials in plasticity theories can be applied to the development of critical state models if the state boundary surface is considered to be a yield surface. The first elasto-plastic critical state model, called Cam clay, was described by Schofield and Wroth (1968). A second model called the Modified Cam clay model was described by Roscoe and Burland (1968).

2.2.2 Basic Assumptions in Critical State Theory

The exercise of model development becomes difficult if the complex nature of soil and every aspect of its stress-strain behaviour are to be considered. Some assumptions can be made to simplify the task, without compromising the usefulness of the model.

Some assumptions for the formulation of constitutive relations for soils, based on the critical state theory were outlined by Kurtay and Reece (1970). Based on recent development in the study of unsaturated soils, modified forms of these assumptions are:

- (i) Soil material is homogeneous and isotropic.
- (ii) The mechanical behaviour of soil can be expressed in terms of the appropriate stress state variables (i.e., the effective stress for a saturated soil, and the independent stress state variables for an unsaturated soil).
- (iii) The mechanical behaviour of soil can be described by a macroscopic, continuum mechanics model. It is not necessary to relate behaviour to the properties of, and interactions between, individual particles.
- (iv) There are no time dependent aspects of mechanical behaviour. The soil is not viscous.

These assumptions are referred to in model formulation and soil preparation methods in this study. For example, in laboratory experiments aimed at investigating the critical state concept, there is the need to ensure the formation of homogenous specimens in view of the first assumption.

2.2.3 Definition and Fundamental Concepts

Fundamental concepts of the critical state theory and some applications for saturated soils, have been presented by Schofield and Wroth (1968), Atkinson and Bransby (1978), Atkinson (1981), Wood (1990) and Atkinson (1993). Critical state is one of the many states in which a soil specimen can exist. It is a failure state at which the specimen under stress continues to deform with no further change in volume or stress.

With the assumption that the soil is isotropic, the original and standard critical state variables based on a macroscopic viewpoint, are, p' , q and v and are defined as:

$$p' = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} - u_w \quad (2.9)$$

$$= p - u_w$$

$$q = \sigma_1 - \sigma_3 \quad (2.10)$$

$$v = 1 + e \quad (2.11)$$

where,

p' = effective mean or hydrostatic stress

p = total mean stress

q = deviatoric stress

v = specific volume

e = void ratio

$\sigma_1, \sigma_2, \sigma_3$ = principal stresses.

For triaxial conditions, where σ_2 is equal to σ_3 , and equation 2.9 becomes:

$$p' = \frac{\sigma_1 + 2\sigma_3}{3} - u_w \quad (2.12)$$

Critical state is defined by a unique line in the stress-specific volume space ($p' - q - v$). This three-dimensional space is an amalgamation of two two-dimensional planes. There are no shear stresses on the $v - p'$ plane, while no volume changes occur on the $q - p'$ plane. The complete state boundary (Figure 2.9) is located within the space created by these two planes.

Critical state is characterized by large shear distortion and no changes in stress and specific volume. There are two attributes of significance for a soil that is approaching critical state: the stress path taken to critical state, and the inherent volume change as critical state is approached.

Figure 2.9 Complete state boundary surface (Atkinson 1981)

It has been shown that attainment of critical state is independent of stress path. Atkinson and Bransby (1978) analyzed data from Parry (1960) and showed that triaxial compression of normally consolidated saturated weald clay approached a unique critical state line for both drained and undrained stress paths. This observation (shown in Figure 2.10), along with analyses from other studies showed the independence of the critical state line of stress paths for saturated soils. Though critical state is independent of stress path, the nature of volume changes that take place are influenced by the soil stress history. Normally consolidated soils generally tend to reach critical state with a decrease in volume in a ductile manner. Brittle or overconsolidated soils on the other hand tend to dilate on approaching critical state. Figure 2.11 shows the basic forms of soil behaviour predicted by critical state theory.

2.2.4 State Boundary Surface for Saturated Soils

It has been postulated that much of the behaviour of a specimen is defined if its state in the (p' - q - v) space is known (i.e., soil behaviour depends on its mean stress, the shear stress imposed, and the physical arrangement of its particles). The critical state boundary surface for a soil (Figure 2.9) is bounded by lines that define the normal consolidation line (ncl), a unique critical state line (csl), tension cut off and the v - axis.

Generally, soils within the state boundary surface undergo elastic and recoverable strains. State paths cannot cross the state boundary surface, hence soils cannot exist above the surface. However state paths can traverse any of the surfaces; soils in this condition undergo volume changes that are plastic, irrecoverable and permanent. The end point of a

Figure 2.10 Critical state line defined by drained and undrained tests on normally consolidated saturated clay (data from Parry 1960, plotted by Atkinson and Bransby 1978).

Figure 2.11 Basic forms of soil behaviour predicted by the critical state theory (Hettiaratchi 1987).

state path traversing the surface is the critical state line where it continues to deform or flow without any further change in p' , q or v . The state boundary surface is made up of three distinct surfaces, namely, Roscoe, Hvorslev and no-tension surfaces.

The Roscoe surface is bounded by the isotropic normal consolidation GH in the $v - p'$ plane ($q = 0$) and the critical state line AB (Figure 2.9). For large specific volumes, (i.e., large void ratio or loose soil), and low mean stress, the soil is soft and weak, thus it can only sustain a small deviatoric stress, q , before it fails or reaches the critical state line AB. If this same sample is compressed to H in isotropic compression, it exists at a much lower v , hence it is stronger and will sustain greater shear before failing. Thus, the critical state line rises as the soil becomes denser.

The Hvorslev surface (planar and gentle slope) is bounded by CD and AB. It represents the state boundary surface for overconsolidated or dilative soils. When a soil in a state lying to the origin side of the critical state line is subjected to normal and shearing stresses, its path will rise to intersect the Hvorslev surface, after which it will traverse this surface until it reaches the critical state line.

The no-tension surface is inclined (3:1 slope) and is bounded by CD and EF along the v -axis. The surface indicates the condition of zero tensile stress in soils.

State paths traversing the Roscoe surface are associated with a decrease in volume (compaction), while an increase in volume (loosening or dilation) characterizes the state paths on the Hvorslev surface. A critical state wall is defined as the plane of projection of the critical state line vertically downward to the $v - p'$ plane. The wall divides the state

space into two volumetric zones referred to by Hettiaratchi (1987) as subcritical and supercritical zones, respectively. Failure associated with these zones is matched as:

subcritical	(usually normally or lightly overconsolidated soils),	ductile failure
supercritical	(typically heavily overconsolidated soils),	brittle failure

2.2.5 Critical State Models and Parameters for Saturated Soils

A convenient way to represent the critical state condition is by projecting the state boundaries on to the $v - p'$ and $q - p'$ planes as shown in Figure 2.12.

The equations from this representation are:

- (i) Isotropic normal consolidation line on $v - p'$ plane

$$v = N - \lambda \ln p' \quad (2.13)$$

$$q = 0 \quad (2.14)$$

- (ii) rebound (overconsolidation) line on $v - p'$ plane

$$v = v_{\kappa} - \kappa \ln p' \quad (2.15)$$

$$q = 0$$

- (iii) critical state line

$$v = \Gamma - \lambda_{cs} \ln p' \quad \text{on } v - p' \text{ plane} \quad (2.16)$$

$$q = Mp' \quad \text{on } q - p' \text{ plane} \quad (2.17)$$

where,

λ = slope of normal consolidation line on the $v - p'$ plane

λ_{cs} = slope of critical state line on the $v - p'$ plane

κ = slope of the rebound line on the $v - p'$ plane

M = slope of the critical state line on the on the $q - p'$ plane

Figure 2.12 Projection of ideal critical state boundaries on the $v - p'$ and $q - p'$ planes (Adapted from Atkinson and Bransby 1978). (ncl - normal compression line csl - critical state line url - unloading rebound line).

N = intercept of the normal consolidation line with the v - axis

v_{κ} = intercept of the rebound line with the v - axis

Γ = intercept of the critical state line with the v - axis.

For a saturated clay, Atkinson and Bransby (1978) observed that λ is equal to λ_{cs} .

The parameters λ , λ_{cs} , κ and M are soil constants that can be determined from laboratory experiments. The intercept parameters N , v_{κ} and Γ are generally obtained from the $v - \ln p'$ graph at a reference mean stress of 1 kPa. Selection of a reference mean stress of 1 kPa could however introduce errors in the evaluation of the intercept parameters, especially if p' equal to 1 kPa falls outside the range of experimental data. Wheeler and Sivakumar (1993) described a different form of equation 2.13 in order to minimize this error. The expression (equation 2.18) is also dimensionally consistent:

$$v = N - \lambda \ln \left(\frac{p'}{p_a} \right) \quad (2.18)$$

where,

p_a = atmospheric pressure (i.e., 101.3 kPa).

N = specific volume v , at $\left(\frac{p'}{p_a} \right) = 1$

The parameter λ (from three-dimensional isotropic consolidation), can be likened to the compression index C_c (from one-dimensional consolidation). Its magnitude determines the relative position of the critical state wall and hence the relative proportions of the Roscoe and Hvorslev surfaces. As λ decreases, the Roscoe surface expands while

the Hvorslev surface contracts, and hence the probability of subcritical (ductile) failure is higher. Chances of supercritical (brittle) failure are higher under the reverse conditions.

The frictional parameter M does not affect the proportions of the state boundary surface significantly and the general trend is for all the surfaces to increase in size with M .

The parameter M is related to the internal friction of the particles at critical state.

$$M = \frac{6 \sin \phi'_{cs}}{3 - \sin \phi'_{cs}} \quad (2.19)$$

where,

ϕ'_{cs} = effective angle of internal friction at critical state.

2.3 Application of Critical State Concepts to Unsaturated Soils

This section describes the modification and adaptation of the original critical state theories to unsaturated soil behaviour.

2.3.1 *Critical State Framework for Unsaturated Soils*

The application of critical state concepts to unsaturated soil behaviour is relatively new in soil mechanics, hence the dearth of literature in this area. Toll (1990) and Alonso *et al.* (1990) proposed the first forms of a critical state framework for unsaturated soil. Aspects of the framework proposed by Alonso *et al.* (1990) were experimentally verified by Wheeler and Sivakumar (1992, 1993) for an unsaturated clay soil.

The framework proposed by Toll (1990) is based on the critical state model for saturated soils but embraces factors relevant to unsaturated soil behaviour. The importance of soil fabric (i.e., structure formed for a particular water content and bulk

density combination) was particularly emphasized. Toll used the degree of saturation as an important parameter of soil state. While the experimental specimens had varying degrees of saturation, six were of values greater than 90%, these were considered to be saturated. A true critical state was not achieved in the unsaturated tests and the soil fabric (or structure) was found to play a very significant role.

Toll's (1990) framework was formulated based on independent stress state variables for unsaturated soils. The following five variables were involved:

q	deviatoric stress
$(\sigma - u_a)$	net total stress
$(u_a - u_w)$	matric suction
v	specific volume
S_r	degree of saturation

To separate the effects of mean normal stress and matric suction, Toll (1990) extended the original Cam clay model to include unsaturated soil behaviour. The following forms of equations for the state boundaries were proposed:

$$v = \Gamma_{aw} - \lambda_a \ln (p - u_a) - \lambda_w \ln (u_a - u_w) \quad (2.20)$$

$$q = M_a(p - u_a) + M_w(u_a - u_w) \quad (2.21)$$

where,

M_a = total stress ratio (variation of deviator stress with normal stress)

M_w = suction ratio (variation of deviator stress with matric suction)

λ_a = slope of normal consolidation line with respect the total stress

λ_w = slope of normal consolidation line with respect to suction

Γ_{aw} = specific volume when $(\sigma - u_a)$ and $(u_a - u_w)$ equal unity.

The parameters M_a , M_w , λ_a , and λ_w , are all dependent on the degree of saturation and soil fabric.

In discussions of Toll's (1990) paper (Toll 1991), some concerns were raised. First, based on the comparison of the results from different samples, the uniqueness of the relationships presented in the paper was questioned because the specimens were compacted at different water contents and compactive efforts thereby producing varying soil fabrics. Therefore the parameters, M_a and λ_a will vary under this condition, uniqueness may not be obtained from the comparison of results from specimens of varying structures. However, uniqueness might be anticipated if initial soil fabrics were destroyed during shear to produce a similar structure at the end of the test. Another issue raised was that while a non-linear failure envelope, characterized by the equation 2.6 could be established using data from identical specimens, results from non-identical specimens as used by Toll (1990), might be mistakenly interpreted as having the non-linear characteristics of the shear strength envelope.

Alonso *et al.* (1990) presented a constitutive model for expressing the mechanical behaviour of unsaturated soil in the critical state framework. The empirical model was also based on the use of independent stress state variables. The model was intended for unsaturated soils which were moderately expansive. Figure 2.13 shows the graphical representation of the model proposed by Alonso *et al.* (1990). The following are the mathematical expressions for the model (matric suction is held constant). The normal compression line can be expressed as:

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$$v = N(s) - \lambda(s) \ln \left(\frac{p'}{p^c} \right) \quad (2.22)$$

The rebound line is expressed as:

$$N(s) - \lambda(s) \ln \frac{p_o}{p^c} + \kappa \ln \frac{p_o}{p_o^*} + \kappa s \ln \frac{s + p_a}{p_a} = N(0) - \lambda(0) \ln \frac{p_o^*}{p^c} \quad (2.23)$$

On the $q - p'$ plane, the critical state line is defined by,

$$q^2 - M^2(p' + p_s)(p_o - p') = 0 \quad (2.24)$$

The parameters in the above equations are:

- p' = excess of mean net stress over air pressure (mean net stress)
- p^c = reference stress for which v is equal to $N(s)$
- p_o = preconsolidation stress
- p_o^* = preconsolidation stress for saturated conditions
- p_a = atmospheric pressure
- p_s = intercept of the critical state line at a matric suction s (Figure 2.13)
- s = matric suction ($u_a - u_w$)
- q = deviatoric stress ($\sigma_1 - \sigma_3$)
- $N(s)$ = specific volume at p' equal to p^c
- $\lambda(s)$ = stiffness parameter for changes in p for virgin states of the soil
- M = slope of critical state line on $q - p'$ plane

Alonso *et al.* (1990) compared the predictions from the model with experimental observation and found close agreement between observed and computed results. It was observed however that constant water content tests cannot be reproduced with the proposed model unless a relationship between water content (or, alternatively, degree of saturation, S_r) and the stress state p , q and $(u_a - u_w)$ is established.

Wheeler (1991), using the data from Toll (1990), also proposed an alternative framework for unsaturated soil behaviour. It was pointed out that there were some difficulties and limitations in the framework proposed by Toll (1990). Wheeler's proposal involves one less independent variable (i.e., degree of saturation). Wheeler's proposal is of the form, for $q - p'$ plane:

$$q = f [(p - u_a), (u_a - u_w)] \quad (2.25)$$

and for $v - p'$ plane:

$$v_w = f [(p - u_a), (u_a - u_w)] \quad (2.26)$$

where,

v_w = specific water volume (i.e., volume of water plus solids in a volume of soil containing a unit volume of solids).

With Toll's (1990) data and this framework, Wheeler's (1991) results were consistent with those presented by Escario and Saez (1986) and Gan *et al.* (1988). Also with Toll's (1990) data, Wheeler predicted critical state values for the deviatoric stress and specific water volume respectively, with the following forms,

$$q = M(p - u_a) + f(u_a - u_w) \quad (2.27)$$

$$v_w = \Gamma - \lambda \ln(p - u_a) + f(u_a - u_w) \quad (2.28)$$

Wheeler justified the first form with Toll's (1990) data, while analysis with the second form was not conclusive. Wheeler also discussed two practical situations where critical state values of the deviator stress could be predicted. These are: (a) shearing with the water phase undrained (i.e., constant water content stress path) but the air phase is drained. This holds for rapid loading conditions as used by Toll (1990) in his shear tests; (b) shearing with both air and water phases drained (i.e., fully drained stress path).

Wheeler and Sivakumar (1992) used a five-state-variable critical state framework similar to that of Toll (1990). Isotropic compression states were defined by three equations embracing the five state variables, while critical states were defined by three equations.

The overall framework defined the unsaturated critical state model in p' , q , w , v , s , five dimensional space, that included equivalents of the isotropic normal consolidation line and the critical state line. Wheeler and Sivakumar (1992) postulated the following relationships for isotropic compression states:

$$\begin{aligned} q &= 0 \\ v &= f(p', s) \\ w &= f(p', s) \end{aligned} \quad (2.29)$$

and for critical state conditions,

$$\begin{aligned} q &= f(p', s) \\ v &= f(p', s) \end{aligned}$$

$$w = f(p', s) \quad (2.30)$$

where,

$$s = u_a - u_w$$

$$w = \text{water content.}$$

An experimental program was designed to examine these relationships. Tests in modified triaxial apparatus were carried out at two suction values of 100 kPa and 200 kPa over a range of p' in unsaturated compacted Kaolin. The results from the tests largely supported the proposed relationships.

Wheeler and Sivakumar (1992) pointed out that the data of Toll (1990) could be represented by a unique critical state relationship linking water content to mean net stress p' and matric suction s . Wheeler and Sivakumar (1992) also noted that Alonso *et al.* (1990) did not include a relationship for water content in the development of the critical state model, but this could easily be added without changing the existing nature of the model.

Wheeler and Sivakumar (1993) went further to examine the relationships between the five state variables p' , q , v , w and s . Constant suction range was increased to include 0 kPa and 300 kPa along with the previous 100 kPa and 200 kPa. Various stress paths like constant water content, fully drained, constant volume and constant p' shear tests were conducted. The findings further confirmed the uniqueness of the critical state line.

In a recent study, Wheeler and Sivakumar (1995) defined a critical state framework for unsaturated soil in terms of four state variables, p' , q , v and s instead of five in an earlier study. Water content was removed as a state variable. It was suggested

that it is possible to calculate plastic changes in water content (w) by the application of a flow rule.

Based on the experimental findings of Wheeler and Sivakumar (1993) and Petersen (1994), the following general forms of the modified Cam clay model for unsaturated soils can be postulated:

$$v = N(s) - \lambda(s) \ln p' \quad (\text{normal compression line on } v - p' \text{ plane}) \quad (2.31)$$

$$v = \Gamma(s) - \lambda_{cs}(s) \ln p' \quad (\text{critical state line on } v - p' \text{ plane}) \quad (2.32)$$

$$q = q_0 + M(s) p' \quad (\text{critical state line on } q - p' \text{ plane}) \quad (2.33)$$

where,

q_0 = intercept of the critical state line on the $q - p'$ plane

$N(s)$, $\Gamma(s)$, $\lambda_{cs}(s)$, $\lambda(s)$, are values of the critical state parameters at a given matric suction, s .

The mean net stress, p' , is defined for unsaturated soils as:

$$p' = \frac{\sigma_1 + \sigma_2 + \sigma_3}{3} - u_a \quad (2.34)$$

2.3.2 Application of the Critical State Concept to Agricultural Soils

Application of the critical state theory to the study of agricultural soil behaviour is relatively new. Theoretical basis for the application has been extensively discussed; however, experimental evidence has only really been shown in recent years. These studies show that the critical state concept can be applied in the formulation of constitutive relations for agricultural soil (Hettiaratchi and O'Callaghan 1980).

Bailey and Vandenberg (1968) proposed a critical state model for agricultural soils. Kurtay and Reece (1970) and Reece (1977) reviewed and discussed the relevance of the concept to agricultural soils. These studies presented a theoretical and qualitative basis for applying the critical state framework to practical agricultural problems.

Spoor and Godwin (1979) observed the critical state behaviour of three agricultural subsoils at different water contents and confining pressures in a series of undrained triaxial compression tests. It was shown that at low confining pressures, the soils failed in compression (wet side of critical state line) at high water contents (low matric suction). Brittle failure was observed on the other hand at low water contents. The transition from brittle to compressive failure occurred between suction of 35 kPa to 100 kPa. The authors surmised that the critical state concept explains the qualitative behaviour of the soils at high but not at low water contents.

Hettiaratchi and O'Callaghan (1980) presented a simplified version of the critical state theory. The purpose was to qualitatively analyze compaction and cultivation problems. With the critical state concept, soil behaviour during field operations (e.g., slipping wheels, cutting blades or tines) was considered within a sub or super critical domain. Practical possibilities of the rare tensile failure was also considered. This study also proposed that the effect of water content be considered in applying the critical state concept to agricultural soils. The study also demonstrated the need to establish the appropriate relationship between p and q .

Stafford (1981) applied the framework to a tillage problem involving rigid tines. Two general modes of failure (brittle and flow) were identified within the two regions of

the critical state model. Stafford introduced the concept of a “failure number” to enable the prediction of the failure mode from the magnitude of various parameters. Stafford defined a failure coefficient F which has a critical value above which soil fails in flow mode, and below which it fails in brittle mode. This coefficient was shown to be a function of water content, speed, rake angle and the reciprocal of density, making a dimensionless form of F impossible. Stafford (1981) suggested that by identifying other factors, a dimensionless F might be established. There was also a problem because of a lack of knowledge on the relationship between q and p . Effect of water content on the failure mode was also considered. It was asserted that there exists a critical range of water content pertinent to performance of a rigid line and a "wet side" and "dry side" approach to soil failure about the critical wall was suggested.

One of the important contentions of Hettiaratchi and O'Callaghan (1980) is that the critical state line varies with the water content of the soil. Leeson and Campbell (1983) investigated this by compressing two agricultural soils with water contents between 5 to 30% and degrees of saturation of 10 to 40%. The study indicated that the critical state line varies with water content and a modification of the state boundary surface due to changes in water content was proposed. Figure 2.14 shows that as the soil dries, the critical state line rises and the state boundary surface shifts away from the origin. It was concluded that the critical state theory could indeed be extended to unsaturated soils, and hence was

Figure 2.14 State boundary surfaces of saturated and unsaturated soils in p , q , v space (adapted from Leeson and Campbell 1983).

capable of predicting mechanical behaviour of soil during tillage and compaction even for soils of low water contents.

Towner (1983) highlighted the complexity of quantifying stresses in unsaturated soils and the implications in the application of the critical state theory. It was suggested that appropriate stresses can be obtained by directly comparing the total stresses on unsaturated soils with the stress on the skeleton of saturated soils for similar stress paths. Towner suggested that the application of the critical state theory to unsaturated agricultural soils must be validated with exhaustive tests on soil deformation along several different stress paths.

Hettiaratchi and O'Callaghan (1985) investigated the influence of water content on shear strength and the role of the microstructure of the soil. The authors explained that a cemented microstructure is obtained when an initially loose soil dries out while the particles maintain contact, and that a continuity of the air phase is formed. Remoulded microstructures are formed on the other hand, when an unsaturated soil is subjected to mechanical disturbance which redefines the sizes and distribution of the pore contents. Hettiaratchi and O'Callaghan (1985) investigated the influence of the water content and degree of saturation on volume change behaviour within the critical state framework. A "pivot point" concept was introduced to explain the variation of critical state parameters with water content. It was postulated that critical state line changes in slope (or rotates) about a point in the $v - p$ plane. Based on limited experimental evidence for remoulded soils, it was concluded that there exists a "sensitive" water content range beyond which a totally different volume change behaviour occurs.

Hettiaratchi (1987) described a universal but simple critical state model for agricultural soils. The state boundaries for a sand, a loam and a clay soil of varying water contents and soil microstructures were experimentally found. Critical state line was located at the end point of plastic failure for fully drained slow triaxial compression tests. Variation of critical state parameters in remoulded and cemented soils was discussed. In conclusion, the importance and relevance of the critical state theory to solving agricultural problems was reiterated. The need to develop simple experimental techniques to define the state boundaries was suggested.

Kirby (1989) measured the yield surfaces and established the critical state condition for four agricultural soils. An oedometer was used to investigate compression (or volume change) behaviour and a constant volume direct shear box used to determine shear behaviour. Each soil was tested at a single water content but with degrees of saturation between 20 to 98%. Different stress paths were considered in the shear tests. It was observed that soils in states looser than critical compacted in shear on approaching critical state, while the specimens in states denser than critical dilated. Kirby concluded that, for practical convenience, the yield surface, critical state line, normal compression line and rebound lines could be described in terms of total stress or applied stress. It was also pointed out that in order to formulate an unsaturated critical state framework, some issues must be addressed. Some of these are, quantifying stresses in unsaturated soils, the non-homogeneous deformation characteristic of the Hvorslev surface, and the volume change that occurs in unsaturated soil in the so-called undrained tests.

Kirby (1991) determined the critical state parameters for some field soils in eastern Australia. The compression behaviours of eighteen different soils were determined in an oedometer while shear behaviours were established with a direct shear box. The variation of critical state parameters and the inter-relationships with water content and bulk density was investigated. Kirby used the total stress approach but conceded that critical state parameters can be expected to vary when defined in terms of total stress. It was found out that in these soils, critical state parameters were dependent on water content, Atterberg limits and density of the soil. Another importance observation from this study was that the soils expanded in failure (dilated) if the normal stress during shear was less than about half the pre-compression stress, and failed in compression when the normal stress during shear was more than about half the pre-compression stress. Considerable scatter was also observed for the critical state parameters. Figure 2.15. shows that the compression and rebound indices, λ and κ increase with pre-compression stress. Potential causes in the variation of data in this study were discussed. Apart from water content, density, Atterberg limits and the use of the total stress approach, other sources of variation might have been the stress history of the soil and the use of field samples, which in themselves, vary from one sampling point to the other.

Petersen (1993) determined the variation of critical state parameters of two agricultural soils (a sandy loam and a loam) and the variation with water content. Cylindrical specimens were formed from air dried samples. The specimens were first wetted to zero suction and then brought to desired water content by drying on sand tables

Figure 2.15 Variation of compression (λ) and rebound (κ) indices with pre-compression stress for some eastern Australian soils (Kirby 1991).

or ceramic plates. This led to the formation of a cemented microstructure. Hence a similar microstructure was retained at the various water contents. Based on the observed effects on loading rate on the two soils, the rate was kept low.

From the data on the two soils, Petersen observed that λ and N are not linearly dependent on each other. Therefore the variation of the normal compression line with water content did not agree with the pivot point concept of Hettiaratchi and O'Callaghan (1985). Figure 2.16 shows variation of the critical state parameters of the two soils with water content. For both soils, λ increased in value with increasing water content for drier soils and decreased with increasing water content for wetter soils. Over certain water content ranges, λ did not vary much in the two soils. Variation of other critical state and intercept parameters are shown in Figure 2.16. These trends agree with the observations by Wheeler and Sivakumar (1993), for a cemented clay soil. While Wheeler and Sivakumar presented the variation of critical state parameters with matric suction, Petersen (1993) showed the variation of critical state parameters with water content (Figure 2.14). Petersen (1993) concluded that the role of water content was expressed in the values of the measured critical state parameters and that these parameters vary in a systematic manner with water content. Volume change behaviour was also consistent with the critical state concept.

Wulfsohn (1994) reviewed the essence of plasticity theory in critical state soil mechanics. A brief historical account of the application of the critical state theory to

Figure 2.16 Variation of critical state parameters with water content (a-f, after Petersen 1993) and with matric suction (after Wheeler and Sivakumar 1993).

agricultural soils was presented. The implication of an unsaturated soil mechanics framework for critical state studies was introduced.

O' Sullivan *et al.* (1994) derived critical state parameters for three agricultural soils (a sandy loam, clay loam and a clay) in the constant cell volume triaxial system developed by Hettiaratchi *et al.* (1992) (discussed below). Tests were conducted on remoulded specimens over a range of water contents. It was observed that the normal compression lines tended to be linear on the $v - \ln p'$ plane up to a degree of saturation about 85%. At higher saturation, the soil became incompressible, but highly deformable. The slopes of the projected critical state lines were slightly higher than those of the normal compression lines. Data from the three soils indicated that both the normal compression and critical state lines pivoted about a point as water content increased.

Kirby (1994) examined the ability of a critical state model to predict volume change and shear behaviour of agricultural soils in laboratory specimens. The model in this study was an extension of the modified Cam Clay model. It was extended in such a manner as to produce a smoother transition from elastic to plastic behaviour. Kirby showed that model predictions generally compared well with experimental data, but observed numerical problems for brittle soils.

Petersen (1994) also made some modifications to the Modified Cam Clay model to account for volume change and shear observations in two soils (i.e., a sandy loam and a loam), each at three different water contents. The modified model was used to predict the maximum shear strains for triaxial specimens for two different stress paths. The closeness between predicted and observed maximum shear strains depended on the stress path and

soil water content. The model failed to predict maximum shear strain at stress states close to the critical state stress.

From the historical review of the application of critical state theory to agricultural soils, it is evident that there are some recurring issues which still need to be addressed. A generally acceptable approach is yet to be agreed on, for example in sample preparation techniques or in the analysis of the critical state parameters. The differences in the approaches used by researchers could be attributed to the nature of problem under investigation, the type of soil, the available testing apparatus and experimental methods. However, it is generally accepted that it is necessary to investigate the variation of critical state parameters (hence state boundaries) with water content (or matric suction) and the influence of microstructure. Some of these issues are now discussed.

It appears from the reported studies, that critical state parameters are influenced by the method of specimen preparation. Different micro-structures are produced in remoulded and cemented specimens. Variation of critical state parameters with water content differs in remoulded and cemented specimens (Hettiaratchi 1987).

The water content or the soil suction are important variables in unsaturated soils. These must be considered in any critical state model that is formulated for agricultural soils. While water content could be considered as a deformation variable, matric suction is a stress state variable. The influence of matric suction as additional stress in the soil appears to outweigh changes in pore-water volume during deformation.

Many researchers have analyzed the variation of critical state parameters with water content. This means that numerous tests must be conducted over ranges of practical

water contents. Adams *et al.* (1994) suggested the use of matric suction as variable in conjunction with the soil-water characteristic for agricultural soils. This could reduce the number of tests required to study soil behaviour. The use of matric suction also permits the isolation of the effect due normal stresses from that due to the negative pore-water pressure according to the shear strength equation 2.6.

In the application of critical state theories to agricultural soils, many researchers have used the total stress approach. This was justified by Hettiaratchi (1987) on the premise that pore-water does not interfere with pore volume and pore-water pressure change. Others used the total stress approach for practical convenience. Results from many studies appear to validate the use of the total stress approach, but a closer look at the experimental data suggests that the total stress approach may be limited by the soil micro-structure, the water content and texture of the soil. It has been suggested that the variation of critical state parameters and the orientation of the critical state line may be due to the use of the total stress approach (O'Sullivan *et al.* 1994; Petersen 1993). Towner (1983) pointed out that the uniqueness of the critical state line is more justifiable when appropriate stresses for unsaturated soils are considered.

While some studies reported ductile behaviour of agricultural soils on the wet side of the critical state wall (i.e., on or below the Roscoe surface), some showed brittle behaviour on the dry side (i.e., on or below the Hvorslev surface). The validity of the critical state theory on the wet (sub-critical) side was questioned by Spoor and Godwin (1979), but other workers upheld the validity for both ductile and brittle behaviour. Compaction problems can be studied by considering soil behaviour on the Roscoe surface.

This will involve the testing of normally compressed or lightly overconsolidated soil. Tillage studies can be considered within the Hvorslev domain by testing overconsolidated or dry soils.

For the determination of critical state parameters and the state boundary surface, standardized testing methods and apparatus are yet to emerge. While most researchers have used various modifications of the triaxial apparatus (Leeson and Campbell 1983; Hettiaratchi *et al.* 1992; Petersen 1993) some have used a combination of oedometer and direct shear box (e.g., Kirby 1989). One advantage of using the triaxial apparatus to determine critical state parameters is that volume change and shear strength behaviour are integrated.

Typical values of critical state parameters determined for unsaturated soils are shown in Table 2.1. These parameters were determined for soils at various water contents.

Table 2.1 Typical ranges of critical state parameters from other studies.

Soil type	Parameter					
	λ	λ_{cs}	κ	M	N	Γ
Petersen (1993)						
Sandy loam	0.08 - 0.18	0.08 - 0.18	-	0.5 - 1.6	2.3 - 2.6	2.0 - 2.6
Loam	0.06 - 0.22	0.14 - 0.24	0.016*	0.5 - 2.2	2.2 - 2.7	2.1 - 2.7
O'Sullivan <i>et al.</i> (1994)						
Sandy loam	0.05 - 0.01	0.01 - 0.13	-	0.9 - 1.4	2.0 - 2.4	1.7 - 2.4
Clay loam	0.05 - 0.25	0.05 - 0.30	-	0.2 - 1.7	1.9 - 2.8	1.9 - 2.8
Clay	0.17 - 0.40	0.22 - 0.45	-	0.7 - 1.7	3.2 - 4.4	3.4 - 4.4
Kirby (1991)						
Silty clay	0.16	-	0.009	0.94	-	-
Wheeler and Sivakumar (1993)						
Kaolin	0.12 - 0.20	0.12 - 0.20	-	-	2.04 - 2.20	-

* average value from several tests

2.4 Triaxial Testing of Unsaturated Soils

Triaxial testing of soil involves the application of normal stress in three principal directions to a cylindrical soil specimen. The triaxial apparatus can be used to study the shear and volume change behaviour of soils. Various soil volume change and shear strength parameters are determined with the apparatus. Soil pore-water and pore-air pressures can be measured. Various stress paths can be examined using the triaxial cell.

Conventional triaxial testing apparatuses have been designed for testing saturated soils. Some modifications are necessary if the apparatus is to be used to study unsaturated soil behaviour. To determine the shear strength of unsaturated soils, Bishop and Henkel (1962) and Fredlund *et al.* (1978), amongst many other recent workers have recognized and adopted several methods to modify the conventional triaxial apparatus.

Generally, a triaxial strength testing program involves :

- Preparation of cylindrical specimens from soil samples.
- Mounting of specimen in triaxial cell while ensuring contact with end stones, volume change measuring device and pore-water pressure devices.
- Application of confining pressure prior to shearing.
- Loading to shear failure by the application of deviator stress. Common stress paths followed during shearing are consolidated undrained (CU), consolidated drained (CD), constant water content (CW) or unconsolidated undrained (UU).

Modifications for unsaturated soils involve not only the apparatus, but also testing procedures. Specimen preparation techniques and the use of special procedures, like axis-

translation and backpressuring are some of the important considerations. Others factors and precautions to be considered in the modifications are;

- Provision for matric suction measurement or control
- Provisions for pore-air pressure measurement or control.
- Arrangement for air volume change measurement
- Prevention or suppression of air diffusion through specimen rubber membrane, stones and plumbing lines.
- Diffused air volume indicator (DAVI)

2.4.1 Outline of Testing Procedures

Specimen preparation

The terms soil sample and specimen will be used in the following context. The sample is a whole batch of soil taken as representative of a larger body of soil. A specimen is a portion of the sample which is taken out for use in laboratory tests; thus several specimens can be formed from the same sample batch. The soil sample is air dried, mixed thoroughly with some distilled water to the required water content and stored in an air tight container for at least 24 h to allow equalization of the water content.

Cylindrical triaxial specimens are formed from this sample. Specimens are usually formed by compaction in moulds. This process is used to bring the soil to desired (remoulded) density. Specimens for triaxial tests can also be taken from sample cores taken from the field.

As a general rule of thumb, the dimension of the specimen is such that the length to diameter ratio (L/D) is 1:1.5 to 2. Bishop and Henkel (1962) described the preparation of triaxial specimens by the Proctor or British standard compaction method. With this method, 102 mm diameter samples are usually prepared in a three part split mould 203 mm long. The split mould prevents a change in the density when the specimen is being removed after compacting. Barden and Sides (1970) adopted full and half standard Proctor efforts for compacting specimens used in the study of the engineering behaviour and structure of compacted clays. The U.S Department of Interior (1974) outlined a method of dividing the weighed sample into ten equal parts with each part placed separately in the mould and compacted in layers one after the other.

The fabric or soil structure, and hence the strength of soil, is affected by the compaction method and water content as exemplified by the studies of Lambe (1958, 1961) and Seed and Chan (1959). Compaction could be static, dynamic or kneading depending on the objective of the test, soil type and attainment of a particular fabric. Based on these studies, Lee and Haley (1968) suggested that in places where strength is the primary objective of the test, one might choose to compact the soil in such a way as to achieve a flocculated structure, for example by static methods. However, if plasticity or flexibility is desired, the soil could be compacted to a dispersed structure by kneading the soil wet of optimum. Escario (1980) used the static method to prepare specimens compacted at 100% Proctor (at maximum density and water content of 29%) for penetration tests, while triaxial specimens (38 mm diameter) were compacted to the same

state, but by kneading compaction. Gulhati and Satija (1981) used static compaction to produce high density and low density triaxial specimens. Wheeler and Sivakumar (1993) opted for lower compactive effort in order to prepare specimens of low pre-consolidation pressure. Static method was used to prepare 50 mm diameter specimens at water content of 25% (4% less than optimum). The samples were compacted in nine layers, each layer statically compacted in a compression frame at a fixed displacement rate of 1.5 mm/min to a pressure of 400 kPa.

The density of a compacted specimen is affected by the water content of the sample. An optimum water content exists at which a compacted specimen attains maximum density. The resulting specimen may not necessarily have the same water content as the batch from which it was prepared, hence the water content is more accurately determined from the measurements made at the end of the test.

From laboratory investigations, Peterson (1988) surmised that specimens moulded by static or dynamic compaction were stronger, more brittle and tended to swell as compared to those moulded by kneading compaction. Also, specimens that were compacted dry of optimum water content were stronger, more brittle and tended to swell in comparison with specimens that were compacted wet of optimum, which were weaker, more ductile and tended to consolidate.

Prior to placement in the triaxial cell, the specimen is weighed in air and water for mass and volume determinations (dimensions are also taken). End plates are placed at the top and bottom to support the specimen, while fitting the rubber membranes which are

enlarged by a vacuum expander. Rubber O-rings are rolled into place to grip the membrane tightly against the end discs.

In the preparation of compacted laboratory specimens from unsaturated soils samples, attention should also be paid to the soil suction. Attempts are usually made to simulate the in-situ suction. This could be achieved by conditioning the specimen prior to testing. Chu and Chen (1976) reviewed some pre-testing treatment methods and also developed an apparatus for simulating the field water conditions of unsaturated soils. Ho and Fredlund (1982a) relaxed the specimen suction by introducing water through the air pressure line and giving several hours to allow distribution throughout the sample.

Backpressure

To determine the saturated soil properties from an unsaturated sample, the specimens are first saturated prior to testing. Saturation involves dissolution of free air in pore-water. Two methods of saturation are: applying compression under undrained condition; and backpressuring of de-aired water into the specimen.

In the first method, by increasing the confining pressure under undrained condition, pore-air is compressed and goes into solution. With this method, the water content of the specimen remains constant. However, the accompanying volume change is a major setback.

Saturation may also be achieved by increasing the pore-water pressure, u_w , while the confining pressure is simultaneously increased in order to maintain a constant effective stress ($\sigma_3 - u_w$). The increase in pore-water and confining pressure is termed backpressure.

Backpressuring of de-aired water into the specimen compresses and dissolves pore-air. The saturation process here is performed such that the water content and degree of saturation increase.

By considering other factors including required pressure, water volume available for dissolution of air, and the efficiency of the applied pressure to cause saturation, Head (1986) showed that backpressuring is a better method of saturating.

Backpressuring is particularly important for unsaturated soils as it allows the dissolution of entrapped air during drained and undrained tests. For tests where pore pressure measurements are required, backpressuring helps to eliminate air bubbles from the measuring system.

Backpressuring is also very useful when testing dense samples which are expected to dilate during shear, especially for unsaturated soils. Application of large deviator stresses may lead to a decrease in pore-water pressure. The decrease may be sufficient to lower pore-water pressure below atmospheric, thereby leading to cavitation in the pore pressure system (Bishop and Eldin 1950). Under this condition, a constant volume state can not be achieved for CU tests and the applied stresses will no longer be the effective stresses for CD tests (Bishop and Henkel 1962).

Pore-water pressure control and measurement

In the testing of unsaturated soil, there is the need to separate the water phase from the air phase in the measuring devices. Hence pore-air and pore-water pressures are measured or controlled in different lines.

In the pore-water pressure measurement line, a saturated ceramic porous disk sealed by epoxy onto the base pedestal. The disk (with fine pores) allows passage of water, but cuts off the flow of liberated air, provided the air entry value of the disk is not exceeded. The disk is connected from the underside to a water compartment and a pressure measurement or control system. Pore-water pressure measurement devices have been classified as direct or indirect. Direct methods are used on the triaxial apparatus.

As a direct method of pore-water measurement, water filled tubes and mercury U-tubes in a null method arrangement were used at the Imperial College. This set up is described by Bishop and Henkel (1962). More recently, pore-water pressure has been measured by pressure transducers which are plumbed directly into the pore-water line. Ho and Fredlund (1982a) emphasized the need to use porous stones of the appropriate air entry values for testing unsaturated soils. To ensure a continuous connection between the specimen pore-water and the pressure transducers, the matric suction of the soil must be less than the air entry value of the disk.

Pore-air control

In a similar arrangement, pore-air pressure is controlled along a line which is connected to the specimen via another porous disk. A coarse corundum disk, which in this case allows passage of air, is commonly used. The disk is usually sandwiched between the loading cap and the top of the specimen. The pore-air line is usually connected to an air compressor via a regulator. Adams *et al.* (1996) connected the pore-air line to a digital pressure-volume controller.

Accuracy of pore-air measurement is dependent on the breakline or separation of the menisci between the water of the soil and the water in the disk. Conditions may arise when air and water intermix at the specimen-disk interface, resulting in a disruption of air flow and leading to inaccurate readings. Such conditions could arise during the loading stage when a high degree of saturation develops at this interface or high suction in a fine grained soil pulls water from the disk.

The US Department of Interior (1974) applied the following steps to overcome this problem. To enhance further breakline between the water of the soil and disk and prevent addition or removal of water from the disk, a fine (#200 mesh) screen sprayed with silicone mold release was sandwiched between the disk and the soil, and a water control burette was also plumbed to the disk.

Axis-translation technique

For unsaturated soils with high matric suctions, water cavitation is a problem in the pore-water pressure control or monitoring system. This could lead to erroneous readings, especially with the direct method of pore-water pressure determination. This problem can be overcome by the axis-translation technique of Hilf (1956), described by Fredlund and Rahardjo (1993a). Bishop and Blight (1963), Ho and Fredlund (1982a) and Wulfsohn *et al.* (1994) used this technique in testing unsaturated soils.

Axis-translation requires continuity of air space in the specimen. This cannot be achieved when the degree of saturation in the specimen exceeds 70%. Hence, application of axis translation is limited to the range of high suctions (Komornik *et al.* 1980). Limitations of this technique are described by Bocking and Fredlund (1980).

The general idea is to translate the pore-air and pore-water pressures into the positive range by creating an artificial atmospheric pressure greater than 101.5 kPa. Hence the axis translation is usually applied when suctions greater than about 100 kPa are to be imposed during testing. To maintain a constant suction across the specimen, the pore-water pressure is increased by the same amount as the pore-air pressure.

Volume change measurement

Volume change commonly occurs in a triaxial specimen when the cell pressure or axial load is changed. In the testing of saturated specimens, volume change is significant for CD tests, while it is negligible for CU tests. In unsaturated specimens, however, volume changes in CU, CD and CW tests are significant and have to be measured. Volume change measurements are required in order to determine the soil compressibility and the changing cross-sectional area of specimen.

Volume change in unsaturated soils involves change in pore-water or pore-air volume. For soils of low degree of saturation, air volume change predominates, while there is little or no pore-water volume change. For wetter soils, pore-water volume change

could be significant. Each of these components could be measured separately, or together as an overall volume change.

Bishop and Henkel (1962) described some methods of volume change measurement for triaxial specimens. Volume change can be measured with twin burette volume change indicators under two arrangements:

- (i) the twin burette is plumbed to the cell to indicate the volume of water that is added into the cell when specimen changes volume as a result of change in cell pressure or axial load.
- (ii) the twin burette is plumbed to the pore-water line to indicate the volume of pore water under drained conditions.

The first arrangement is used to measure overall volume change of the specimen in CU, CD and CW tests. There are some problems however, if this method is to be used for long term testing. Bishop and Henkel (1962) and Fredlund and Rahardjo (1993a) highlighted these as:

- Expansion of cell due to increase in cell pressure.
- Displacement of water from the cell by the movement of loading ram into the cell during loading.
- Leakage or volume change of cell fluid due to pressure and temperature variations.
- Air trapped in top cap of cell or between sample and rubber membrane. A correction for the latter scenario according to Boyle's and Henry's law is detailed in Bishop and Henkel (1962).
- Diffusion of air through pore-water and the rubber membrane.

The second arrangement described by Bishop and Henkel (1962) is popular for drained tests (CW and CD) only. Here, the burette measures the pore-water volume change plus diffused air volume moving into the compartment below the high air entry disk. The volume of diffused air can be measured separately and deducted from the total water volume change. A diffused air volume indicator (Fredlund and Rahardjo 1993a) may be used for this purpose.

Total volume change can also be determined by a direct measurement of changes in the specimen's radial and axial dimensions. Linear differential variable transducers (LVDT) or a dial gauge are commonly used to measure axial strains. A lateral strain indicator for 102 mm specimen was described by Bishop and Henkel (1962). Non-contacting transducers have also been used by Cole (1978), Khan and Hoag (1979) and Drumright (1989) to measure radial strain. Lateral strain measurements are however limited to small radial deformations and thus may not be suited for most agricultural soil testing.

Hettiaratchi *et al.* (1992) measured total volume change of specimens, using a constant volume triaxial testing system. The compliance of a closed hydraulic system was used to determine volume changes in specimens as external stresses were applied. Predictions by a mathematical model of the system were compared with experimental data. Preliminary findings showed that the system could be used to acquire quantitative data relevant to the shear and volume change behaviour of unsaturated soils.

Escario (1980) used an optical method (Escario and Uriel 1961) to measure the changing cross sections of triaxial specimens for an overall volume change determination.

For CD tests, Karube (1988) attached a pair of distortion or lateral strain sensors crossing the diameter to measure overall volume change of the specimen. The volume of the specimen was calculated by averaging a column approximation and an approximation for a revolved parabolic body in which both ends keep the initial diameters.

Adams *et al.* (1996) customized a digital pressure-volume controller originally designed to measure liquid volume change, to measure air volume change. The cylinder of this device was plumbed into the triaxial apparatus via a copper tube and a valve. This valve in turn was connected to the specimen top cap via another tube. This arrangement allowed direct air volume changes to be recorded and at the same time, readily permitted the implementation of the axis-translation technique.

Problems of air diffusion and leakage

Some problems that are encountered in triaxial testing of unsaturated soils arise from the presence of diffused and dissolved air. Air could diffuse from the specimen through pore-water and through the rubber membrane. Air can also dissolve under pressure into pore-water and then pass through the ceramic disk into the water compartment.

These problems will directly affect measurement of pore-water pressure in undrained tests, and also measurement of overall volume change in drained tests. For true undrained tests, air and water must not leave the specimen during testing. In drained tests,

it may be necessary to separate the volume changes due to water and air, as both pass through the ceramic disk into the volume change indicator.

Some techniques have been used to overcome the problem of air diffusion through the rubber membrane. These include substitution of mercury for de-aired water as cell fluid and use of a composite membrane (e.g., an aluminum sheet sandwiched between two rubber membranes). Air escaping into the drainage line through the ceramic disk can be flushed off or passed into a diffused air volume indicator for measurement.

Bishop and Donald (1961) used mercury as the cell fluid in order to prevent the diffusion of pore-air through the rubber membrane into the cell during undrained tests. It was shown that there was significant drop in pore-air pressure due to diffusion when de-aired water was used as cell fluid in comparison with mercury. Escario (1980) acknowledged the use of mercury columns to apply lateral pressures, but cautioned that application of air pressure into specimen in excess of lateral pressure might blow out the membrane and suddenly expel the mercury.

Dunn (1965) showed that diffusion of pore-air through the pore-water and membrane can be greatly reduced by using two slotted aluminum sheets with silicone grease sandwiched between. These are in turn sandwiched between two rubber membranes. Komornik *et al.* (1980) used rubber membranes soaked in silicon oil to prevent the leakage of air or water from the specimen. Poulos (1964) gave further details on checking leakage from triaxial specimens.

A detailed procedure for flushing out diffused air in the drainage line is described by Fredlund and Rahardjo (1993a). Diffused air is periodically flushed from the water

compartment into a DAVI during consolidation and shearing. To achieve flushing, a pressure differential of 7 to 70 kPa is set up between the base plate and DAVI. Gulhati and Satija (1981) used a bubble bump to flush out air bubbles in pore-water and drainage lines.

Application of confining pressure or isotropic compression.

After mounting the specimen and setting up the triaxial cell, a confining pressure (σ_3) is then applied. The purpose is to allow the specimen to consolidate (or reach equilibrium) under a typical value of in-situ stress. The confining pressure could be applied as a ramp or step pressure.

Some factors which could affect the selection and application of confining pressures are: the specimen's pre-compression pressure; and the specific purpose of testing. Confining pressure could also be applied incrementally along with pore (air or water) pressures in order to prevent premature compression of the specimen.

Lee and Haley (1968) indicated that the method of compaction affects whether the specimen will consolidate or swell. Hence, the style as well as extent of compaction should be carefully chosen such that the specimen consolidates or swells as desired for the selected confining pressures. Wheeler and Sivakumar (1992) used a low compactive effort to statically mould specimens of low values of apparent pre-consolidation pressure, so that the specimens could be normally consolidated to a virgin state.

As an experimental precaution, Komornik *et al.* (1980) chose a range of confining pressures such that the lower limit (e.g., 25 kPa) would ensure full contact between the

membrane and inner surface of the specimen. An upper limit (e.g., 295 kPa) was chosen such that membrane leakage is eliminated.

Peterson (1988) isotropically consolidated a group of specimens (moulded by kneading compaction) to stress states of 70, 140, or 280 kPa. A second group was compressed to 280 kPa and then followed by rebounds to 140 or 70 kPa before shearing. A third group was compressed to 1100 kPa and rebounded to 550, 280, 140, or 70 kPa before shearing. The purpose of variation of the confining pressure in this manner was to subject the soil to different stress histories prior to shearing.

In isotropic compression tests, Karube (1983, 1988) increased the confining pressure stepwise from an initial equilibrium value of 20 kPa every 6 h or 8 h while keeping pore-water pressure (gauge), u_w equal to 0 (i.e., drained condition). In the tests, it was intended that the specimens be taken through various stress paths.

Wheeler and Sivakumar (1992) first brought specimens to an equilibrium mean net stress p' of 50 kPa and suction of 100 or 200 kPa using the axis translation technique. The specimens were then isotropically consolidated under a constant suction by ramping the cell pressure at a constant rate of 0.6 kPa/hr up to a pre selected value of p' . The ramped consolidation took the specimens to the virgin state after having been stressed to levels beyond the apparent pre-consolidation pressure from the initial compaction process.

Loading and rates of testing

When a specimen comes to equilibrium under a confining pressure, or when a pre-selected value is reached via isotropic compression, the appropriate drainage conditions (CU, CD, or CW tests) are imposed and the sample can then be taken to shear failure by applying an axial load (which produces a deviator stress). The axial load could be applied as controlled deviatoric stress ($\sigma_1 - \sigma_3$) or at a controlled strain. Other loading paths are also possible.

Critical state parameters of granular soil are usually obtained under drained ($\delta\sigma'_3 = 0$) triaxial tests. But care must be taken especially when testing very dense specimens, as it has been shown by Hettler and Vardoulakis (1984) that shear bands and other kinds of non-homogeneous deformations develop prior to the soil reaching critical state. It was shown that this phenomenon affects the soil properties, and as such puts some uncertainty on critical state parameters obtained under conditions where non-homogeneous deformations have occurred.

Chances of non-homogeneous deformation are more likely to occur

- (i) in poorly prepared specimens,
- (ii) with inappropriate testing rates, and
- (iii) with eccentricity in vertical alignment.

From this study it appears that attainment of critical state could be sensitive to the homogeneity of the soil. Care must be taken to prepare uniform specimens. During placement in the cell, the loading cap placed on top of the specimen must also be properly

aligned with the loading ram. The choice of an appropriate strain rate also becomes pertinent. Fast testing rates may induce non-homogeneity.

Selection of suitable rates for drained tests may be found by a process of trial and error, but it is useful to be able to calculate in advance the approximate time required for the test. Gibson and Henkel (1954) and Bishop and Gibson (1963) applied the theory of consolidation to estimate suitable values of strain rate. This method appears to be appropriate for saturated soils.

Ho and Fredlund (1982b) pointed out some of the complications involved in the selection of suitable strain rates for testing unsaturated soils. The two major complications are the high air entry disk placed below the specimen, and the physical properties of unsaturated soils. The low coefficient of permeability of the disk impedes flow in and out of the specimen. This, along with the coefficient of permeability and compressibility of unsaturated soil, affects the ability of the measuring system to dissipate excess pore pressures. These factors must be considered in the choice of a strain rate.

Studies have shown that shear strength testing of unsaturated soils should be performed at extremely slow rates in order to ensure equalization of induced pore pressures (in CU tests) or dissipation (in CD tests). Some criteria have also been suggested regarding the approximation of strain rates. Donald (1961) suggested that the effect of strain rate on maximum deviator stress should be used as a criterion. For constant water content tests, Satija and Gulhati (1979) suggested changes in matric suction as a criterion.

To ensure that the chosen rate of strain permitted the dissipation of pore-water pressure in CD tests, Williams and Shaykewich (1970) conducted preliminary tests over a wide range of strain rates, by geotechnical engineers' standards. Strain rates were varied from 0.02 to 0.002 cm/min. Final selection of strain rates was based on the rates which did not significantly affect the deviator stress at failure.

Ho and Fredlund (1982a) used constant strain rate of 0.001% strain per minute, the choice being based on coefficient of consolidation observed during consolidation of sample.

Karube (1988) used the controlled stress method of loading in which applied stresses were changed stepwise every 8 h. It was observed that the exact condition of failure was seldom defined because of this method. Estimation of failure stresses had to be made based on presumptions and extrapolations.

2.4.2 Modification of the Conventional Triaxial Apparatus

For the testing of unsaturated soils, major modifications to the conventional triaxial apparatus involve the separation of the air and water phases. The changes are made on the cell base ring, pedestal and plumbing lines. Figure 2.17 shows a modified triaxial cell (a) and the baseplate (b). The following are the summary of the modifications needed to adapt a conventional triaxial apparatus for testing unsaturated soils.

(a)

(b)

Figure 2.17 Triaxial equipment for unsaturated soils (a) Modified cell (b) Modified triaxial baseplate (after Fredlund and Rahardjo 1993a).

Replacement of stones at top and bottom

The sintered bronze porous stones that are normally placed at the ends of the specimen for saturated tests, are replaced by a fine ceramic disk (at the bottom) and a coarse corundum disk (on top).

Provision of air supply and drainage line

An air drainage tube is connected by a metal fitting to the corundum disk. The disk can be machined into the top cap to form a single unit. The drainage tube is connected to an external port on the cell base ring. The external port, in turn, is connected to a compressed air supply line which is required in order to implement the axis-translation technique.

Provision of water compartment and extra drainage line on the baseplate

The baseplate should be redesigned such that a ceramic disk can be mounted. When the disk is mounted, there should be a clearance which will form a water compartment between the top of the pedestal and the underside of the disk. The water compartment is required so that the ceramic disk remains saturated during a test.

A single drainage line is usually provided on the baseplate of the conventional triaxial apparatus. An additional line is needed for unsaturated soil testing. The extra line is required for flushing and diffused air volume measurement. The two drainage lines are connected through a spiral groove which is machined into the top of the baseplate. The spiral groove improves the air flushing process.

Provision of extra ports on the cell base ring

The base ring of a conventional cell usually has two functional ports, one for the pore-water line and the other for cell water. Two extra ports are needed for unsaturated soil testing. One extra port will allow the connection of the pore-air space with external air supply line while the other permits the connection of the flushing line on the baseplate to an external DAVI or bubble collector.

Arrangement for air volume change measurement

An arrangement is required such that the volume change of the specimen can be recorded. This could be any of those described in section 2.4.1.

Extension of loading ram stroke and load cell capacity

Unsaturated soils are usually stronger than saturated soils. Larger deformations could also be involved when loose unsaturated soils (like agricultural soils) are tested. Hence the loading ram on the triaxial apparatus should be able to accommodate a longer stroke, the load cell capacity must also be considered.

CHAPTER 3

EXPERIMENTAL PROGRAM, EQUIPMENT AND PROCEDURES

3.1 Problem and Assumptions

This research will study the behaviour of an agricultural soil in a modified triaxial apparatus. There are limitations to the application of laboratory data to field conditions. For instance, the soil structure, loading pattern and drainage may not accurately represent what is occurring in-situ. Because of the interest in a broad based model, some assumptions were made, additional to the standard assumptions made in critical state modelling (Section 2.2.2).

In classical soil mechanics, when a saturated soil medium is loaded statically, the total stress is first supported by the pore fluids which build up pressure. Subsequently the stress is transferred to the soil skeleton. For most agricultural operations however, the soil is rapidly loaded and unloaded (e.g., intermittent passes of traction members).

For the span of time involved in this kind of loading it can be assumed that :

- (a) the ensuing deformation results from reduction in the pore-air volume as air is expelled from the pores, and
- (b) soil gravimetric water content remains unchanged.

In this regard, triaxial constant water content tests (CW) form the core of the experimental program. Fully drained (CD) as well as undrained (CU) tests were both involved in order to investigate the effects of the stress path and to establish state boundaries with respect to matric suction.

Loading of agricultural soils is generally dynamic. However, it is not appropriate to apply field speeds in laboratory triaxial tests aimed at obtaining fundamental soil properties. At such speeds, a pore pressure gradient is set up across the specimen and end effects are rapidly mobilized resulting in non-homogeneity between the soil near the end caps and other parts of the specimen. Under these conditions, it becomes difficult to interpret shear strength and volume change properties. Yong and Japp (1968) recognized this problem during high speed compression of triaxial specimens. Corrections were applied to the tests to account for the effect of inertial forces (due to mass of the loading cap) on soil deformation. Corrections were also made to account for acceleration sensitivity of the force transducer which was located below the specimen to measure axial force.

Apart from speed, another indicator of dynamic behaviour is the drainage condition of the soil. Soil could be rapidly loaded in a manner such that the pore-water volume is unchanged even though the pore-water pressure might increase. This scenario is represented by undrained or constant water content tests in the laboratory. It is generally easier to apply field drainage conditions than field speeds in laboratory studies.

Based on the assumptions and the discussion above, slow testing rates were applied (for static consideration) and an undrained water phase (for dynamic

consideration) in a series of constant water content tests. This quasi-static approach was taken in the experimental investigations.

The implication of a quasi-static approach in the testing of an unsaturated soil was also investigated. Of the independent stress state variables, matric suction is particularly sensitive to testing rate in constant water content tests (Satija and Gulhati 1979). Because of the low permeability of the high air entry disk and the physical properties of the soil, unsaturated soils are usually tested at slow rates in order to allow equalization or dissipation of pore-air and pore-water pressures (Fredlund and Rahardjo 1993a). Variation in matric suction was carefully monitored in all the constant water content tests to investigate whether matric suction should be included or ignored in the soil models.

3.2 Models Under Investigation

A form of the modified Cam clay model which is adaptable to unsaturated soils was investigated. The model involves different constitutive relationships for the critical state planes and state boundary surface. Matric suction is considered as an additional soil stress state variable. The mean net stress p' is defined as for unsaturated soil in equation 2.34.

(i) *Isotropic deformation on the $v - p'$ plane*

Isotropic deformation on the $v - p'$ plane is compression that excludes any shear stress ($q = 0$); deformation is due to isotropic stress change only. However, volume changes are involved and are ascribed to both pore-air and pore-water. On the critical state $v - p'$ plane, a form identical to equation 2.29 was experimentally investigated:

$$v = f(p', u_a - u_w) \quad (3.1)$$

Because of the proximity of agricultural soils to the surface, soil elements experience smaller geostatic stresses in comparison to those at deeper layers. Hence in examining this deformation model, low mean net stresses were generally imposed on the test specimens. Higher mean stresses were imposed in some experiments in order to locate limits of critical state boundaries.

(ii) *Vertical loading*

In this research two stress paths for the unsaturated soil were examined, namely,

- (i) constant water content loading in which matric suction varied during shearing, and
- (ii) fully drained tests under constant suction.

The locations of the critical state points for the constant water content tests and the orientation of constant suction critical state lines on the $q - p'$ plane were examined.

The forms of the model during shearing for CW and CD tests are:

$$v = f(p', u_a - u_w) \quad (3.2)$$

for the projection of the critical state line on the $v - \ln p'$ plane, and

$$q = f(p', u_a - u_w) \quad (3.3)$$

for the model on the $q - p'$ plane. These are similar to equations 2.30 and 2.31, respectively.

3.3 Overview of the Experimental Program

A series of pilot experiments were carried out along with the main triaxial experiments. Results of these experiments were subsequently used to support or complement the main experiments.

Soil samples were prepared in batches using the procedures described below. A batch is a soil sample which is prepared at a particular water content. In the experiments, specimens were prepared from specific sample batches.

Standard tests were conducted to obtain soil index properties. These included determination of particle size distribution, Atterberg limits (i.e., plastic limit and liquid limit), specific gravity and compaction test.

Soil-water characteristic curves were determined using specimens prepared from batches at two water contents. The specimens were prepared in a similar manner in order to produce the same structure as those of the specimens used in the uniaxial compression, direct shear and triaxial tests.

Direct shear tests were conducted on specimens prepared from sample batches at five different water contents. The specimens were sheared under various normal stresses in order to obtain the strength parameters. Effect of rate of shearing was also investigated.

Uniaxial compression tests were conducted in order to determine pre-compression stresses and to investigate the variation of void ratio with imparted stress. Specimens that were used for uniaxial compression tests were prepared from two sample batches.

A triaxial experimental program was carried out using a triaxial apparatus which was first modified for testing unsaturated soils. Unsaturated tests were conducted as constant water content (CW), and constant suction drained (CD) while the saturated tests were conducted as consolidated undrained (CU) tests. Each type of test was selected for particular reasons. For example, a constant water content path represents the state of the soil during most agricultural operations whereas constant suction tests define the boundary for a particular suction. A combination of CW and CD tests means that the relative position of the constant water content tests (which involves change in suction) to this boundary could be investigated.

Saturated (CU) tests were included in order to obtain effective strength soil properties c' and ϕ' as well as to compare these properties, the stress paths and the state boundaries with those of the unsaturated tests. This was necessary because critical state theory was developed for saturated tests and unsaturated soil behaviour is an extension of saturated soil behaviour.

Results from 23 triaxial tests were used in this study. Other tests had to be terminated or discarded due to problems like collapse during saturation and buckling during shearing.

3.4 Equipment

The following equipment and apparatus were used at various stages of the experimental project. In some cases, standard laboratory equipment was used. In others, equipment had to be modified or even re-designed and fabricated. In this section the equipment are classified as those for specimen preparation, for soil-water characteristic determination, for direct shear and for triaxial testing.

3.4.1 *Specimen preparation apparatus*

A standard assumption in the critical state theory is that the soil is homogeneous. It is thus desirable to prepare specimens having a structure as uniform as possible. A specially fabricated unit and a hydraulic laboratory press were used to prepare cylindrical soil specimens. The following describes the devices in some detail.

(a) *Graduated piston, mold and extruder*

A special piston and mold unit was designed and constructed for the preparation of specimens from soil samples. The piston shown in Figure 3.1(b) is an aluminum ram with circular notches at equally spaced intervals.

The total length, diameter and the graduated intervals on the pistons were determined based on the dimensions of the cylindrical mold (a) and the desired number of layers during specimen preparation. An acrylic cap is fitted to the end of the piston. Two semi-circular brass rings (e) are joined circumferentially by soft springs to form the

Figure 3.1 Specimen preparation unit: (a) mold (b) graduated piston (c) extruder
(d) plain base piston (e) circular brass ring

circular stopper that fits into the notches on the piston. This limits the extent to which the piston moves into the mold.

The mold (a) is simply an aluminum cylinder. A plain piston (d) remains stationary at the base of the mold during compaction. The specimens formed are removed from the mold by the extruder (c) which is a stainless steel piston and designed with a clearance such that it can be driven all the way through the cylindrical mold like a ram.

In operation, the mold is seated on the base piston, the brass stopper is placed at a desired position and the graduated piston is placed covering the top of the mold. A known volume is confined in the cylindrical mold when the top piston is stopped at some position. This arrangement improved the ease of preparing uniform specimens.

A soil layer of desired bulk density is produced when a predetermined and carefully weighed soil sample is statically compacted into a confined volume. Other identical layers are produced by adjusting the stopper position along the piston and introducing the same weight of soil. The end result is a well formed, uniform and homogeneous cylindrical soil specimen.

(b) *Hydraulic press*

A 12 tonne capacity laboratory hydraulic press (Model C, Fred S. Carver Inc., Menomonee Falls, Wisconsin) (Figure 3.2) was used to statically compact the soil into the mold. The graduated piston and mold unit is mounted between its two platens. The

Figure 3.2 Laboratory press

lower platen sits on top of a hydraulically actuated ram while the upper platen is fixed. A large easily read gauge indicates the load imposed by the two end platens in static compaction.

3.4.2 *Pressure plate apparatus*

The soil-water characteristic relationships were determined in 500 kPa and 1500 kPa pressure plate extractors as well as in a 500 kPa acrylic pressure cell. The pressure plate apparatus is essentially an air pressure chamber that accommodates high air entry disks which are kept in contact with water in a compartment to ensure saturation at all times during a test.

(a) *Pressure plate extractors*

Soil-water characteristics of the soil were measured in pressure plate extractors (Soil Moisture Equipment Company, Santa Barbara, California). The apparatus consists of high air entry ceramic disks and pressure vessels which can withstand internal air pressures up to 500 or 1500 kPa.

The ceramic disks are fitted into rubber bases that form a water compartment just below the disks. The vessel is connected to an external compressed air pressure line and also has external ducts that connect the disks and the water compartment to a water monitoring device. A rubber O-ring is embedded in the top lid of the vessel to ensure proper seal when fastened into place by the bolts and nuts.

A null pressure plate apparatus designed at the Civil Engineering Department, University of Saskatchewan was used to measure the initial matric suction of specimens of identical structure to those used in the soil-water characteristics and triaxial tests.

(b) University of Saskatchewan pressure cell

The acrylic cell (Figure 3.3) was fabricated in the Engineering Shops at University of Saskatchewan. It can withstand pressures up to about 700 kPa. Any of a series of high air entry disks rated at 100, 300 and 500 kPa can be used in the cell. A grooved water compartment is located below the disk to keep it saturated and to flush out dissolved air.

3.4.3 Direct shear apparatus

A direct shear apparatus (model WF 25317, Wykeham Farrance Engineering Limited, Bucks, England) was used for the direct shear tests. The specimen was held in a 100 x 100 mm split box while normal load was applied via a hanger with a loading arm ratio of 10:1. Horizontal load was measured with a 454 kg load cell (Interface Inc., Scottsdale, Arizona). The apparatus has five gear settings and can drive the carriage at speeds ranging between 0.00048 to 1.2 mm/min. A Campbell 21X Micrologger (Campbell Scientific Inc., Logan, Utah) and software served as the data acquisition device.

Figure 3.3 University of Saskatchewan pressure cell.

3.4.4 Triaxial testing system and modifications

The main experiments in this study were conducted with a computer controlled triaxial testing system. The equipment manufactured by GDS Instruments Ltd. Surrey England, is a 1 Hz frequency system capable of static as well as dynamic tests. The testing system shown in Figure 3.4 is made up of :

- a. Digital pressure-volume controller
- b. Host computer and control software
- c. Data acquisition and signal conditioner
- d. Cabinet mounted hydraulic triaxial cell
- e. Plumbing system (built as an auxiliary)

The system was developed to specific configurations with respect to loading rates, specimen size and displacement limits, but the plumbing ports on the cell and controlling software were originally configured for testing saturated soils. Before embarking on the testing program, a series of modifications was undertaken to components of the system to enable testing of unsaturated soil specimens. The modifications were mainly to accommodate the monitoring of pore-air pressure and volume changes.

The following describes components of the system and the modifications.

Figure 3.4 Triaxial testing system (a) digital pressure-volume controller (b) host computer (c) signal conditioner and data acquisition unit (d) cabinet-mounted triaxial apparatus (e) plumbing system

3.4.4.1 Modified Triaxial cell

The hydraulic cell is based on the design of Bishop and Wesley (1975). The top is an acrylic cylinder with circular reinforcements. It can withstand pressures up to 1700 kPa. Figure 3.5(i) shows the cell top in place and while Figure 3.5(ii) shows inner details with the top removed. Vertical steel rod screws are used to fasten cell top and base ring.

Modifications to the plumbing on the triaxial cell are also shown. The cell base ring originally came with three ports (b, c and d) that connect the cell and specimen to external pressure lines and volume change devices. An extra port (a), was built into the base ring. Port (b) was adapted to measure pore-water pressure and to allow flushing of the water compartment on top of the pedestal. Port (c) was originally built for water backpressure but is now used as an air backpressure port. Figure 3.6 shows a schematic diagram of the modified cell.

3.4.4.2 Modified base pedestal and loading ram

A base pedestal supports the specimen during testing. It fits onto a hydraulically driven loading ram which moves vertically up and down in a linear guide. The ram has a total stroke of 100 mm and is balanced (water volume equivalent to the volume of ram displacement in or out of the cell is removed or added to cell water) so that the net volume change caused by its movement into and out of the cell is zero. This ensures that cell pressure is not disturbed and axial load capacity is not reduced by cell pressure.

The movement of the ram can be controlled interactively by a set of function keys on the computer keyboard, in order to bring the pedestal to a particular position. It is also

(i) closed

(ii) opened

Figure 3.5 Modified plumbing on the triaxial cell (a) pore-water pressure control and volume change measurement (b) pore-water measurement and flushing line (c) pore-air pressure control and volume change measurement (d) cell pressure line.

Figure 3.6 Modified hydraulic triaxial cell (schematic).

computer controlled during some tests; for example it can be programmed to move in a step, a ramp or in a cyclic manner. Axial deformation is measured at the encoder of the axial actuator which pushes the ram through the base of the cell.

A new pedestal (Figure 3.7) was designed for use in this study since the original one was suitable for saturated soil testing only. The new pedestal has two drainage holes connected on top by a spiral groove to allow periodic flushing of diffused air in the water compartment below the high air entry disk and atop the pedestal. These drainage holes also connect the water compartment to a pore-water pressure transducer (at port b) and an external pressure and volume change ports (port a in Figure 3.5).

3.4.4.3 Porous stones and rings and top cap

High air entry disks were fixed rigidly into flanged and threaded aluminum rings. (Figure 3.8). The modified pedestal can take up any of these mounted disks. Fine ceramic stones with rated air entry values of 200, 300 and 500 kPa were available in the laboratory. These stones allow water to flow through but prevent air flow as long as the rated air entry value is not exceeded. Special grooves machined around the circumference of the aluminum rings (Figure 3.8), allow the O-rings to seat better into the aluminum ring and membrane.

A water compartment is formed between the top of the pedestal and bottom of the high air entry disk. This arrangement keeps the stone saturated, while allowing both

Figure 3.7 Modified base pedestal

Figure 3.8 Mounted porous stones (a) top corundum disk (b) high air entry disk.

control or measurement of pore-water pressure as well as pore-water volume change measurements. Dissolved air (indicated by bubbles) is flushed out once a day from the water compartment during tests.

While the soil pore-water space is connected to external pore-water lines through the water compartment, the pore-air space is connected via a top cap made of a coarse corundum disk of low air entry value (Figure 3.8). This allows free flow of air between the soil pore-air space and external air volume and pressure change unit. The disk is mounted in an aluminum ring with a semi-spherical indentation which takes up the load cell button. Grooves similar to those on the high air entry units were machined on the aluminum ring in order to allow better seating of the top rubber O-rings.

3.4.4.4 Transducers, load cell and calibrations

Two diaphragm type pressure transducers (Applied Measurements Limited, Berkshire, England) were used to measure pore-water and cell chamber pressures. The calibration information for these transducers at 24°C as supplied by GDS Instruments Inc. (and verified before the experiments) are shown in Table 3.1.

Table 3.1 Calibration of pressure transducers.

<i>Transducer</i>	<i>Pore pressure</i>	<i>Cell pressure</i>
Operation mode	Gauge	Gauge
Pressure range (kPa)	3447.5	3447.5
Excitation voltage (Vdc)	10.0	10.0
Full range output (mV)	197.4	200.0
Sensitivity (kPa/mV)	17.5	17.2
Thermal zero shift (kPa/°C)	< 0.018 % FRO	< 0.018 % FRO
Thermal sensitivity shift	< +/- 0.018 % rdg. / °C	< +/- 0.018 % rdg. / °C
Non-linearity, hysteresis and non-repeatability	+/- 0.08 % FRO	+/- 0.07 % FRO

rdg. = reading or measured value FRO = full range output

A zero offset routine was run each time zero drifts (due to temperature change) were observed on the transducers or the load cell. Values obtained during such runs were saved and subsequently applied to the readings from these devices later on. This routine was particularly useful as transducer zeroes varied slightly with temperature. Details of the implementation of this routine are given by Menzies and Hooker (1993).

A submersible load cell (Maywood Instruments Limited, Basingstoke, England) was used to measure the load applied to the specimen during vertical loading tests. It was mounted at the end of a shaft fitted centrally into the top of the hydraulic cell (Figure 3.6). The position of the load cell at the upper part of the cell can be changed by an adjustable screw. A lock nut is used to keep it in position before commencement of tests. The load cell end is threaded to take up a button shaped to fit into the top cap placed on the specimen. Verified calibration data of this load cell (as supplied by the manufacturers) at 21°C is shown in Table 3.2.

Table 3.2 Calibration of load cell.

Load range (kN)	5.00
Excitation voltage (Vdc)	10.00
Full range output, FRO (mV)	26.32
Sensitivity (kN/mV)	0.19
Non-linearity	< +/- 0.05% FRO
Hysteresis	< +/- 0.05% FRO

FRO = full range output

Because of the large axial deformations experienced during trial tests on the soil, it became necessary to increase the reach of the load cell button in order to maximize the stroke of the driving ram. Hence extensors were built to increase the length between the button and the load cell body.

3.4.4.5 Plumbing unit

A wall mounted unit comprising of gauges, valve switches, regulators, reservoirs, Swagelok fittings and tubings constitute the plumbing system. This served as an external auxiliary unit to supply water and pressure to the cell.

3.4.4.6 Digital pressure-volume controller

The controller is a programmable multi-purpose device (Figure 3.9) designed to measure and control pressure and volume changes of fluids. Its 200 ml steel cylinder can withstand pressures up to 2000 kPa. An integral pressure transducer measures the cylinder pressure. The device is described in detail by Menzies (1988) and Menzies and Hooker (1993).

The cylinder was originally designed for liquids like water and oil. In this research program, the device was used to measure and control pore-air pressure and volume changes. Before using the device for this purpose however, an extensive testing program was undertaken to evaluate the device using air as the working fluid. The cylinder was also insulated to minimize temperature fluctuations.

Figure 3.9 Digital pressure-volume controller

Results of this evaluation showed that with adequate precautions like insulating the steel cylinder, minimizing and pre-pressurizing the control volume, the device performed very well as an air pressure and volume change device (Adams *et al.* 1996).

3.4.4.7 Computer control algorithm and data acquisition unit

Electronic software and hardware

A BASIC (HTBasic, TransEra Corporation, Provo, Utah) programme MINIDYN (GDS Instruments 1993) served as the control software for the system. However with the modification of the system, the software was reprogrammed and called MINIDYN2. A new subroutine called AVM was also written in HT BASIC to control and measure air volume changes during isotropic compression.

The controlling software was run from an Intel 486/33 DX computer. Data from the load cell and pressure transducers were collected and saved directly to the computer hard drive via a transducer interface as the test proceeded. An IEEE interface logged volume and pressure change readings from the digital pressure-volume controller.

During testing, several variables like axial, deviatoric and mean net stress and matrix suction were continuously computed. An on-line graph of two pre-selected variables (e.g., axial strain versus deviator stress) was displayed on the computer monitor, while the other variables were numerically displayed alongside. This allowed a continuous visual evaluation of the on-going experiment.

3.5 Procedures

The experimental program involved a series of experiments. Following are the description of the methodology involved for the various experiments. Standardized soil laboratory testing techniques were closely adhered to.

3.5.1 *Determination of soil index properties*

Soil type and engineering classification

The soil used in this study is a topsoil. Particle size analysis was done using the ASTM D 422-63 (1993) procedure. Atterberg limits (i.e., liquid and plastic) were determined using the ASTM D 4318-84 (1993) procedures. Low compactive effort (about 500 kPa per soil layer) was used in the compaction test to determine the optimum water content and optimum dry density.

3.5.2 *Batch sample preparation*

Soil obtained from the experimental soil bin at the Agricultural Engineering Department, University of Saskatchewan was air-dried and passed through a US Standard sieve series #10 with 2 mm opening. Distilled water was sprayed as fine droplets and mixed thoroughly with the soil. It was then placed in sealed polythene and placed in a controlled environment (kept at about 10°C and relative humidity of 80%) for 3 to 5 days for preservation and equalization of moisture. Batch samples were prepared at two water contents; dry of optimum (i.e., 16%) and wet of optimum (i.e., 20.5%). Specimens for the uniaxial compression tests, soil-water characteristic determination and triaxial tests were prepared from these batches using same compactive effort as in the compaction test.

3.5.3 *Soil-water characteristic determination*

Soil-water retention capacity was determined in pressure plate extractors and a pressure cell following the ASTM D 2325-68 (1993) procedure. Soil specimens of 70 mm diameter and about 10 mm thickness were placed on previously saturated high air entry disks in the pressure plate extractors. Similar specimens were prepared and trimmed to fit the disk in the pressure cell. The specimens were inundated over several hours to saturation. The pressure vessel was sealed after excess water had been syringed off the high air entry disks. Desired air pressures were imposed in the vessels. Water flow from the specimens were monitored until equalization of air pressures in the vessel and pore-air space (hence equalization of soil matric suction). This happened when the water flow from the specimens became very small or insignificant. The specimens were then quickly removed from the disks and weighed before placement in the oven for water content determination.

3.5.4 *Direct shear tests*

Several batches of soil samples were prepared at five different water contents ranging from 3% to 21.1% dry basis using the same procedure described in section 3.5.2. Soil sample from a batch was placed in the square (100 mm x 100 mm) specimen holder such that a specimen with bulk density of about 1.2 Mg/m^3 was formed after placement of the loading cap. A pre-set normal load was then applied via the loading arm to the specimen loading cap. The vertical normal stress used in these experiments ranged

between 25 to 150 kPa. Horizontal load was applied at a strain rate of 0.9 mm/min until the specimen failed along the shear plane. Experiments were also conducted at 0.0072, 0.036, 0.06 and 0.4 mm/min in order to investigate the effect of strain rate on shear stress and the failure pattern.

Data collected were analyzed to determine the frictional properties of the soil. The analyses also included the effects of strain rate and water content on shear strength.

3.5.5 Uniaxial compression tests

Cylindrical specimens were formed and left in a mold after creating a structure similar to that of the triaxial and pressure plate specimens. The specimen was sealed at the top and bottom by low air entry value porous stones to allow dissipation of pore pressure during the application of an external load. Having the specimen in the cylindrical mold ensured that soil lateral movement was restrained. A loading cap was placed on top of the soil. The whole specimen while still in confinement within the mold was transferred and mounted onto the triaxial pedestal. The load cell button was engaged to the specimen load cap and a ramped vertical loading test was carried out. A loading rate of 0.04 kN/min was applied to deform the soil beyond its pre-compaction pressure along its virgin consolidation line.

3.5.6 *Triaxial testing*

The following steps were taken during the main experiments. In a typical test, a cylindrical soil specimen was prepared and placed in the triaxial cell. Predetermined stresses were imposed at the initial state. It was then loaded to failure. Data were collected and saved on computer disk for analysis.

(a) *Preparation of triaxial specimen*

Five predetermined equal portions of soil sample were weighed and placed into the mold one at a time. A triaxial test specimen was formed by statically compacting the soil sample in five identical layers in order to obtain a homogeneous specimen. With this procedure, cylindrical specimens of 70 mm diameter, 140 mm height and about 645g wet mass were formed.

Values of typical oven dry mass, wet mass and soil specific gravity were used to estimate initial volumetric state of the specimen (Table 3.3). Identical soil specimens were formed by static compaction at a bulk density of 1.2 Mg/m^3 from the same batch of sample. Each triaxial specimen was stored in an air-tight bag for about 2 hours (in order to dissipate any excess pressure that may have built up during the compaction process) before setting it up in the cell.

Triaxial specimens formed had similar structures to those used in the null pressure plate apparatus for determining the initial matric suction of specimen, and to those used for the soil-water characteristic determination. An average value of 39 kPa was measured

as the initial matric suction for specimens compacted wet of optimum (20.2%) and about 285 kPa for specimens compacted dry of optimum (16.1%).

(b) Initialization of air entry disks

Prior to commencement of each test, the top corundum disk was cleaned and dried. The high air entry disks were kept in de-aired water over 24 hours for saturation. After mounting it in place over the pedestal, the disk was further de-aired by flushing the water compartment with a water pressure of about 50 kPa several times until no bubbles were seen in the water flowing out. The 50 kPa water pressure was then maintained in the water compartment for about two hours. The pore-water lines were also thoroughly de-aired. The transducers were open to the atmosphere and initialized, off-sets were calculated and saved for the correction of future readings. Thin layers of petroleum grease were applied to the surface of the aluminum rings around the grooves in order to create a better seal between the membrane and grooves.

(c) Initialization of the digital pressure-volume controller

The device was switched on at least one hour before any test began in accordance with the manufacturer's recommendation. This allowed a stabilization of the device prior to use. The outlet line of the cylinder was disconnected from the triaxial cell. The cylinder was then filled with air and opened to the atmosphere. Filling the cylinder with air placed the piston at the beginning of its (full volume) stroke such that movement in the opposite direction will allow the pressure of confined air within the cylinder to rise.

Table 3.3 Initial (as-compacted) state of triaxial specimens.

Quantity	Specimens wet of optimum	Specimens dry of optimum
Wet mass ($\times 10^{-3}$ kg)	647.8	646.6
*Bulk density (Mg/m^3)	1.2	1.2
*Dry density (Mg/m^3)	1.0	1.02
*Compaction water content (%)	20.2	16.1
Total volume ($\times 10^{-6}$ m ³)	538.8	538.8
Void volume ($\times 10^{-6}$ m ³)	337.1	328.3
Water volume ($\times 10^{-6}$ m ³)	109.5	89.2
Air volume ($\times 10^{-6}$ m ³)	227.6	239.1
Soil volume ($\times 10^{-6}$ m ³)	201.6	210.5
*Void ratio	1.67	1.56
*Degree of saturation (%)	32.5	27.2

- * These values of these quantities here are similar to those of the specimens used in :
- (i) the determination of soil-water characteristic
 - (ii) measurement of the initial suction of compacted specimens
 - (iii) uniaxial compression tests.

(d) Set up of triaxial specimen

The high air entry disk was mopped using moist paper before the specimen was placed. The specimen was carefully removed from the air tight bag and weighed. It was handled very gently and carefully placed on the high air entry disk.

A stretcher-cum-suction tube was used to place rubber membranes around the specimen. This resulted in a specimen that was totally sealed by a composite membrane made up of two rubber membranes sandwiched by a slotted aluminum foil and grease. Six rubber O-rings were snapped into place around the composite membrane, three into the top cap grooves and three into the high air entry disk aluminum ring grooves.

The pore-air line was connected to the top cap and the cell top was placed and fastened into place. De-aired water was introduced into the cell around the specimen. At some time during the preparation of the specimen, the water in the cell water tank was de-aired for about one hour using a vacuum pump. For saturated tests, the specimens were saturated by applying a backpressure of de-aired water as described in section 2.4.1.

(e) Axis-translation and isotropic compression

After setting up the specimen, the digital pressure-volume controller was connected to the pore-air line, while an external burette was connected to the pore-water line from the water compartment. At this stage, the pore-air pressure was atmospheric while the pore-water started to go negative. The axis-translation technique was used to restore the pore-water pressure to atmospheric pressure by using the digital pressure-

volume controller to gradually elevate the specimen's air backpressure to a predetermined value. Continuity of soil pore-water and water in the saturated disk was thus ensured.

Ballooning of the membrane was prevented by keeping the elevation of air backpressure in unison with an increase of the cell water pressure. This was carefully carried out so that a premature compression of the specimen was avoided. Air backpressure from the digital pressure-volume controller was strategically applied such that the piston in its cylinder ended up at close to the end (minimal or near zero volume) stroke. This procedure was recommended by Adams *et al.* (1996) for when the digital pressure-volume controller is to be used for air volume change measurement.

This brought the specimen to a net confining pressure ($\sigma_3 - u_a$) of 0 kPa and a matric suction ($u_a - u_w$) equal to the air backpressure value imposed from the digital pressure-volume controller. The external valves to ports (a), (c) and (b) (Figure 3.6) were shut off.

The desired net confining pressure which would be used to isotropically compress the specimen was pre-set by increasing the cell pressure at the plumbing system. After the set pressures had stabilized at or near desired values, the pre-set net confining pressure was imposed by opening valve (b). Valves (b) and (c) were opened in quick succession. The experimental mean net stress ranged from 0 to 350 kPa while matric suction ranged between 0 to 300 kPa. An increase in air backpressure at the digital pressure-volume controller was immediately observed. This was dissipated and translated to air volume changes. The air volume measurement subroutine AVM was run within the main

programme MINIDYN2 to maintain a constant air backpressure in the specimen and to collect air volume change data.

Pore-water volume changes were noted at the burette. Periodic flushing (once a day) was carried out to rid the water compartment of air bubbles during isotropic compression.

(f) *Shearing of specimen*

Following the compression stage, the new specimen dimensions were calculated using equation 3.7 (GDS Instruments Limited 1993). These were back substituted into the controlling programme MINIDYN2 for computations during the shearing stage.

$$\begin{aligned} L_c &= L_o \cdot z \\ D_c &= D_o \cdot z \end{aligned} \tag{3.4}$$

where,

$$z = \left(1 + \frac{\Delta V}{V_o} \right)^{\frac{1}{3}}$$

L_c = new specimen length after isotropic compression

D_c = new specimen diameter after isotropic compression

V_o = initial volume of specimen

ΔV = total volume change of specimen after isotropic compression

Before commencing the shearing stage, the baseplate was thoroughly flushed; however, diffused air volume was not measured. The load cell was first moved downward manually such that the button came very close to the specimen top cap. A docking subroutine in the control software MINIDYN2 was used to gradually lower and seat the button gently into the top cap.

At the prompt of the control program, the specimen's new dimensions, the type of loading test (in this case ramp deformation) and an estimated terminal deformation were entered at the computer keyboard. The shearing stage involved one of three kinds of triaxial tests namely, constant water (CW) and drained (CD) tests for the unsaturated specimens and undrained (CU) tests for the saturated tests.

Constant water content tests were carried out at a ramp deformation of 10 mm/h, while fully drained constant suction tests were at a rate of 2 mm/h for the unsaturated specimens. The saturated specimens were failed undrained at 2 mm/h.

The specimens were taken to large deformations in order to get them to, or close to critical state. During the shearing phase, on-line graphical computation of deviator stress versus axial strain was displayed on the computer monitor so as to observe the approach to failure and critical state.

After the failure of the specimen, the pressures were relaxed starting with u_a , then σ_3 and u_w . This order ensured that u_a is less than σ_3 so that the membrane does not “blow out”. The cell was opened to the atmosphere and drained. The specimen was removed, carefully stripped of the composite membrane, weighed and transferred to the oven for determination of the mass-volume relationships.

(g) *Data collection and analysis*

Raw data collected from all the devices were saved on the computer hard drive in binary form. These were processed during the test for on-line display of numeric values and a graphical plot.

The experimental data which were saved on the hard drive in binary format were previewed by plotting several pairs of variables. The values and plots obtained from this initial analyses were noted and compared later with those from a spreadsheet program. The binary data were converted to the ASCII format and then imported to the EXCEL spreadsheets program.

An EXCEL template was created to calculate desired variables from the experimental data. Select graphs from these variables were also plotted. The spreadsheet calculations and graphs were cross-checked with those obtained in the initial analyses.

CHAPTER 4

ANALYSIS AND PRESENTATION OF RESULTS

Overview

Results from the experimental program are presented in this chapter. Data and analysis for each series of tests are presented. In the next chapter, the test results will be compared and discussed more thoroughly and in conjunction with findings from other studies.

Data from the supporting experiments are presented first and analyzed. Basic soil properties and engineering classification as well as some mechanical properties of the soil are presented followed by the results from soil-water characteristic determination, direct shear and uniaxial compression tests.

Results from the main triaxial testing program are presented separately for the unsaturated and saturated test series (Figure 4.1). The unsaturated series are further subdivided as results from series compacted wet and dry of optimum.

Two broad methods are used in data analysis of these tests,; namely, (i) the traditional or classical approach and (ii) the Cambridge critical state soil mechanics approach. The relationships between state variables are established and appropriate soil properties obtained for both approaches.

CSSM - Critical state soil mechanics

Figure 4.1 Format for the presentation of results from triaxial testing program.

4.1 Results from supporting experiments

4.1.1 *Engineering classification of soil*

The soil used in the experimental program with 48.1% sand, 23.6% silt and 28.3% clay, is classified as a sandy clay loam based on the USDA texture triangle (McKyes 1989). Table 4.1 shows other engineering classification parameters for the soil.

4.1.2 *Soil-water characteristics*

Figure 4.2 shows the relationship between the gravimetric water content and matric suction for specimens compacted from two sample batches. A curve fitting program CFVIEW (developed by the Geotechnical Division, Civil Engineering Department, University of Saskatchewan) according to equations described by Fredlund and Xing (1994) was used to fit the experimental soil-water characteristic data.

The specimens prepared at 20% water content (wet of optimum) produced a structure that retained slightly more water as desaturation commenced under the imposed matric suction when compared with the structure from the specimens prepared at 16% (dry of optimum). This is probably because the initially higher void ratio produced after compaction (Table 3.3) held more water within the pores. However, the variation of degree of saturation with matric suction is essentially identical for the two different structures (Figure 4.3) despite the difference in the “as compacted” degree of saturation (Table 3.3).

Table 4.1 Engineering classification of soil

Parameter	Value
<i>Particle analysis</i>	
Sand	48.1%
Silt	23.6%
Clay	28.3%
Specific gravity	2.65
<i>Atterberg consistency limits</i>	
Liquid limit, water content	32.9%
Plastic limit, water content	18.8%
Plasticity index	14.1%
<i>Optimum compaction state</i>	
Water content	18%
Dry bulk density (Mg/m^3)	1.65

Figure 4.2 Soil-water characteristic curves (reference to gravimetric water content) for two soil structures.

Figure 4.3 Soil-water characteristic curves (reference to degree of saturation) for two soil structures.

This observation indicates that the two structures retained similar proportions of water in the pores with respect to the total pore volume as the specimens desaturated.

Figures 4.2 and 4.3 also show that the two different structures produced pore volumes having a low air entry value. Desaturation commenced almost immediately as matric suctions as low as 0.5 kPa were applied. This could be due to the loose state of the specimens (a bulk density of 1.2 Mg/m^3) and the relatively high fraction of sand in the soil (i.e., 48.1%).

4.1.3 *Direct shear box tests*

Typical shear stress versus displacement curves for the soil under various normal loads are shown in Figure 4.4. The trend fits the general pattern of elasto-plastic behaviour observed for agricultural soils. The initially rapid rise in shear stress with deformation gradually reduced and approached steady values as the soil reached the state of plastic flow.

Figure 4.5 shows the variation of shear strength at five different water contents ranging from 3% to 21.1% dry basis, under various vertical (normal) loading conditions. For lower normal stresses (i.e., 25 to 75 kPa), there was an increase in shear strength with decreasing water content from 21.1% (wet of optimum) to 14.6% (dry of optimum). Beyond this however, the shear stress remained fairly constant. At higher normal stresses (100 to 150 kPa), shear strength continued to increase with reductions in moisture content from 21.1% to 14.6%, but a decrease in strength was observed with further reduction of

Figure 4.4 Typical shear stress versus displacement curves from direct shear tests on specimens at water content of 9.2%.

Figure 4.5 Variation of shear strength with water content in direct shear tests.

water content below 9.2%. This might be because the combination of high normal stress with horizontal shear stress on the soil fractured the dry grains.

Noticeably lower shear strengths were developed for specimens prepared wet of optimum at 21.1% compared with all other specimens (i.e., dry of optimum) as shown in Figure 4.6. The relationship between shear strength and normal stress became non-linear at higher normal stresses. This was markedly different from the linear relationships obtained for the drier specimens. A linear regression of the data from the specimens prepared at water contents of 9.2% and 16.9% yielded soil cohesion values (c) of 5.3 kPa and 6.2 kPa and angles of internal friction (ϕ) of 37.2° and 36.9° respectively.

The horizontal deformation rate did not cause significant changes in the shear strength developed during the experiments for the range of speeds tested. Figure 4.7 shows that the shear strength developed at a deformation rate of 0.0072 mm/min did not change significantly when the rate was increased 5 times to 0.036 mm/min and 125 times to 0.9 mm/min. With the speed of 0.9 mm/min as a reference, the strength developed at the two other speeds were all within the 10% error bars. The strain at which failure occurred also appear identical.

Figure 4.6 Shear stress versus normal stress in direct shear tests.

Figure 4.7 Effect of deformation rate on stress-displacement relationship in direct shear tests on specimens at a water content of 14.6% and normal stress of 100 kPa.

4.1.4 Uniaxial compression tests

Since both the specimens (i.e., wet and dry of optimum) were compacted to the same bulk density, the drier specimen had slightly more grains per unit volume and a lower initial void ratio (Table 3.3). Figure 4.8 shows that the drier specimen with a water content of 16% has a pre-compaction pressure (i.e., the stress beyond which specimen starts to undergo normal compression) of about 48 kPa and the specimen at 20% water content exceeds its pre-compaction pressure at about 28 kPa.

The two structures attained identical voids ratios at around an axial stress of 40 kPa (Figure 4.8). The wet specimen was already on its normal compression line at this pressure while the dry specimen was yet to undergo normal compression. Beyond this stress, the wetter specimen deformed more than the drier one. However the compressibilities (i.e., change in void ratio per unit increase in axial stress) for the two structures appear identical, as indicated by the slopes of the graphs in Figure 4.9. The change in void ratio with respect to the applied stress, was more rapid at the beginning of compression for axial stresses up to about 10 kPa, after which it approaches a steady value at around 100 kPa for both structures. Figure 4.9 shows that the two structures responded in an identical manner to applied stress. This observation will be used to explain a similar trend in the triaxial tests.

Figure 4.8 Uniaxial compression of specimens compacted wet and dry of optimum.

Figure 4.9 Void ratio change with axial stress in uniaxial compression for specimens compacted wet and dry of optimum.

4.2 Results from the Main Experiments Presented Under the Unsaturated Soil Mechanics Framework

4.2.1 *Triaxial test results for specimens compacted wet of optimum*

4.2.1.1 Isotropic compression

Generally, the first stage of the triaxial tests was isotropic compression. Table 4.2 gives the states of fifteen specimens after isotropic compression under initial net confining stresses ranging between 0 to 250 kPa, and a matric suction of around 50 kPa. All specimens were initially in loose states as shown by the values in Table 3.3 and the first row of Table 4.2. Eleven of the specimens were used for constant water content (CW) tests while the remaining four specimens were used for consolidated-drained (CD) constant matric suction tests.

Pore-air volume changes accounted for nearly all the total volume changes in the specimens. Pore-water volume changes were relatively small. This was not unexpected for the specimens with a low initial degree of saturation (i.e., around 32%).

Large changes in the soil pore-air space were produced by the isotropic compression of the specimens. These changes can be associated with the initially loose state and low degree of saturation of the specimens. For example, the gravimetric water content remained fairly constant but a 45% increase in the degree of saturation was observed under an isotropic net confining stress of 60 kPa, from 32% to 46.3%, and the

Table 4.2 State of the triaxial specimens compacted wet of optimum after isotropic compression (classical analysis)

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	ΔV_a (ml)	ΔV_w (ml)	w (%)	e	S_r (%)	ρ (Mg/m ³)
Initial	0	0	0	0	20.1	1.65	32.3	1.2
F0104-CW	1.5	50.8	0.0	0.0	20.2	1.65	32.5	1.20
F0828-CW	8.7	49.9	-16.1	0.0	20.3	1.57	34.2	1.24
F0718-CW	18.8	49.9	-29.9	0.0	20.2	1.50	35.6	1.27
F0721-CW	29.5	49.1	-57.2	-0.2	20.1	1.37	38.9	1.34
F0725-CW	37.6	49.4	-80.4	-0.2	20.1	1.25	42.4	1.41
F0920-CW	48.0	51.0	-88.7	-0.3	20.0	1.21	43.9	1.44
F0727-CW	60.4	50.4	-101.4	-0.3	20.0	1.15	46.3	1.48
F1102-CW	150	50.8	-135.2	-4.1	19.7	0.98	53.3	1.60
F1110-CW	250	50.6	-162.0	-7.9	19.0	0.83	60.7	1.72
G0927-CW	52.8	100.1	-73.9	-2.6	20.0	1.29	41.3	1.39
H0932-CW	51.7	150.1	-86.4	-5.4	19.5	1.20	43.1	1.44
J0405-CD	0	49.6	0.0	0.0	19.9	1.65	32.0	1.20
J0821-CD	25.3	49.1	-48.5	-0.2	20.3	1.42	37.0	1.30
J0430-CD	46.5	50.4	-88.4	-0.8	20.3	1.22	44.0	1.44
J0704-CD	67.6	49.6	-110.1	-2.0	20.1	1.11	48.1	1.51

bulk density increased 25% from 1.2 Mg/m^3 to 1.5 Mg/m^3 . The rapid increase in the degree of saturation is because the confining stresses are sufficient to expel large volumes of air from the loose specimens, but not sufficient to expel substantial quantity of pore-water. The higher rate of increase in degree of saturation as compared with the bulk density indicates that the externally applied stresses caused larger changes within the soil pores than against the bulk of the soil.

Specimens compressed at higher net confining stresses (150 to 250 kPa) had higher increases in the degree of saturation and bulk density. For example, the specimen at a net confining stress of 250 kPa had undergone an almost 90% rise in the degree of saturation despite losing more water than any other specimen. The bulk density at this time had increased 43% from 1.2 to 1.72 Mg/m^3 . A dense structure was produced under this condition as evident by the low void ratio of 0.83 (i.e., reduced by almost 50% from the initial state of 1.65).

4.2.1.2 Constant water content (CW) tests for specimens compacted wet of optimum

Constant water content tests were carried out on eleven specimens prepared wet of optimum, nine under initial net confining stress range of 1.5 to 60 kPa and two in the range of 150 to 250 kPa. The separation into these two ranges became necessary in view of the observed differences in the results during the shearing phase of the tests.

Figure 4.10 shows typical stress-strain relationships during constant water content tests. A general three-stage pattern was observed in the changes in deviatoric stress with axial strain. A rapid rise in deviatoric stress over a short axial strain range was quickly followed by a fairly wide range of constant rate of change in deviatoric stress with axial strain. These transition stages indicate the progressive mobilization of the shear strength of the specimens. The third stage shows a greatly reduced (almost steady) change in deviator stress with axial strain as the specimen approached critical state. This pattern was observed for the change in the mean net stress. There was little change in the matric suction during tests under net confinement of 1.5 to 60 kPa. Volume changes were due to the expulsion of air during the shearing phase and volumetric strains were relatively large.

Figures 4.11 through 4.14 show the stress-strain variations of the seven constant water content tests conducted under net confining pressures between 1.5 kPa to 60 kPa. Most of the specimens approached failure states at around 40% axial strain. The specimen at a net confining pressure of 8.9 kPa, however, had not attained a steady deviator stress even at an axial strain of 50% (Figure 4.11).

Generally, maximum deviator and mean net stresses (Figures 4.11 and 4.12) were realized after large axial deformations because of the initially loose states of the specimens.

Figure 4.10 Typical stress-strain relationships for specimens compacted wet of optimum in constant water content test.

Figure 4.11 Deviator stress-strain relationships for specimens compacted wet of optimum in constant water content tests
(net confinement of 1.5 to 60 kPa).

Figure 4.12 Mean net stress-strain relationships for specimens compacted wet of optimum in constant water content tests (net confinement of 1.5 to 60 kPa).

Figure 4.13 Volumetric-axial strain relationships for specimens compacted wet of optimum in constant water content tests (net confinement of 1.5 to 60 kPa).

Figure 4.14 Variation of matric suction with axial strain for specimens compacted wet of optimum in constant water content tests (net confinement of 1.5 to 60 kPa).

The observed volumetric strains fell between 20% to 25% for six of the specimens (Figure 4.13) the exception being the specimen with net confinement of 1.5 kPa. Because of the low confining stress, the specimen failed immediately on applying the deviator stress with very little volume change. The matric suction varied between 43 kPa to 52 kPa (Figure 4.14 and Table 4.3), which represents small changes (within 14%) from the initial values of around 50 kPa.

Figures 4.15 through 4.18 show the stress-strain variations for two constant water content tests conducted under net confining pressures ranging between 150 to 250 kPa. Failure conditions were approached at about 25% axial strain (Figures 4.15 and 4.16). The volumetric strains shown in Figure 4.17 (between 1% to 7.5%) were not as large as the previous series of seven constant water content tests. Figure 4.18 shows the variation of matric suction with axial strain. The specimens in this series not only reached a matric suction of zero, but proceeded to develop positive pore-water pressures. Pore-water pressures had risen up to and above the pore-air pressure which was maintained at 50 kPa throughout the test. Positive pore-water pressures are generally associated with saturated soils, but these specimens had only attained degrees of saturation of about 63% (Table 4.3). This phenomenon will be explained in the next chapter.

Figure 4.19 shows the failure envelope for the seven constant water content specimens compacted wet of optimum under net confinement of 1.5 to 60 kPa. It should be noted that these specimens failed at similar values of matric suctions. Based on this series of constant water content tests, the soil has a total cohesion c of about 20 kPa and an internal angle of friction ϕ' of 35°.

Table 4.3 States of triaxial specimens compacted wet of optimum at failure.

Test #	$\sigma_3 - u_a$ (kPa)	$\sigma_1 - u_a$ (kPa)	$u_a - u_w$ (kPa)	ΔV_a (ml)	ΔV_w (ml)	e	S_r (%)
F0104-CW	1.5	20.6	50.5	0	-	1.65	32.5
F0828-CW	8.9	102.7	45.6	-127.7	-	0.98	56.5
F0718-CW	18.9	138.3	47.7	-115.5	-	0.93	56.0
F0721-CW	29.2	190.7	52.4	-111.3	-	0.82	63.5
F0725-CW	37.8	208.0	44.4	-107.8	-	0.72	72.3
F0920-CW	48	237.6	43.3	-99.3	-	0.71	75.3
F0727-CW	60.6	296.6	43.6	-93.8	-	0.69	76.2
F1102-CW	149	416.9	+5.0*	-29.3	-	0.87	60.0
F1110-CW	249	603.0	+23.3*	-4.5	-	0.80	62.7
G0927-CW	49	256	86.4	-101.9	-	0.78	66.9
H0932-CW	48	274.7	128.3	-98.7	-	0.71	70.7
J0405-CD	1.2	14.3	50.0	0	0	1.65	31.8
J0821-CD	25.3	165.3	49.7	-110.6	-0.6	0.88	61.2
J0430-CD	46.5	219.6	52.5	-80.4	-0.3	0.82	63.6
J0704-CD	67.6	316.9	48.7	-80.5	2.0	0.71	74.4

* Positive pore-water pressure

Figure 4.15 Deviator stress-strain relationships for specimens compacted wet of optimum in constant water content tests (net confinement of 150 to 250 kPa).

Figure 4.16 Mean net stress-strain relationships for specimens compacted wet of optimum in constant water content tests (net confinement of 150 to 250 kPa).

Figure 4.17 Volumetric-axial strain relationships for specimens compacted wet of optimum in constant water content tests (net confinement of 150 to 250 kPa).

Figure 4.18 Variation of matrix suction with axial strain for specimens compacted wet of optimum in constant water content tests (net confinement of 150 to 250 kPa).

Figure 4.19 Mohr-Coulomb failure envelope for specimens compacted wet of optimum in constant water content tests.

4.2.1.3 Fully drained constant suction tests for specimens compacted wet of optimum

In this series of tests, constant suction was maintained by keeping both the air backpressure and the pore-water pressure constant. The deviatoric and mean net stress changes with axial strain for the constant suction tests (Figures 4.20 and 4.21) were similar in pattern to the constant water content specimens (Figures 4.11 and 4.12). The axial strains at failure fell within 40 to 50 %, however the stresses in the specimen under a net confinement of 27 kPa were still changing significantly in this strain range.

The specimen under zero confining stress failed almost immediately and with little volume change (Figure 4.22). The trend in volume change showed that specimens under higher confining stresses had lower volumetric strains than those under lower net confinement. A possible explanation is that the pore space was reduced more during the isotropic compression phase for these specimens.

The Mohr-Coulomb failure envelope for the constant suction tests are shown in Figure 4.23. The envelope defines a soil angle of internal friction ϕ' of 34° and total cohesion of 18.8 kPa.

Figure 4.20 Deviator stress-strain relationships for specimens compacted wet of optimum in drained constant suction tests (net confinement of 0 to 70 kPa).

Figure 4.21 Mean net stress-strain relationships for specimens compacted wet of optimum in drained constant suction tests (net confinement of 0 to 70 kPa).

Figure 4.22 Volumetric axial-strain relationships for specimens compacted wet of optimum in drained constant suction tests (net confinement of 0 to 70 kPa).

Figure 4.23 Mohr-Coulomb failure envelope for specimens compacted wet of optimum in drained constant suction tests.

4.2.1.4 Effects of matric suction on specimens compacted wet of optimum

Figures 4.24 through 4.26. show the stress-strain relationships for three specimens that were isotropically compressed at the same net confining pressure (around 50 kPa) but different matric suctions. The specimens approached failure at axial strains of about 40 to 45%. There was about 21% increase in the deviator stress for the specimen at an initial matric suction of 150.1 kPa over the specimen initially at matric suction of 51 kPa. The increase in the mean net stress (Figure 4.25) was about 12% for these two specimens.

Figure 4.26 shows the changes in matric suction from the initial values during the constant water content tests. The matric suction reduced by about 15% from the initial values in the three tests and was still changing at failure. The changes in volumetric strain with axial strain were identical for the specimens (Figure 4.27). The Mohr circles and assumed failure envelopes are shown in Figure 4.28.

Figure 4.24 Effect of matric suction on deviator stress for three specimens compacted wet of optimum in constant water content tests at different initial matric suction (net confinement around 50 kPa).

Figure 4.25 Effect of matric suction on mean net stress for three specimens compacted wet of optimum in constant water content tests at different initial matric suction (net confinement around 50 kPa).

Figure 4.26 Variation of matric suction with axial strain for three specimens compacted wet of optimum in constant water content tests at different initial matric suction (net confinement around 50 kPa).

Figure 4.27 Volumetric-axial strain relationships for three specimens compacted wet of optimum in constant water content tests at different initial matric suction (net confinement around 50 kPa).

Figure 4.28 Mohr's circles and assumed failure envelopes for three specimens compacted wet of optimum in constant water content tests at different initial matric suction (net confinement around 50 kPa).

4.2.2 Triaxial test results for specimens compacted dry of optimum

4.2.2.1 Isotropic compression

Table 4.4 shows the states of five specimens compacted dry of optimum, after isotropic compression. Under net confining pressures ranging between 29.2 to 68.2 kPa and matric suction around 300 kPa, little quantity of water left the soil pores in comparison to the amount of air expelled. The degree of saturation was still low despite the expulsion of air from these specimens. This is not surprising because the “as compacted” degree of saturation was initially also low (around 27%) (Table 3.3).

4.2.2.2 Constant water content and fully drained constant suction tests

For the shearing phase, two of the specimens were failed in constant water content tests at net confinements of 38.5 and 60.4 kPa while three were failed in fully drained constant suction tests at confinements of 29.2, 51.4 and 68.2 kPa. Results from the series of constant water content and drained constant suction tests are presented together.

Figures 4.29 and 4.30 show the variation of deviator and mean net stresses with axial strain. All the specimens, including those under lower confining stresses approached steady values around axial strain of 41%. A slight decrease in the stresses was observed in the two specimens under net confinements of 60.4 and 68.2 kPa.

Figure 4.31 shows the variation in matric suction with axial strain during the shearing phase. At failure, the matric suctions had only changed slightly (0.53 to 1.5%) from the initial values for the constant water content specimens.

Table 4.4 States of triaxial specimens compacted dry of optimum after isotropic compression (classical analysis).

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	ΔV_a (ml)	ΔV_w (ml)	w (%)	e	S_r (%)	ρ (Mg/m ³)
Initial	0	300	0	0	16.1	1.56	27.3	1.20
M0811	29.2	300.8	64.2	2.2	15.7	1.25	33.3	1.36
M0819	38.5	301.3	76.1	1.0	15.8	1.19	35.2	1.40
M0815	51.4	299.6	92.8	0.2	16.2	1.12	38.2	1.45
M0801	60.4	299.9	113.9	0.2	16.0	1.02	41.6	1.52
M0805	68.2	299.1	106.3	1.1	16.1	1.05	40.5	1.50

Figure 4.29 Deviator stress-strain relationships for specimens compacted dry of optimum in constant water content and constant suction tests (net confinement of 25 to 70 kPa).

Figure 4.30 Mean net stress-strain relationships for specimens compacted dry of optimum in constant water content and constant suction tests (net confinement of 25 to 70 kPa).

Figure 4.31 Variation of matric suction with axial strain for specimens compacted dry of optimum in constant water content and constant suction tests (net confinement of 25 to 70 kPa).

Figure 4.32 shows large volumetric strains between 18 to 27% for all the specimens. Again the specimens at higher net confinement had lower volumetric strains in comparison to the ones at lower confinement.

Table 4.5 gives the states of the specimens compacted dry of optimum at failure. The Mohr-Coulomb failure envelope is presented in Figure 4.33. The internal friction angle (ϕ) from these series of tests is 39.9° while soil total cohesion is 10.2 kPa.

4.3 Results from the main experiments presented under the critical state soil mechanics framework

4.3.1 *Results from specimens compacted wet of optimum*

4.3.1.1 Isotropic compression and unloading in the $v - p'$ plane

After compression the specimens reached the states shown in Table 4.6. This table is similar to Table 4.2, the main difference being that two critical state variables (i.e., p' and v) are included in Table 4.6. The slight differences in the second and fourth columns is because the second column represents the state after compression and the fourth column the state at the commencement of the shearing stage.

Figure 4.34 shows the normal compression line of the specimens compacted wet of optimum on a semi-log plot in the $v - p'$ plane. From the data points, the pre-compression pressure is estimated at 12 kPa. Beyond this stress, the relationship between net confining stress and specific volume was observed to be linear as indicative of normal

Figure 4.32 Volumetric-axial strain relationships for specimens compacted dry of optimum in constant water content and constant suction tests (net confinement of 25 to 70 kPa).

Table 4.5 States of triaxial specimens compacted dry of optimum at failure.

Test #	$\sigma_3 - u_a$ (kPa)	$\sigma_1 - u_a$ (kPa)	$u_a - u_w$ (kPa)	ΔV_a (ml)	ΔV_w (ml)	e	S_r (%)
M0811	30.3	178.4	298.6	-91.4	-1.7	0.81	50.8
M0819	38.4	229.3	298.4	-87.6	0.0	0.78	54.7
M0815	49.6	292.8	299.7	-83.5	-1.8	0.72	57.2
M0801	59.0	329.8	295.4	-63.2	0.0	0.72	57.8
M0805	70.1	393.3	298.8	-75.5	-1.1	0.69	59.5

Figure 4.33 Mohr-Coulomb failure envelope for specimens compacted dry in constant water content and drained constant suction tests.

Table 4.6 Conditions of specimens compacted wet of optimum after isotropic
compression (critical soil state mechanics analysis).

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	p' (kPa)	v	S_r (%)
F0104-CW	1.5	50.8	1.2	2.65	32.5
F0828-CW	8.7	49.9	9.3	2.57	34.2
F0718-CW	18.8	49.9	22.5	2.50	35.6
F0721-CW	29.5	49.1	31.3	2.37	38.9
F0725-CW	37.6	49.4	41.3	2.25	42.4
F0920-CW	48.0	51.0	50.5	2.21	43.9
F0727-CW	60.4	50.4	61.8	2.15	46.3
F1102-CW	150	50.8	151.5	1.98	53.9
F1110-CW	250	50.6	254.4	1.83	60.7
J0405-CD	0	49.6	3.6	2.65	32.0
J0821-CD	25.3	49.1	27.0	2.41	37.8
J0430-CD	46.5	50.4	50.4	2.22	43.9
J0704-CD	67.6	49.6	71.8	2.10	47.4

CW Constant water content tests

CD Drained constant suction tests

Figure 4.34 Isotropic compression and unloading lines for specimens compacted wet of optimum on the $v - p'$ plane.

compression on a semi-log plot. Data for the two specimens at net confinements of 1.5 kPa and 8.9 kPa fell outside the range of the normal compression line. From regression of this linear portion, the slope of the line (λ) is 0.25. The specimen under a net confinement of 250 kPa was unloaded in three stages to about 20 kPa. There was little recovery of deformation as shown by a rebound line with a slope (κ) of 0.012.

4.3.1.2 Isotropic compression and critical state line in the $v - p'$ plane

The conditions of the specimens at critical state are shown in Table 4.7. The states shown for specimens F1102-CW and F1110-CW are those at the end of test rather than at critical state (specimens F1102-CW and F1110-CW did not reach the critical state, a reason will be given later).

In Figure 4.35, five of the constant water content and three of four constant suction tests reached the critical state line shown. This appears to be further evidence that attainment of critical state is independent of stress path. A regression through the critical state points gives the slope of the critical state line on the $v - p'$ space (λ_{cs}) as 0.26. The two specimens marked (*) at net confinements of 150 kPa and 250 kPa did not reach the critical state line defined by the other specimens.

4.3.1.3 Stress paths on the $q - p'$ plane

Constant water content stress paths for specimens compacted wet of optimum are shown in Figure 4.36. The specimen under net confinement of 1.5 kPa did not reach the critical state line. A regression analysis was carried out on the deviator stress values of the specimens that approached critical state at a matric suction about 50 kPa.

Table 4.7 Conditions at critical state for specimens compacted wet of optimum.

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	p' (kPa)	q (kPa)	v	S_r (%)
F0104-CW	2.3	48.6	9.3	20.9	2.66	32.4
F0828-CW	8.7	45.6	41.0	96.9	1.97	57.1
F0718-CW	18.9	48.1	59.2	121.2	1.93	56.4
F0721-CW	29.2	52.6	83.9	163.2	1.81	64.4
F0725-CW	37.8	44.7	94.4	170.4	1.72	72.7
F0920-CW	48.0	42.7	111.7	191.0	1.71	76.5
F0727-CW	60.4	43.3	138.8	235.2	1.69	76.3
F1102-CW	149.1	+3.3*	239.2	270.3	1.87	60.3
F1110-CW	249.1	+32.9*	368.3	357.5	1.80	62.9
J0405-CD	0.9	50.3	5.5	13.8	2.65	31.8
J0821-CD	25.1	50.0	72.0	140.6	1.88	61.2
J0430-CD	45.9	51.4	112.6	200.1	1.78	67.1
J0704-CD	67.9	48.9	151.2	250.0	1.70	75.0

CW Constant water content tests

CD Drained constant suction tests

* Positive pore-water pressures

Figure 4.35 Isotropic compression and critical state lines for specimens compacted wet of optimum on the $v - p'$ plane.

(*) Specimens developed positive pore-water pressures during shearing.

Figure 4.36 Constant water content stress paths for specimens compacted wet of optimum on the $q - p'$ plane (net confinement of 1.5 to 60 kPa).

The critical state line from these tests was superimposed on the stress paths. The slope of the critical state line on this plane (M) is 1.39.

The specimens confined at 150 and 250 kPa also did not reach critical state. It appears that these specimens failed before reaching critical state. The relative positions of these tests at failure in comparison to the critical state line from other constant water content tests (at net confinement of 1.5 to 60 kPa) are shown in Figure 4.37.

Figure 4.38 shows the stress paths from four constant suction tests at net confinements from 25 to 70 kPa. The end points of these tests at critical state lie on the critical state line defined by the constant water content tests. The stress path for the specimen at about zero net confinement however, did not reach this line.

Figure 4.37 Constant water content stress paths for specimens compacted wet of optimum on the $q - p'$ plane (net confinement of 150 to 250 kPa).

Figure 4.38 Drained constant suction stress paths for specimens compacted wet of optimum on the $q - p'$ plane (net confinement of 0 to 70 kPa).

4.3.2 Results from specimens compacted dry of optimum

4.3.2.1 Isotropic compression and critical state line on the $v - p'$ plane

The states of the specimens compacted dry of optimum after isotropic compression and at critical state are presented in Tables 4.8 and 4.9, respectively. Between these two conditions, the changes in specific volume ranged between 15 to 20 %, while the changes in degree of saturation ranged between 40 to 52%.

The isotropic compression line with a slope, λ , of 0.28 for five specimens compacted dry of optimum is shown in Figure 4.39. The slope is slightly different from the value obtained for the specimens compacted wet compacted of optimum (i.e., λ equal to 0.25). A critical state line was also fitted by regression of the critical state data from three constant suction and two constant water content tests. These tests had all ended at matric suctions around 300 kPa. The slope of this critical state line (λ_{cs}) is 0.16. It appears again that this line is independent of stress paths. The slope here is not as close though to the slope of the critical state line for the specimens compacted wet of optimum (where λ_{cs} equal to 0.26).

4.3.2.2 Stress paths on the $q - p'$ plane

All the specimens compacted dry of optimum and under net confinement between 29.2 to 68.2 kPa reached a critical state line. This line is superimposed on the stress paths (Figure 4.40) of the three constant suction and two constant water content tests. The slope of the critical state line on this plane (M) is 1.76.

Table 4.8 Conditions of specimen compacted dry of optimum after isotropic
compression (critical state soil mechanics analysis)

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	p' (kPa)	ν	S_r (%)
M0811	29.2	300.8	32.0	2.25	33.3
M0819	38.5	301.3	40.6	2.19	35.2
M0815	51.4	299.6	53.4	2.12	38.2
M0801	60.4	299.9	61.7	2.02	41.6
M0805	68.2	299.1	69.8	2.05	40.5

Table 4.9 Conditions at critical state for specimens compacted dry of optimum.

Test #	$\sigma_3 - u_a$ (kPa)	$u_a - u_w$ (kPa)	p' (kPa)	q (kPa)	ν	S_r (%)
M0811	30.4	298.2	79.9	148.8	1.81	50.7
M0819	38.6	297.7	101.8	189.7	1.77	54.9
M0815	50.8	299.1	132.5	245.2	1.72	57.3
M0801	57.5	295.1	147.0	268.4	1.72	57.8
M0805	70.6	298.0	175.2	313.9	1.70	58.6

Figure 4.39 Normal compression and critical state lines for specimens compacted dry of optimum on the $v - p'$ plane.

Figure 4.40 Stress paths for specimens compacted dry of optimum on the $q - p'$ plane.

4.4 Results from saturated specimens

4.4.1 *Classical soil mechanics analysis*

Figure 4.41 shows the variation in deviatoric stress with axial strain for three saturated specimens in CU tests. The axial strain for the specimens under confinements of 50 and 75 kPa at failure is about 5%. The specimen under confinement of 150 kPa failed at an axial strain of about 25%.

The development of effective mean stress with axial strain is shown in Figure 4.42. In the three specimens, effective mean stresses initially increased over a small range of axial deformation (about 1.5% axial strain) before it declined to values lower than the initial mean net stress. The first 1.5% of axial strain is the deformation range when the pore-water pressure build up had not yet been initiated. As soon as the pore-water pressure increased, the mean stresses decreased by an amount equal to the pore-water pressure change. The observed pore-water pressure changes are shown in Figure 4.43.

Figure 4.44 shows the Mohr circles and failure envelope for the three tests on the saturated specimens. The cohesion c' is 0 kPa while the soil effective angle of internal friction ϕ' is 33.1°

Figure 4.41 Deviator stress-strain relationships for saturated specimens.

Figure 4.42 Effective mean stress-strain relationships for saturated specimens.

Figure 4.43 Pore-water pressure versus axial strain for saturated specimens.

Figure 4.44 Mohr-Coulomb failure envelope for saturated specimens.

4.4.2 Critical state analysis

The isotropic compression and unloading lines for four saturated specimens are shown in Figure 4.45. The specimen under a net confining pressure of 250 kPa was unloaded in three stages to about 60 kPa. The slope of the normal compression line on the $v - p'$ plane (λ) was found to be 0.092 while that of the rebound line (κ) is 0.001.

Figure 4.46 shows the isotropic normal compression and critical state lines for the saturated specimens on the $v - p'$ plane. The test on the specimen under confinement of 250 kPa had to be discontinued during the shearing stage due to a technical problem. The slopes of the compression and critical state lines (λ and λ_{cs}) were identical.

The stress paths and their relative positions to the critical state line on the $q - p'$ plane are shown in Figure 4.47. The slope of the critical state line on this plane (M) is 1.40.

4.5 Summary of soil properties and critical state parameters

The properties and parameters obtained over the series of triaxial tests for saturated and unsaturated conditions are summarized in Table 4.10. For the saturated tests, the parameters obtained experimentally are also compared with those estimated using the following empirical relationships.

$$\sin\phi' = 0.82 - 0.24\log(I_p) \quad (\text{Kenny, 1959})$$

$$\lambda = \frac{I_p G_s}{\ln 100} \quad (\text{Wood, 1990})$$

$$\Gamma = 1 + G_s(w_L + 0.31I_p) \quad (\text{Wood, 1990})$$

where,

I_p = plasticity index

G_s = specific gravity of solid

w_L = liquid limit

The parameters obtained from the unsaturated series compare favorably to the ranges obtained by other researchers for loam or clay loam soil (Table 2.1). A summary of the models determined from the tests series is given in Table 4.11.

Figure 4.45 Isotropic compression and unloading lines for saturated specimens on the $v - p'$ plane.

Figure 4.46 Isotropic compression and critical state lines for saturated specimens on the $v - p'$ plane.

Figure 4.47 Undrained stress paths and critical state line for saturated specimens on the $q - p'$ plane.

Table 4.10 Summary of soil strength properties and critical state parameters

Parameter	Unsaturated		Saturated	
	Wet of optimum	Dry of optimum	Experimental	Empirical*
ϕ (°)	35.0	39.9	-	-
c (kPa)	20.0	10.2	-	-
ϕ' (°)	-	-	33.1	33.0
c' (kPa)	-	-	0.0	-
λ	0.25	0.28	0.092	0.082
λ_{cs}	0.26	0.16	0.090	-
κ	0.012	-	0.001	-
M	1.39	1.76	1.40	-
ϕ_{cs}	34.4	42.9	34.6	-
N	2.62	2.61	1.85	-
Γ	2.38	2.15	1.83	1.98
ν_{κ}	1.85	-	1.56	-

* Derived by using empirical relations and Atterberg's limits

Table 4.11 Summary of determined critical state relationships.

Test Series	Boundary line	Model	R^2
Saturated	ncl	$v = 1.85 - 0.092 \ln p'$	0.99
	csl ($v - p'$)	$v = 1.83 - 0.090 \ln p'$	0.99
	csl ($q - p'$)	$q = 1.40p'$	-
Unsaturated (wet of optimum)	ncl	$v = 2.62 - 0.25 \ln p'$	0.98
	csl ($v - p'$)	$v = 2.38 - 0.26 \ln p'$	0.99
	csl ($q - p'$)	$q = 1.39p' + 40.3$	0.99
Unsaturated (dry of optimum)	ncl	$v = 2.61 - 0.28 \ln p'$	0.93
	csl ($v - p'$)	$v = 2.15 - 0.16 \ln p'$	0.96
	csl ($q - p'$)	$q = 1.76p' + 10.2$	0.99

CHAPTER 5

DISCUSSION

Overview

The results from the previous chapter are discussed more extensively in this chapter. Relevant experimental results from other works are presented and are used either to support or to explain the observations in this study.

First, the supporting experiments are discussed with emphasis on the role of soil structure and texture. This is followed by a discussion of some experimental observations of the main tests which are of particular interest in the unsaturated and critical state frameworks. The different series of triaxial experiments are then presented for the purpose of comparisons within the study and with earlier studies. The roles of soil structure and matric suction were considered in this discussion. A general discussion including the relevance to agricultural soil problems summarizes this chapter.

5.1 Supporting (or pilot) experiments

Results from these experiments are used either to explain agricultural soil behavior or to complement results from the main triaxial tests.

5.1.1 *Soil-water characteristics and relationship to shear strength*

The soil-water characteristic plays an important role in the compaction of agricultural soil. Many studies have related soil-water characteristics to soil type and initial pore space and distribution. Figures 5.1 and 5.2 show the effect of bulk density and soil texture on the water characteristics of some agricultural soils. Figure 5.1 shows that soil specimens at initially low densities retain more water at low suctions than denser ones, while the converse is true at higher suctions. Reicosky *et al.* (1981) showed this trend in a loam soil (Figure 5.1). Warkentin (1971) noted that this trend was not as prominent however, at high matric suctions for clay soils in comparison with coarse grained soils (Figure 5.2b). Rendig and Taylor (1989) observed that water content at a particular matric suction is greater in clay than in loam or sand because there are more surface area and points of contact per unit volume of soil in fine textured soils (Figure 5.2a).

A close examination of the soil-water characteristic curves indicates that agricultural soils appear to have low air entry values around or less than 1 kPa. This means drying commences at low matric suctions. The clay soils interestingly also had low air entry values.

The sandy clay loam soil used in this study exhibited some of the trends seen in these other agricultural soils as shown in Figure 4.2. Even though all the specimens were

Figure 5.1 Effect of density on soil-water characteristics of some agricultural soils

(a) Reicosky *et al.* (1981) (b) Jamison (1953), redrawn by Warkentin (1971)

(a)

(b)

Figure 5.2 Effect of soil texture on water characteristics of some agricultural soils
(a) Rendig and Taylor (1989) (b) Redrawn by Warkentin (1971).

at an initial bulk density of 1.2 Mg/m^3 , the slight difference observed for the two structures produced could be due to the difference in the initial void ratio and quantity of water present during formation of the specimens by compaction. The water in the wetter soil at 20% combines with the clay particles present to form aggregates and a structure that has more contact points and area in comparison to the drier soil at 16%. The aggregates formed by the wetter soil thus has more confined and less inter-connected pore spaces with more water holding capacity. Hence the gravimetric water content of the wetter specimens was higher over the range of matric suction.

The importance of the soil-water characteristic curve is not only relevant to soil structure, compaction and agronomic purposes but also to shear strength, as has been shown in earlier studies. In recent studies, the soil-water characteristic curve has been used to predict the variation of shear strength with matric suction (Fredlund *et al.* 1995).

Figure 5.3 shows how the shear strength versus matric suction relationship may be approximated from the soil-water characteristic curves of a typical sandy soil of low air entry value and a clay soil of higher air entry value. Fredlund and Rahardjo (1993b) showed that the increase in shear strength with suction is more rapid before the air entry value and that this increase is governed by ϕ' . After this, the strength reduces and remains steady or changes at a constant rate as it is controlled by ϕ^b , or could even decline as matric suction further increases. However, some soils at high water contents become sticky and may actually increase in strength (Mullins *et al.* 1990)

Figure 5.3 Relationship between the soil water characteristic curve and shear strength for a sand and clayey silt (Fredlund and Rahardjo, 1993b)

A computer program being developed at Civil Engineering Department, University of Saskatchewan was used to predict the relationship between shear strength and matric suction using the soil-water characteristic data and the saturated strength properties ϕ' and c' . The plot of predicted and experimental data showed good agreement (Figure 5.4). Between matric suctions of 0 and about 43 kPa, the shear strength had increased about 63% as compared to a 17% increase for matric suction changes from about 43 kPa to about 128 kPa. This implies a reduction in the increase in shear strength with increase in matric suction, an observation which has been verified in many experimental studies. However, Figure 5.4 does not show a distinct location of the point of transition between the control of shear strength variation by ϕ' and ϕ^b .

In view of the above discussions, the application of the soil-water characteristic to predict strength of agricultural soil will be beneficial in tillage and compaction studies. This approach will greatly reduce the amount of laboratory tests and produce reasonable estimates of soil strength properties at less cost.

5.1.2 Direct shear tests

The departure from linearity for the higher water content at high normal stresses seen in Figure 4.6 was also observed by Graecen (1960). Non-linearity was observed in two loam soils (stable and plastic clays) at water content of about 32.5% (8.4% above the plastic limit) after the normal load was increased beyond 44.1 kPa. An explanation that has been given for this behaviour is that the applied normal load was being carried partly by the soil water. Under this condition, the pore-water does not escape fast enough as the soil is loaded in shear leading to the development of positive pore-water pressure.

Figure 5.4 Experimental and predicted shear strength versus matric suction for specimens compacted wet of optimum.

The above explanation would appear adequate for wet clay as was the case in Graecen's experiments.

From the data in Figure 4.2, non-linearity was observed in specimens at a water content of 21.1% (2.3% over the plastic limit) after a normal stress of about 100 kPa was exceeded. In view of the texture of the soil here (sandy clay loam), reduced permeability and development of positive pore-water pressure may not adequately explain the non-linearity. Thus an alternative explanation is suggested as follows.

Tension in the water, held over the surface of the particles may be relaxed (i.e., the negative pore-water pressure may be neutralized and surface tension reduced) by the increased normal load thus giving the water more freedom to move on the particle surface and into interconnected pores. The relaxation of the water first reduces or neutralizes the contribution of suction to the soil strength. Strength is further reduced as the water begins to act like a "lubricant". This makes the particles "roll over" each other instead of rubbing against adjacent ones, which causes reduced interparticle friction, and to a loss of apparent strength.

The nature of the relationship between water content and direct shear strength in Figure 4.5 can be seen and explained in a similar manner to the matric suction and shear strength relationships discussed above. Decreasing water content corresponds to increasing matric suction, and as shown in the Figure 4.5, shear strength increased initially from the wet state as the water content was reduced from 21.1% to about 15%. It appears however, to stabilize over the drier range. Though the role of water content in agricultural

soil strength has previously been established, the trend produced in Figure 4.5 also confirms that matric suction plays an important role in the strength. The difference is that water content is a *deformation* state variable, whereas matric suction is a *stress* state variable.

Shear strength depends largely on the normal stresses on the soil and the structure (as determined by its density and water content). Shear strength can also be expressed as a function of water content or matric suction. Soil water content can be varied by wetting or by drying. If soil is compacted at different water contents, different soil microstructures are produced. However if a compacted soil is wetted or dried, water moves into or out of the pores, hence matric suction reduces or increases but the structure is essentially retained.

By expressing agricultural soil strength as a function of matric suction, the effect due to structural difference is minimized. Variation of strength due to environmental cycles like drying and wetting could be estimated. Also the contribution of normal stresses and matric suction can be distinctly separated. This is necessary in the formulation of appropriate constitutive relations using the stress state variables described by Fredlund and Morgenstern (1977).

5.1.3 Uniaxial compression

Figure 4.8 shows that the specimens prepared from the soil samples at 16 and 20% and a bulk density of 1.2 Mg/m^3 were on the re-loading lines at the start of the tests. Large deformations characteristic of plastic behaviour commenced after the pre-compression

stresses were exceeded as the specimens entered the normal compression phase. The normal compression lines intersected at a vertical stress of about 40 kPa after which more stress was required to deform the drier specimen to some void ratio in comparison with the wetter one. At this stage, the dry specimen had a higher void ratio at a given stress than the wetter one. Similar observations were reported by O'Sullivan (1992).

In Figure 4.9 the behaviour of the two specimens appears to have completely overlapped above the intersecting axial stress of 40 kPa. The compression indices for the tests are 0.35 and 0.36 for the specimens at 16 and 20% water content, respectively.

Though both specimens were compacted wet and dry of the optimum water content, and had different initial void ratio and pre-compression stresses, the specimens exhibited similar responses to the external vertical stresses at some range of stress starting at the intersecting stress. This is seen in the essentially identical and overlapping curves for axial stresses over 40 kPa. Values of the co-efficient of compression were also similar.

5.2 Experimental observations from main tests

5.2.1 *Deformation, deviator stress and volume change*

A common trend seen in all the triaxial tests was the large deformations that occurred during the shearing phase. This is not unusual with agricultural soils which are characterized by highly compressible air-filled pores because of their unsaturated nature and low bulk densities.

From a series of tests on a sandy loam soil, Dawidoswki *et al.* (1990) reported axial strains up to 40% and 65% in cylindrical specimens loaded in a triaxial and a simple

distortion apparatus, respectively. After inspection of the deformed grid under the piston of the distortion device, it was concluded that the assumption that specimens remain homogeneous at large deformations was justified. Dawidowski and Koolen (1987) also reported axial strains up to about 50% in wet dense unsaturated specimens of a silty clay loam soil. It should be mentioned too, that large deformations are to be expected if the purpose is to bring a soil specimen to critical state.

The trend of the volume changes in both isotropic compression and shearing phases for this soil was different from the trend in conventional triaxial tests on saturated specimens. Figures 4.13, 4.17, and 4.22 show a tendency for lesser volume changes in specimens under higher net confining pressures in comparison with the specimens under lower confining pressures. This trend was observed by Herkal *et al.* (1995) and Adams *et al.* (1996). The latter explained that this was probably because the higher net stresses had reduced the pore spaces significantly during isotropic compression which preceded the shear loading stage. An explanation in terms of the soil structure is that a reduction of pore space and inter-connections will result from more closely packed particles. Close particle packing also gives specimens more resistance to external stress and could prevent further expulsion of the entrapped pore air or pore-water.

This observation can also be used to explain the trend in the mobilization of maximum deviator stress during the shearing phase. In Figures 4.11 and 4.20 the specimens under higher net confining stresses developed higher rates of change of deviator stress with axial strain (as indicated by the gradient). It would appear that isotropic compression with higher net confining stresses brought the soil particles in the specimens

closer, thus, at the commencement of shearing, the proximity of the particles and the frictional contacts produced more resistance to the shearing stress in comparison to a typical specimen under a lower net confining pressure. A combination of high normal and shear stresses in a soil medium with greatly reduced pore space can lead to structural damage and a medium unfavourable to crop emergence and yield.

5.2.2 *Strength of unsaturated soil*

The shear strength of agricultural soils is affected by water content, (or matric suction) and applied normal stresses acting on the shearing plane. A qualitative assessment of the relationship between shear strength and matric suction in Figure 5.3 and 5.4 and several other studies shows a particular trend. The extent of shear strength increase as a result of a matric suction increase or a decrease in water content seems to differ for different soil textures. For example, in a series of direct shear tests on the sandy clay loam soil, reduction of gravimetric water content beyond 15% did not appear to increase strength any further (Figure 4.5). For the triaxial tests in Figure 5.4, increasing matric suction from 0 to 125 kPa increased the strength. Shear strength could thus be increased by either increasing the matric suction or decreasing the water content.

5.2.3 *Matric suction variation*

Matric suction is a stress state variable like mean net stress and deviator stress, and could be affected when the soil is subjected to external pressure like a shearing stress. Matric suction change will be affected by a number of factors including duration of load (e.g., chances of matric suction equilibration are higher for slower testing rates), the water content, degree of saturation, soil pore space distribution and texture.

Deviator and mean net stresses reach a state at which further changes are minimal during shearing (e.g., see Figures 4.11 and 4.12), but due to the time required for pore-water response, matric suction requires a longer period to stabilize. In the series of constant water content tests in section 4.2.1.2 and as presented in Figure 4.14, maximum matric suction change was about 12% from the initial value for the highest net confinement. As the pore-air pressure was maintained constant, this change will be directly related to the pore-water pressure change, measured at the base of the specimen.

Bishop *et al.* (1960) measured pore-water pressure changes at the base and mid-height of cylindrical triaxial specimens (102 x 205 mm) of compacted shale. The reading at the base was 55 kPa higher than the change measured at the mid-height when the specimen was sheared at strain rate of 2.5%/h. The pore-water pressure change at the base was only 17 kPa higher than at mid-height with a strain rate of 0.17%/h. One possible reason for difference in pore-water pressures measured at the base and mid-height is end effects. The portions of the specimen in contact with and near to the ends experience higher increase in pore-water pressure than the mid-height. This will gradually equalize over the specimen if the testing rate is slow enough.

In the series of constant water content tests in this study, for the specimens under lower net confining pressures (1.5 to 60 kPa), pore-water pressure changes measured at the base were about 2 kPa to 8 kPa different from the initial values of 0 kPa. Given the initially low degree of saturation, density and texture of the soil, a microstructure of well connected pore spaces is expected. This microstructure increases the chances of better and faster equalization between the base and other parts of the specimens. This means that

minimal pore-water pressure gradient is anticipated in the specimens during the shearing stage though there is still the possibility of end effects. Figure 4.14 shows matric suction variation at the base of the specimen where higher pore-water pressure (hence lower matric suction) is expected. Though pore-water pressure at the middle of the specimen was not measured, based on the experimental observations during constant water content tests at net confinement of 1.5 to 60 kPa, a qualitative prediction of the matric suction at the middle portion of the specimens could be made. First, the pore spaces (in a loose soil and with a low degree of saturation) is not expected to develop high positive pore-water pressures, provided the pore spaces remain loose under the applied stress. Secondly, pore-water pressure is usually lower at mid-height of a triaxial specimen. Thirdly, the specimens failed in compression (without any dilation). As a maximum pore-water pressure of 8 kPa was measured at the base for the series (shown in Figure 4.14), a small (if any) increase of pore-water pressure is anticipated at the mid-height. This means the matric suction could be assumed constant under these conditions.

The discussion above provides conceptual and experimental evidence as to the variation of matric suction in agricultural soil where low stresses are involved, and provides a basis for assuming constant suction even in constant water content drainage situation for some practical agricultural problems. This will simplify the constitutive relations for agricultural soils within an unsaturated soil mechanics framework. It should be recognized though that this assumption may apply to low net confinement and stress situations only. A different trend was observed in pore-water pressure change and thus matric suction change, for the specimens under higher net confinement. Figure 4.18 shows

that the pore-water pressure had increased from the initial values of 0 kPa to 50 kPa and beyond in two specimens under net confinements of 150 and 250 kPa. This is a marked difference from the observation in the specimens under lower confinements. The specimens under higher net confinement were denser and “wetter” (higher degree of saturation as shown in Table 4.2) after isotropic compression prior to shearing. There is a strong likelihood of pore-water pressure gradient or change in matric suction under these conditions (i.e., denser microstructure) as the specimens are sheared, even though the water content remains unchanged. It thus appears that there is a stress range and transition point beyond which constant suction assumptions will not hold. Thus matric suction could be expected to change significantly in some agricultural operations even under constant water content conditions. For example, it could increase substantially during tillage operations (loosening) or decrease under severe or excessive compaction.

Dawidowski and Koolen (1987) observed soil suction increases in a series of triaxial experiments at nearly constant degree of saturation. The authors also compared the shearing and deformation experienced in a triaxial specimen to that of a soil in the center of a rut during passage of a vehicle. Within this analogy, the hard pan will correspond to the bottom of the specimen in a triaxial set up. For one or several passes of traction members, pore-water pressure in the soil close to the hard pan may increase as observed at the base of a triaxial specimen for high stresses. Under this condition, the matric suction at the hard pan may be reduced significantly. A “wet” layer (with high degree of saturation and a denser structure) may thus emerge close to the hard pan. This could improve water retention capacity of this layer, but seriously impede aeration and infiltration.

5.2.4 Type of failure

The manner in which failure is reached in triaxial specimens will affect the validity of the stresses and strains (defined in critical state terms) measured near failure. Development of a premature failure plane makes quantification of stress more difficult, and could hinder the attainment of critical state. All the triaxial specimens in this work failed by bulging symmetrically under large deformations. No critical failure planes developed and there was no peak strength. The specimens failed and approached critical state in a plastic and ductile manner characteristic of loose homogenous specimens compacted wet of optimum (Taylor 1948, Whitman 1960). Figure 5.5 shows the shape of the triaxial specimens at failure. Multiple failure cracks can be seen around the specimens. Although end restraints may have caused some axial non-uniformity, the symmetrical deformation means that variables based on principal stresses and strains such as mean stress (p'), deviator stress (q) and volumetric strain (ϵ_v) will be appropriate in describing the behaviour to large deformations (Wood 1990).

5.3 Comparison of saturated and unsaturated test series

The saturated and unsaturated series compared in this section represent specimens that were initially compacted to the same structure (i.e., identical water content, bulk density and compaction effort). After saturation and consolidation, each of the saturated specimens was at a matric suction of 0 kPa and at some desired confining pressure. The unsaturated specimens were compressed under a matric suction of about 50 kPa and the desired net confining stress before shearing.

Figure 5.5 Typical shape of triaxial specimens at failure.

Figure 5.6 shows the Mohr-Coulomb failure envelopes obtained for the two series of tests. The effective cohesion, c' , is zero for the saturated series while the total cohesion (c) is about 19 kPa for the unsaturated series. The increase in cohesion from 0 kPa to 19 kPa could be attributed to the increase in matric suction from 0 kPa to 50 kPa according to equation (2.8) i.e., $c = c' + (u_a - u_w)_f \tan \phi^b$. Over this range of matric suction, ϕ^b is estimated using equation (2.8) as 21° . This is less than ϕ' obtained as 33.1° from the saturated tests, which means ϕ^b controlled the increase in shear strength with matric suction, right from saturation. This verifies the trend observed in Figure 5.4.

There was not much difference between the angle of friction measured in the saturated and unsaturated series which was found to be 33.1° and 35° for the saturated and unsaturated series, respectively (Table 4.10).

Figure 5.7 shows the normal compression and unloading lines for the saturated and unsaturated series on the $v - p'$ plane. The normal compression line for the saturated specimens fell well below that of the unsaturated series and had a smaller compression index. This shows that the saturated specimens were highly deformable but less compressible. The slope of the normal compression line for the unsaturated series (λ equal to 0.25) was about 2.7 times higher than that of the saturated series (λ equal to 0.092). This implies that along their normal compression lines, the unsaturated series were more compressible than the saturated series, probably due to the large amount of air in the pore spaces of the unsaturated specimens. Wheeler and Sivakumar (1992) reported an increase

Figure 5.6 Shear failure envelopes for the saturated and unsaturated test series.

Figure 5.7 Normal compression (ncl) and unloading rebound lines (url) for the saturated and unsaturated test series.

in λ with an increase matric suction up to some value of matric suction, after which λ decreased.

Both series however, showed little recovery of deformation (i.e., low κ values) upon unloading. This was not surprising as the specimens had undergone such large plastic deformations that elastic recoveries became minimal. Low κ values were similarly observed for agricultural soils by Kirby (1991).

Figure 5.8 shows that the slopes of the normal compression and critical state lines were essentially identical within each series of tests, but the values for the saturated case fell below that of the unsaturated case. The percentage difference in λ_{cs} (slope of critical state line) between the saturated and unsaturated series is similar to that observed for λ (slope of the normal compression line). The relative positions of the critical state lines for the saturated and unsaturated series on the $q - p'$ plane are shown in Figure 5.9. The trend is similar to that of the plot of the failure envelopes. The dryness or presence of matric suction had increased the failure and critical state strength of the soil.

5.4 Comparison of two different structures for unsaturated case

Two cases of the unsaturated test series were compared here in an attempt to investigate the effect of different structures on behaviour and on soil properties. The two structures were produced by compacting the specimens at the same bulk density of 1.2 Mg/m^3 , but at different water contents dry and wet of optimum, at 16% and 20% respectively.

Figure 5.8 Normal compression (ncl) and projection of critical state lines (csl) for the saturated and unsaturated series on the $v - p'$ plane.

Figure 5.9 Critical state lines for the saturated and unsaturated series on the $q - p'$ plane.

Figure 5.10 shows the Mohr-Coulomb failure envelopes of both series while Figure 5.11 shows similar representation on the $q - p'$ plane of the critical state space. The angle of friction in the specimens dry of optimum was about 5° higher than those wet of optimum. Increase in the angle of friction between 3 to 5° has been reported by Escario (1980), Escario and Saez (1986) and Drumright (1989) with increases in matric suction or due to changes in void ratio. The frictional parameter M was also higher on the $q - p'$ plane.

These observations could be attributed to the different states of the specimens at the end of isotropic compression prior to shearing. For example, a test specimen wet of optimum attained a bulk density and degree of saturation of 1.48 Mg/m^3 and 46.3 %, respectively after isotropic compression under a net confining stress of 60.4 kPa. A similar specimen dry of optimum had attained bulk density of 1.52 Mg/m^3 and degree of saturation of 41.6 % after compression under the same net confinement stress. Hence, the specimens wet of optimum were generally looser and wetter, whereas those dry of optimum were denser and drier. The particles in the specimens compacted dry of optimum were more closely packed and had more frictional contact and less water in the pores. These conditions are favourable for the development of higher strength.

Another reason for these observations could be due to different values of matric suction in the specimens. The matric suction for the specimens compacted dry of optimum was generally about 300 kPa as against 50 kPa for the specimens compacted wet of optimum after isotropic compression. It is not certain if the difference in slope in Figure 5.11 was due to matric suction, structural or void ratio differences.

Figure 5.10 Failure envelopes for two cases (different structures) of unsaturated series.

Figure 5.11 Critical state lines for two cases (different structures) of unsaturated series on the $q - p'$ plane.

Figure 5.12 shows that on the $v - p'$ plane, the normal compression line for the specimens compacted dry of optimum fell below that for the specimens compacted wet of optimum because of an initial denser structure. Both were on the wet side (sub-critical) of the critical state line. But the slopes were similar (see Table 4.10), which means similar compressibility was exhibited during isotropic compression. Similarity in slope was also observed in the uniaxial compression tests on these two soil structures (Figure 4.9).

A general look at the critical state points of all the specimens in both series seems to indicate that the points lie on the same critical state line. Figure 5.12 however shows that not all the specimens (compacted dry of optimum), lie on the same critical state line drawn for the specimens compacted wet of optimum. Only those specimens confined at a net stress between 40 kPa to 50 kPa (reaching mean net stress of 80 kPa to 150 kPa at critical state) appear to lie on the same critical state line drawn for the specimens compacted wet of optimum. This observation suggests that the critical state lines intersected at some point within the range of applied stresses. The orientation of the critical state lines in Figures 5.11 and 5.12 verifies this intersection. It also verifies that the structures produced in the specimens compacted wet and dry of optimum may have similar behaviour only at a limiting range of applied stress, but the behaviour differs outside this range. This similarity was also observed over a range of applied stress in the uniaxial compression tests (Figure 4.8). From Figures 5.10 through 5.12, differences in behaviour appear to outweigh similarities for the specimens wet and dry of optimum.

Figure 5.12 Normal compression and projection of critical state lines for two cases (different structures) of unsaturated series.

Some properties obtained from these series of tests also had different values (Table 4.10). Bearing in mind that the matric suction was different in these series, it however appears that the structure (based on the above observations) rather than the matric suction accounts for the differences in behaviour and soil properties determined from both series of unsaturated tests.

5.5 General

A general discussion of the results presented in chapter 4 and in sections 5.1 to 5.4 in this chapter in terms of implications for agricultural soil problems is presented below. Both macro- and micro-structural nature of this soil are considered in the discussions.

5.5.1 *Structure*

The importance of soil structure for agricultural activities cannot be overemphasized. Soil parameters such as air permeability, strength characteristics, infiltration capacity, pore size distribution and erodibility depend on the structure of the soil (O'Sullivan and Simota 1995).

Compaction is a keenly monitored process in arable and forest lands. Compaction affects soil structure and hence hydraulic properties (Horton *et al.* 1994; Dawidowski and Lerink 1990; Guerif 1990). O'Sullivan (1992) and Fredlund and Rahardjo (1993a) noted

that different structures could be produced by compacting the same soil at the same bulk density but different water contents.

Dawidowski and Lerink (1990) defined “wet” and “dry” compaction based on concepts by Koolen (1987). Dry compaction occurs when volume reduction upon loading is limited by the increase of the soil compressive strength. Under wet compaction, the volume reduction upon loading is limited by entrapped soil water.

Evidence of compaction was observed in various phases of the experimental program in this study. The specimens were prepared mainly by compacting soil in a mold. The uniaxial compression experiments involved vertical loading and compaction of soil specimens within a laterally confined cylinder. Compaction occurred when the normal load was applied in the direct shear tests and during isotropic compression in the triaxial tests. The large deformations experienced during the shearing stage of the triaxial tests also indicate that some compaction occurred in the specimens.

All the triaxial specimens were compacted from soil having the same texture (a sandy clay loam) to give similar structure as in the specimens that were used in all of the supporting experiments as well as in two main test series. This approach was taken so that variations due to structural differences could be eliminated. For example, all the specimens used for the uniaxial compression, shearbox, soil-water characteristic curve determination (20% curve in Figures 4.2 and 4.3) and triaxial tests (saturated and wet of optimum series) were compacted at a bulk density of 1.2 Mg/m^3 and a water content of about 20% to produce identical structures. These tests could thus be to in order to compare or explain the observed behaviour without the uncertainty of structural influence.

The pore size distribution is a soil quality parameter which affects the matric suction (a stress state variable dependent on pore pressures) of the soil. Matric suction can be viewed likewise as a variable which influences soil properties which are dependent on pore size, pore distribution and pore fluids. Matric suction is also known to affect the strength of the soil. The extent of the influence of matric suction on these parameters could be affected by the structure produced after applying external stresses. Matric suction can be considered as a stress state variable which combined the physical quality of a soil (as determined by the microstructure produced) to the soil strength.

Water content and degree of saturation are closely tied to the soil structure. The degree to which water content and degree of saturation affect soil structure is affected by the texture. For the same soil texture, there is evidence to show that the matric suction of soil changed upon loading with no change in degree of saturation (Dawidowski and Koolen 1987). However, in the constant water content tests (at low net confinement) in this study, the matric suction remained almost constant while the degree of saturation increased (Table 4.7). The constant water content test series under high net confinement showed significant reductions in matric suction, the development of positive pore-water pressures and low volumetric strains (Figure 4.17). It appears that at this stage, the structure produced by the greatly reduced pore air space and closely packed particles experienced “wet compaction”.

Larson and Gupta (1980) also observed pore-water pressure increases during compaction in oedometer tests once the degree of saturation was above a threshold value.

The threshold degree of saturation is characteristic of the particular soil and increased with clay content in the range of 30% to 60%.

5.5.2 State boundary planes and the Roscoe surface

The critical state behaviour of the two structures of the soil used in this study with respect to Roscoe boundaries are discussed below.

Series with same structures but different matric suction

For specimens with identical structures, increase in matric suction from saturation resulted in shifts on the $v - p'$ plane such that compressibility of pore space is increased but deformability is decreased (Figure 5.7). On the $q - p'$ plane, an increase in suction translates to an increase in soil strength (Figure 5.9). These shifts indicate the formation of a new state boundary surface as a result of an increase in matric suction (i.e., desaturation).

On the same plane and scale, the unsaturated series has a wider sub-critical domain (Roscoe surface) than the saturated series. Hence, for the same stress path, the saturated soil traverses the Roscoe surface over a shorter path than unsaturated soil for the same initial net stress. This means the saturated tests reached the critical state line under undrained conditions over a shorter range of confining stress. This situation could occur in the field in early spring if traction members or wheels pass over wet topsoil.

Generally, the pore space of unsaturated specimens decreased by about 20% before reaching the critical state line during constant water content test paths. The mean net stress had increased 2 to 2.5 times upon reaching this state. Such a combination of

stress and volume change and the resulting compaction could cause structural damage in terms of the quality and usefulness of the soil to seed germination and crop yield. But the soil at this state could also have a structure which is quite suitable for seed germination (higher degree of saturation) and root establishment. An evaluation of crop performance under the critical state attained could confirm or verify these statements.

An assessment of the positions of the normal compression and critical state lines on the $v - p'$ plane (Figure 4.35) shows that the constant water content stress paths placed the critical state points well below the starting point on the normal compression line because of the large deformations involved. For further experimental study, data points for the upper part of the critical state line could be established for soils that undergo large deformations by considering undrained stress path or by overconsolidating prior to shearing. This will also provide valuable information for tillage studies as critical state will be approached from the super-critical or dry side.

Series with different structures and matric suction

From the $v - p'$ plot in Figure 5.11, it appears that the sub-critical domains for the unsaturated series (i.e., specimens compacted wet and dry of optimum) intersected at some point. The Roscoe surface for the specimens compacted dry of optimum appears to have emerged from below that of the specimens compacted wet of optimum. Though the projections of the critical state lines from both series appear to lie along the same line, the critical state line (limiting the Roscoe surface) for specimens compacted dry of optimum rose above that of the specimens compacted wet of optimum. This rise is seen in the slope of the critical state line in the $q - p'$ plane (Figure 5.10). Hence these two series of tests

gave intersecting state boundaries, with the boundaries for the series compacted dry of optimum rising over the boundaries for the series compacted wet of optimum at the point of intersection.

The observation above suggests that hardening occurred when the soil dried out to a higher matric suction. Even when field activities are carried out at drier water contents, there could still be a tendency to create a hardened soil medium as the soil compacts from the passage of agricultural machinery.

Figure 5.13 shows the boundaries of the Roscoe surface on the $p' - q - v$ axes for the unsaturated series (i.e., compacted specimens wet of optimum). Tests data were within the sub-critical domain, showing that the specimens approached the critical state line on the wet side of the critical state wall. The approach to critical state started out on the normal compression line at the narrow portion of the Roscoe surface (loose state). In traversing the Roscoe surface, a typical specimen moved more lengthwise (in the direction of decreasing specific volume, v and increasing mean net stress p') than widthwise (direction of increasing deviatoric stress q') as it approached the critical state line. This explains the large deformation an agricultural soil experiences even when small shear stresses are involved.

5.5.3 Critical state parameters, limiting stresses and effect of structure

The critical state parameters are influenced by several factors like soil structure and matric suction. Studies have also suggested that there are limiting stress ranges for which the parameters of log-linear critical state models are valid.

Figure 5.13 Boundaries of the Roscoe surface for unsaturated specimens compacted wet of optimum

From the second and fourth columns of Table 4.10, a comparison of the unsaturated and saturated series (similar structure) shows that the parameters determined for the unsaturated series generally had higher values than those of the saturated series. This difference can be attributed to the presence of air in the pore space and the higher matric suction. There was not much difference however in the frictional parameter (M) for both series. The rebound parameter (κ) was also low in both cases.

A comparison of the parameters from two unsaturated series at different structures shows that the specimens dry of optimum had a higher value of the frictional parameter (M). However, the intercepts of the critical state line on the $q - p'$ (Figure 5.10) and $v - p'$ (Figure 5.12) planes were smaller. The slope of the critical state line (λ_{cs}) on the $v - p'$ plane was also smaller even though the normal consolidation lines for both series had identical slopes (λ).

O'Sullivan *et al.* (1994) observed that there were limiting stress ranges over which the linear model $v - p'$ applies. It was suggested that this could be a useful indicator of the commencement of structural damage and that a soil would not compact readily above this limit but would deform easily in response to a small shearing stress.

The results for specimens wet of optimum (initially at a matric suction of about 50 kPa), showed that on the $v - p'$ plane, isotropic compression under net confinements ranging from 20 kPa to 350 kPa brought ten specimens to the linear portion of the normal compression line. Eight of these specimens (under lower net confinement range) reached the critical state line with mean net stress ranging between 60 to 140 kPa. Two of the specimens (under higher net confinement), did not reach the critical state line and showed

markedly different behaviour from the other eight. It thus seems that a transition stress falls somewhere between these two net confinement ranges.

The observations in Figure 4.35 indicate that over the net stress range (20 to 250 kPa), the normal compression points lie within a linear range and λ is valid. The linearity of the critical state points on the other hand is limited to a mean net stress of about 150 kPa in this series of tests. Table 5.1 shows the stress ranges for the application of the linear model ($v - p'$) for three series of tests. The ranges specified are not necessarily the limiting stresses, since the testing program spanned over a conservative range of stress. More tests around the transition stresses will be required if the objective is to determine limiting stresses.

Table 5.2 shows the maximum limiting stress for application of the linear model in the $v - p'$ plane for tests on different soils at varying water contents by O'Sullivan *et al.* (1994). From a macroscopic viewpoint, these limits were used to estimate minimum water content at which external stress will likely cause structural damage.

The experimental data in this study suggest limited linear ranges such as observed by O'Sullivan *et al.* (1994) (Figure 5.14). The experimental observations suggest that if attainment of critical state is the objective, then tests should span over a fairly large range of net confinement stresses. Major limitations to this testing range (apart from equipment capabilities) are either the development of positive pore pressures, or the specimens becoming so densely packed that little or no volume change can occur on application of shear stress.

Table 5.1 Linear ranges on the $v - p'$ plane over which the critical state parameters were determined in three series of tests.

Test series	Initial matric suction	NCL		CSL	
		Lower	Higher	Lower	Higher
Saturated	0	50	250	40	130
Unsaturated (w_{opt})	50	15	250	59	150
Unsaturated (d_{opt})	300	30	70	80	180

All stresses in kPa.

(w_{opt}) Specimens compacted wet of optimum water content

(d_{opt}) Specimens compacted dry of optimum water content

Table 5.2 Maximum limiting stress for application of the linear model in the $v - p'$ projection (O' Sullivan *et al.*, 1994)

	Winton			Evesham	
Water content (%)	19.5	23.0	25.3	35.6	39.5
Limit NCL	200	170	70	300	150
Limit CSL	150	100	50	200	70
Specific volume (both lines)	1.60	1.60	1.76	2.08	2.27

All stresses in kPa

Figure 5.14 Location and orientation of the normal compression and critical state lines for three agricultural soils at three water contents (a) Darvel (b) Winton (c) Evesham soil (O'Sullivan *et al.* 1994)

Attainment of critical state also becomes difficult under these conditions because changing pore-water pressures causes the matric suction to be changing at the end of the test. This violates the critical state criterion which requires that little or no change in stress state occur on attainment of critical state. Though the condition of little or no volume change meets the requirement of critical state, a situation could be produced whereby the pore-water behaves in such a manner as to inhibit the mobilization of maximum shear stress.

As explained earlier, if the pore-water starts to act in a “lubricating” manner between the densely packed particles, the strength due to the development of friction is reduced. The concept of “wet compaction” described by Koolen (1987) and Dawidowski and Lerink (1990) could be applied to these observations.

The orientation of the critical state lines and the parameters obtained therein appear to be affected by the structure of the soil. On the $v - p'$ plane, O'Sullivan *et al.* (1994) applied the “pivot point” concept first proposed by Hettiaratchi and O'Callaghan (1985) to their experimental data. Figure 5.14 shows that the slopes of the critical state lines are generally about 12% higher than those of the normal compression lines and that these two lines tended to pivot about a point as water content changed. On the $q - p'$ plane, the critical state line passed through the origin and increased in slope with decreasing water content (Figure 5.15).

The pivot point concept is, however, not applicable to the experimental data of Wheeler and Sivakumar (1992), Petersen (1993) and some of the data in this study.

Figure 5.15 Orientation of the critical state line on the $q - p$ plane from other studies.
 (a) Petersen (1993) (b) Wheeler and Sivakumar (1995) (c) O'Sullivan *et al.* (1994).

Experimental evidence from these studies also showed a difference in the orientation of the critical state line on the $q - p'$ plane.

The experimental data (for two series of tests on specimens with similar structure) in this study agree more with the results of Wheeler and Sivakumar (1992) and Peterson (1993). On the $v - p'$ plane, the normal compression and critical state lines had close values even though differences were noticed at times. The critical state lines also had intercepts on the $q - p'$ plane. A common feature in the experiments in this study and those of Wheeler and Sivakumar (1992) and Peterson (1993) is that soil specimens of similar structures were used for the series of triaxial tests. O'Sullivan *et al.* (1994) on the other hand used remoulded specimens of different structures. Observations from the analysis of the two unsaturated tests series at different structures in this study, however, show qualitative agreement with the observations of O'Sullivan *et al.* (1994).

CHAPTER 6

CONCLUSIONS

6.1 Summary and general conclusions from study

In a pioneering effort, the behaviour of an agricultural soil has been studied within a combined unsaturated and critical state framework. The unsaturated soil mechanics framework recognizes the presence and significance of air in the behaviour of unsaturated soils. This framework also introduces and uses a more comprehensive and appropriate stress state variables of the soil. The critical state framework, on the other hand, allows for the combination of shear strength and volume change behaviour.

To apply these frameworks, a testing program involving a series of tests was designed. Supporting experiments were carried out to complement the main tests which were carried out using a state-of-the-art triaxial testing system. Major modifications were made on the system in order to implement the unsaturated and critical state frameworks. The problem of air volume change measurement was addressed by customizing a digital pressure-volume controller for this purpose. A specimen preparation procedure was strictly adhered to, and a special unit was also fabricated to produce specimens that were as homogeneous as possible.

For this study, the Cam clay model (adapted for unsaturated soil) was investigated experimentally. The effect of matric suction (which is related to water content) and soil

structure on soil behaviour were examined since these two factors play important roles in the physical quality of arable soils.

Sets of critical state parameters were determined from triaxial tests. These parameters are required in numerical analysis to simulate soil behaviour. The study showed that these parameters are influenced by changes in matric suction and the soil structure.

An overall view of the study leads to the following observations and conclusions:

- The unsaturated soil mechanics framework can be applied to agricultural soils. This approach makes it possible to quantify the stresses in the soil by separating stress on the soil structure (i.e., normal stress) from internal stress within the pores (i.e., matric suction). Under this framework, the soil strength is affected linearly by a variation of net normal stress ($\sigma_n - u_a$) and non-linearly by matric suction variation ($u_a - u_w$).
- From experimental results in this study, it is evident that matric suction has a major influence on the soil strength and volume change characteristics. The influence of matric suction should be considered in all agricultural soil mechanics investigations. For agricultural applications, matric suction could be assumed constant under certain loading situations, even under constant water content stress paths. This assumption will be valid so long as the soil is subjected to relatively low stresses compared to its in-situ matric suction. However, this assumption will no longer hold if a transition or threshold stress is exceeded. Depending on the application, it may or may not be desirable to locate this threshold stress.

- Agricultural soil behaviour can be described within the critical state soil mechanics framework. This was achieved in this study while using independent stress state variables to define stresses in the soil. Other researchers used the total stress approach for practical convenience, but this approach does not consider the independent nature of internal stresses (matric or osmotic suction) in an unsaturated soil.
- A testing program based on unsaturated and critical state soil mechanics principles was established to test an agricultural soil. Stress and volume changes were simultaneously observed in a modified triaxial apparatus as the soil responded to external loading.
- Critical state parameters relevant in the formulation of constitutive relations were determined from a series of triaxial experiments involving isotropic compression and shear tests.
- Experimental evidence in this study confirmed the existence of a critical state line and a surface bounded by this line and the normal compression line. The surface, termed the Roscoe surface, is located on the wet side (sub-critical) of the critical state wall. The location of Roscoe surface boundaries provides a basis for the analysis of compaction problems.
- Linear ranges exist on the isotropic normal compression and critical state lines for which critical state parameters are valid. The models were not good when low stresses are involved. A translational shift of the Roscoe boundaries away from the origin occurred as a result of increase in matric suction (Figure 5.8). A rotational shift was however observed with structural changes (Figure 5.12).

- It was experimentally verified that the stress path does not affect the attainment of critical state, model parameters and orientation of the critical state line. The constant water content and drained constant suction tests in each test series reached the same critical state line. However, model parameters and the orientation of the critical state line appear to be influenced by the variation of water content and matric suction. Two observations were made with regard to the influence of water content on critical state parameters. Different effects were observed if water content variation led to different structures in comparison to water content changes that produced similar structures. For example, the frictional parameter M changed significantly in experiments that involved water content change accompanied by structural change. On the other hand, M did not vary much where water content change did not alter the structure of the soil. Also, the orientation of the critical and normal compression lines on both $q - p'$ and $v - p'$ planes are influenced by the consistency of the soil structure in the specimens used.
- The results show that a combination of the critical state and the unsaturated soil mechanics framework shows potential in the development of appropriate models to solve agricultural soil compaction problems.

6.2 Contribution of research

- This study has implemented the combination of unsaturated and critical state frameworks for the first time to the study of an agricultural soil behaviour. Appropriate experimental techniques to achieve this were also designed.
- Critical state parameters based on an unsaturated soil mechanics framework for a sandy clay loam soil were determined in the laboratory for use in the formulation of appropriate constitutive relations for an agricultural soil. The parameters here will be valuable to a collation of critical state parameters from several studies on agricultural soil in the future. Such collation will serve as a strong data base for the formulation of more effective and relevant empirical or constitutive relations for agricultural soils.
- Using matric suction as an independent stress state variable, results from this study were used to resolve or explain some discrepancies reported in the study of the application of the critical state theory to unsaturated agricultural soils.
- The state-of-the-art computer-controlled hydraulic-triaxial testing system which was originally designed for testing saturated soils was modified and re-programmed for the first time to allow testing of both saturated and unsaturated soils.
- Air volume change measurement, which constitutes a major problem in unsaturated soil testing, was effectively addressed in this study, using a combination of established precautions and the customization of a digital pressure-volume-controller.

6.3 Future research opportunities and suggestions

(i) With the establishment of the equipment and testing program, a broader range of matric suction could be examined for soil of same structure but different matric suction. But the problem of the duration of time for equilibration of suction throughout the 70 x 140 mm specimens must be addressed. Smaller specimens could be used.

(ii) Though constant water content and constant suction stress paths are practical for agricultural soils, other paths could be used in order to examine the uniqueness of the critical state line. A stress path that allows the mean net stress (p') to decrease or to remain constant as the specimen is sheared will be particularly useful in defining the yield surface. An additional digital-pressure-volume controller needs to be incorporated to control the cell confining pressure to achieve several other stress paths.

(iii) Establishment of a wider range for the critical and normal compression lines will help in identifying limiting stresses. This could be achieved by investigating several stress paths or changing the stress history by light overconsolidation. Tests on overconsolidated specimens will be useful in establishing a surface on the super-critical side which can be applied to tillage studies. But the development of slip failure planes on overconsolidated specimens might constitute a problem in determining the stresses and in applying the critical state concept.

iv. The framework and models need to be verified for real soil problems.

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APPENDICES

Appendix A: Data for the determination of the soil-water characteristic

Appendix B: Data for uniaxial compression tests

Appendix C: Data for direct shear tests

Appendix D: Identification of triaxial test specimens

Appendix E: Triaxial specimen data

Appendix F: Triaxial shear tests data

Appendix G: Air volume change measurement subroutine

Appendix A: Data from determination of the soil-water characteristic

			EXPERIMENTAL		
Specimens at 20 % water content			Specimens at 16 % water content		
Matric suction (kPa)	Degree of saturation (fraction)	Water content (%)	Matric suction (kPa)	Degree of saturation (fraction)	Water content (%)
0.1	0.92	57.4	0.1	0.99	58.8
0.1	0.97	60.2	0.7	0.92	55.0
0.1	0.98	60.7	3.5	0.68	40.2
0.1	0.98	61.4	15.9	0.44	26.2
0.1	0.99	62.3	32.4	0.36	21.2
0.1	0.98	61.0	300	0.27	15.9
0.1	1.00	60.7	300	0.27	15.9
0.7	0.95	59.1	500	0.25	14.8
2.07	0.75	46.6	1500	0.23	13.4
2.07	0.78	47.3	1500	0.23	13.7
11.72	0.53	32.1			
27.6	0.39	24.3			
29.65	0.39	23.5			
50	0.33	20.4			
50	0.33	20.6			
50	0.33	20.7			
75	0.32	20.1			
75	0.32	20.1			
100	0.31	20.2			
150	0.30	18.9			
200	0.31	19.5			
200	0.30	18.8			
300	0.28	17.7			
300	0.29	17.9			
300	0.30	18.8			
300	0.29	18.0			
300	0.29	17.9			
400	0.28	17.4			
400	0.29	17.9			
400	0.27	16.8			
500	0.27	16.7			
1000	0.27	16.5			
1000	0.27	16.6			
1500	0.26	16.1			
1500	0.26	16.2			

DATA FITTED WITH CFVIEW						
Specimens at 20.4 % water content			Specimens at 16 % water content			
Matric suction (kPa)	Degree of saturation (fraction)	Water content (%)		Matric suction (kPa)	Degree of saturation (fraction)	Water content
0.17	0.99	62.27		0.17	0.99	58.66
0.19	0.99	62.22		0.18	0.99	58.59
0.20	0.99	62.16		0.20	0.99	58.51
0.22	0.99	62.10		0.21	0.99	58.42
0.24	0.99	62.02		0.23	0.98	58.32
0.26	0.99	61.94		0.25	0.98	58.21
0.28	0.99	61.84		0.27	0.98	58.09
0.30	0.99	61.73		0.29	0.98	57.95
0.33	0.98	61.61		0.32	0.98	57.80
0.35	0.98	61.47		0.34	0.97	57.63
0.38	0.98	61.31		0.37	0.97	57.44
0.41	0.98	61.13		0.40	0.97	57.23
0.45	0.97	60.93		0.43	0.96	57.00
0.48	0.97	60.70		0.47	0.96	56.75
0.52	0.97	60.45		0.50	0.95	56.47
0.56	0.96	60.17		0.55	0.95	56.17
0.61	0.96	59.85		0.59	0.94	55.83
0.66	0.95	59.51		0.64	0.94	55.46
0.71	0.94	59.12		0.69	0.93	55.06
0.77	0.94	58.69		0.74	0.92	54.63
0.83	0.93	58.22		0.80	0.91	54.16
0.90	0.92	57.70		0.87	0.91	53.65
0.97	0.91	57.14		0.94	0.90	53.10
1.05	0.90	56.53		1.02	0.89	52.52
1.13	0.89	55.87		1.10	0.88	51.89
1.22	0.88	55.17		1.19	0.86	51.23
1.32	0.87	54.41		1.28	0.85	50.53
1.43	0.86	53.61		1.39	0.84	49.80
1.55	0.84	52.77		1.50	0.83	49.03
1.67	0.83	51.89		1.62	0.81	48.23
1.81	0.81	50.97		1.76	0.80	47.40
1.95	0.80	50.02		1.90	0.79	46.54
2.11	0.78	49.04		2.05	0.77	45.67
2.28	0.77	48.04		2.22	0.76	44.78
2.46	0.75	47.04		2.40	0.74	43.87
2.66	0.73	46.02		2.59	0.73	42.96
2.88	0.72	45.00		2.80	0.71	42.05
3.11	0.70	43.99		3.03	0.69	41.14
3.36	0.69	42.99		3.27	0.68	40.23
3.64	0.67	42.00		3.54	0.66	39.33
3.93	0.65	41.03		3.83	0.65	38.44
4.25	0.64	40.08		4.14	0.63	37.57

Data fitted with CFVIEW continued

4.59	0.63	39.16		4.47	0.62	36.71
4.96	0.61	38.26		4.84	0.61	35.88
5.36	0.60	37.40		5.23	0.59	35.07
5.80	0.58	36.56		5.65	0.58	34.28
6.27	0.57	35.75		6.11	0.57	33.51
6.77	0.56	34.97		6.60	0.55	32.77
7.32	0.55	34.23		7.14	0.54	32.05
7.91	0.53	33.51		7.72	0.53	31.36
8.56	0.52	32.83		8.35	0.52	30.70
9.25	0.51	32.17		9.02	0.51	30.05
10.00	0.50	31.54		9.75	0.50	29.44
10.80	0.49	30.93		10.54	0.49	28.84
11.68	0.48	30.35		11.40	0.48	28.27
12.62	0.48	29.80		12.32	0.47	27.72
13.64	0.47	29.27		13.32	0.46	27.20
14.75	0.46	28.76		14.40	0.45	26.69
15.94	0.45	28.27		15.57	0.44	26.20
17.23	0.44	27.80		16.83	0.43	25.74
18.62	0.44	27.35		18.20	0.43	25.29
20.13	0.43	26.92		19.67	0.42	24.85
21.76	0.42	26.50		21.27	0.41	24.44
23.52	0.42	26.10		22.99	0.41	24.04
25.42	0.41	25.72		24.86	0.40	23.65
27.48	0.40	25.35		26.87	0.39	23.28
29.70	0.40	24.99		29.05	0.39	22.92
32.11	0.39	24.65		31.41	0.38	22.57
34.70	0.39	24.31		33.95	0.38	22.24
37.51	0.38	23.99		36.70	0.37	21.92
40.54	0.38	23.68		39.68	0.36	21.61
43.82	0.37	23.38		42.90	0.36	21.30
47.37	0.37	23.09		46.38	0.35	21.01
51.20	0.36	22.81		50.14	0.35	20.73
55.34	0.36	22.54		54.20	0.35	20.46
59.82	0.36	22.27		58.59	0.34	20.19
64.66	0.35	22.01		63.35	0.34	19.93
69.89	0.35	21.76		68.48	0.33	19.68
75.55	0.34	21.52		74.03	0.33	19.44
81.66	0.34	21.28		80.04	0.32	19.21
88.26	0.34	21.05		86.53	0.32	18.98
95.40	0.33	20.83		93.54	0.32	18.75
103.12	0.33	20.61		101.13	0.31	18.54
111.47	0.33	20.40		109.32	0.31	18.32
120.48	0.32	20.19		118.19	0.31	18.12
130.23	0.32	19.99		127.77	0.30	17.92
140.76	0.32	19.79		138.13	0.30	17.72

Data fitted with CFVIEW continued

152.15	0.31	19.59		149.33	0.30	17.53
164.46	0.31	19.40		161.44	0.29	17.34
177.77	0.31	19.21		174.53	0.29	17.15
192.15	0.30	19.03		188.68	0.29	16.97
207.69	0.30	18.85		203.97	0.28	16.79
224.49	0.30	18.67		220.51	0.28	16.62
242.66	0.30	18.49		238.39	0.28	16.45
262.29	0.29	18.32		257.72	0.27	16.28
283.50	0.29	18.15		278.61	0.27	16.11
306.44	0.29	17.98		301.20	0.27	15.95
331.23	0.28	17.82		325.62	0.27	15.79
358.03	0.28	17.65		352.03	0.26	15.63
386.99	0.28	17.49		380.57	0.26	15.47
418.30	0.28	17.33		411.42	0.26	15.32
452.14	0.27	17.17		444.78	0.26	15.16
488.72	0.27	17.01		480.84	0.25	15.01
528.25	0.27	16.85		519.83	0.25	14.86
570.99	0.27	16.70		561.97	0.25	14.71
617.18	0.26	16.54		607.54	0.25	14.56
667.11	0.26	16.38		656.80	0.24	14.41
721.08	0.26	16.23		710.05	0.24	14.26
779.41	0.26	16.07		767.62	0.24	14.11
842.47	0.25	15.91		829.85	0.24	13.97
910.62	0.25	15.76		897.14	0.23	13.82
984.29	0.25	15.60		969.88	0.23	13.67
1063.92	0.25	15.44		1048.51	0.23	13.52
1149.99	0.24	15.29		1133.52	0.23	13.38
1243.02	0.24	15.13		1225.43	0.22	13.23
1343.58	0.24	14.97		1324.78	0.22	13.08
1452.27	0.24	14.81		1432.19	0.22	12.93
1569.76	0.23	14.64		1548.31	0.22	12.78
1696.75	0.23	14.48		1673.85	0.21	12.63
1834.02	0.23	14.32		1809.56	0.21	12.48
1982.39	0.23	14.15		1956.28	0.21	12.33

Prediction of shear strength versus matric suction from the soil-water characteristic relationship.

EXPERIMENTAL		PREDICTION	
Matric Suction (kPa)	Shear strength (kPa)	Matric Suction (kPa)	Shear strength (kPa)
0	33	0.0	37.0
44	53	0.1	37.1
86	54.5	0.2	37.1
125	62	0.3	37.2
		0.4	37.3
		0.7	37.5
		1.2	37.8
		2.0	38.1
		3.3	38.5
		5.5	39.0
		9.0	39.9
		14.8	41.1
		24.5	43.1
		40.3	46.2
		44.0	46.9
		66.5	51.0
		86.0	54.5
		109.7	58.5
		125.0	61.1
		180.8	70.2

Appendix B: Data for uniaxial compression tests

Specimen at 20% water content		Specimen at 16% water content	
Axial stress (kPa)	Void ratio	Axial stress (kPa)	Void ratio
1.8	1.65	2.1	1.56
3.1	1.65	3.9	1.56
4.9	1.65	5.7	1.56
6.8	1.65	7.3	1.56
8.3	1.65	8.8	1.55
10.4	1.65	10.9	1.55
11.9	1.65	12.7	1.55
13.8	1.65	14.5	1.55
15.6	1.65	16.1	1.55
16.9	1.64	17.9	1.55
18.4	1.64	20.3	1.55
20.3	1.63	22.1	1.55
21.8	1.63	23.4	1.55
22.9	1.62	25.2	1.55
24.9	1.61	27.0	1.55
26.0	1.60	28.6	1.55
27.3	1.59	30.4	1.54
29.4	1.58	31.9	1.54
31.2	1.57	33.8	1.54
32.7	1.56	34.5	1.53
34.3	1.55	36.6	1.53
36.4	1.54	37.9	1.52
37.9	1.53	39.7	1.52
40.0	1.51	42.3	1.51
41.3	1.50	43.6	1.51
43.1	1.49	45.7	1.50
44.9	1.48	47.0	1.50
47.3	1.47	49.1	1.49
48.6	1.46	50.9	1.49
50.4	1.45	61.0	1.45
52.5	1.44	70.1	1.42
54.0	1.43	72.2	1.42
55.8	1.42	80.8	1.39
58.2	1.41	82.1	1.39
59.2	1.40	90.1	1.37

Data for uniaxial compression continued

61.0	1.39		98.4	1.34
62.6	1.38		100.3	1.34
64.4	1.38		119.2	1.29
66.2	1.36		129.9	1.26
68.6	1.35		139.0	1.24
70.9	1.34		149.6	1.22
73.2	1.33		160.3	1.20
76.4	1.32		169.1	1.17
77.9	1.31		176.9	1.15
80.8	1.29		184.9	1.14
83.1	1.28		193.0	1.13
86.0	1.27		200.8	1.12
88.6	1.26		208.8	1.11
91.7	1.25		216.6	1.09
99.0	1.21		224.9	1.08
101.8	1.20		233.2	1.07
105.7	1.19		241.3	1.07
108.6	1.18		249.4	1.06
120.8	1.14		257.4	1.05
123.6	1.13		266.0	1.04
127.3	1.11		274.3	1.03
130.9	1.10		282.9	1.02
134.8	1.09		291.2	1.01
137.9	1.08		299.5	1.00
141.6	1.07		308.1	0.99
145.5	1.06		317.4	0.98
149.4	1.05		326.5	0.98

Appendix C: Direct shear box tests data

Specimens prepared at water content of 3.5%

	Normal stress (kPa)					
	25	50	75	100	125	150
Displacement (mm)	Shear stress (kPa)					
0.0	3.1	2.9	2.2	2.1	2.1	6.4
0.5	10.3	17.2	19.7	23.6	28.0	38.1
0.9	13.5	22.9	29.8	36.4	42.9	56.4
1.4	15.9	27.2	35.7	46.3	53.4	69.0
1.8	17.8	30.8	40.0	53.6	61.2	78.0
2.3	19.1	33.6	43.5	59.1	66.9	84.8
2.7	20.6	35.5	46.5	63.7	72.0	90.1
3.2	21.7	37.4	49.3	67.6	76.4	94.5
3.6	22.5	38.7	51.6	70.8	79.8	98.4
4.1	23.1	39.7	53.7	73.1	82.3	101.8
4.5	23.6	40.5	55.4	75.2	84.3	104.3
5.0	24.0	41.2	56.7	76.5	86.2	106.5
5.4	24.1	41.6	58.0	77.7	87.9	108.4
5.9	24.2	42.1	58.9	78.6	89.1	110.4
6.3	24.3	42.5	59.7	79.0	90.1	111.9
6.8	24.0	42.7	60.5	79.2	90.9	113.5
7.2	23.9	42.7	61.0	79.4	91.3	114.5
7.7	23.8	42.5	61.5	79.3	91.8	115.0
8.1	24.0	42.3	61.6	79.4	92.5	115.1
8.6	24.0	42.0	61.3	79.5	92.5	114.7
9.0	24.0	41.6	60.1	79.4	92.9	114.4
9.5	23.7	41.6	59.1	79.2	92.5	114.1
9.9	23.3	41.4	58.2	78.6	92.2	114.2
10.4	23.1	41.4	57.7	77.7	92.0	114.6
10.8	22.7	41.2	57.5	76.5	91.5	115.0
11.3	22.0	40.7	57.4	74.2	91.2	115.3
11.7	21.8	40.2	57.3	71.2	90.7	115.2
12.2	21.4	39.4	56.9	69.6	90.3	115.6
12.6	21.1	38.3	56.5	68.6	89.8	115.4
13.1	20.9	37.4	56.1	68.2	89.3	115.1
13.5	21.0	36.5	55.5	67.7	88.6	114.4
14.0	21.1	36.2	54.9	67.2	88.1	
14.4	20.8	35.9	54.6	66.9	87.5	
14.9	19.7			66.8		
15.3	19.4			66.8		
15.8	19.3			66.6		
16.2	19.3					

Specimens prepared at water content of 9.2%

	Normal stress (kPa)					
	25	50	75	100	125	150
Displacement (mm)	Shear stress (kPa)					
0.0	2.9	2.0	2.1	1.7	2.3	1.5
0.5	10.8	15.5	8.6	16.3	13.7	24.7
0.9	13.0	19.9	25.7	31.0	36.8	40.9
1.4	14.9	23.3	31.8	39.3	47.7	54.0
1.8	16.2	26.0	35.9	45.6	55.3	63.1
2.3	17.1	28.5	39.2	50.5	61.0	70.4
2.7	18.0	30.1	42.3	54.3	65.9	76.7
3.2	18.9	31.7	44.6	57.6	70.1	82.0
3.6	19.7	33.4	46.9	60.4	73.8	86.5
4.1	20.6	35.0	48.5	62.3	77.1	90.3
4.5	21.2	36.4	50.2	65.0	79.8	93.6
5.0	21.9	37.4	51.9	66.7	82.6	96.9
5.4	22.3	38.3	53.4	68.9	84.9	99.5
5.9	22.7	39.2	54.8	70.5	87.2	102.0
6.3	23.2	40.0	56.1	71.9	89.1	103.9
6.8	23.6	40.6	57.2	72.9	90.4	105.5
7.2	23.8	41.1	58.0	73.7	91.6	107.0
7.7	24.0	41.5	58.6	74.6	92.3	108.0
8.1	24.1	41.7	59.2	75.2	92.9	108.9
8.6	24.3	41.9	59.5	76.1	93.5	109.8
9.0	24.4	42.0	60.0	76.7	94.0	110.7
9.5	24.4	42.1	60.5	77.3	94.9	111.7
9.9	24.4	42.3	60.9	78.0	95.7	112.5
10.4	24.3	42.5	61.6	78.6	96.5	113.4
10.8	24.1	42.7	62.1	79.6	97.4	114.0
11.3	23.9	43.1	62.5	80.2	98.2	115.1
11.7	23.7	43.4	62.6	81.0	99.0	115.9
12.2	23.5	43.8	62.8	81.6	99.4	116.7
12.6	23.3	44.0	62.9	82.0	100.0	117.5
13.1	22.9	44.1	62.9	82.2	100.4	118.2
13.5	22.6	44.0	62.9	82.2	100.8	118.8
14.0	22.3	43.8	63.0		101.2	
14.4	22.0	43.6			101.4	

Specimens prepared at water content of 14.6%

	Normal stress (kPa)					
	25	50	75	100	125	150
Displacement						
(mm)			Shear stress (kPa)			
0.0	1.9	2.0	2.0	1.5	2.3	2.0
0.5	7.9	8.3	12.7	21.0	20.6	32.2
0.9	10.6	16.9	21.4	31.7	35.1	47.1
1.4	12.6	21.2	26.7	38.7	43.7	56.7
1.8	13.9	24.0	30.4	44.1	50.2	63.6
2.3	14.7	25.9	34.1	48.7	55.6	69.8
2.7	15.3	27.6	37.3	52.0	60.7	74.4
3.2	15.8	29.0	39.8	55.6	64.4	78.6
3.6	16.6	30.4	42.3	58.4	67.8	82.1
4.1	17.4	31.7	44.4	60.8	70.3	84.6
4.5	18.1	32.8	46.1	62.7	72.5	87.1
5.0	18.6	34.0	47.6	64.4	74.6	89.4
5.4	19.2	34.9	49.2	66.7	76.6	91.6
5.9	19.6	35.9	50.4	68.0	78.4	93.3
6.3	20.1	36.8	51.8	69.6	80.0	94.7
6.8	20.4	37.5	52.9	70.9	81.6	96.0
7.2	20.8	38.1	54.1	72.1	82.9	97.3
7.7	21.3	38.8	55.2	73.5	84.1	98.8
8.1	21.6	39.4	56.2	74.5	85.1	100.5
8.6	22.0	40.0	57.2	75.4	86.1	101.8
9.0	22.2	40.5	57.9	76.1	87.2	103.1
9.5	22.6	41.0	58.6	76.9	88.2	104.3
9.9	22.8	41.4	59.2	77.8	89.1	105.4
10.4	23.0	41.7	59.8	78.9	89.9	106.4
10.8	23.2	42.2	60.5	79.5	90.8	107.3
11.3	23.3	42.5	61.0	80.2	91.6	108.2
11.7	23.5	42.9	61.6	80.7	92.2	108.9
12.2	23.7	43.3	62.1	81.2	93.0	109.7
12.6	23.7	43.7	62.7	81.7	93.6	110.6
13.1	23.9	44.1	63.3	82.2	94.4	111.7
13.5	23.9	44.3	63.7	82.7	95.0	112.5
14.0	24.0	44.5	64.2	83.1	95.5	113.1
14.4	24.0	44.7	64.6	83.5	96.2	113.5
14.9		44.8	65.0	84.0	96.7	113.6

Specimens prepared at water content of 16.9%

	Normal stress (kPa)					
	25	50	75	100	125	150
Displacement (mm)	Shear stress (kPa)					
0.0	1.5	1.9	1.7	1.6	20.0	1.9
0.5	10.2	14.4	18.1	18.7	37.9	30.8
0.9	12.4	18.8	24.1	28.7	46.1	43.4
1.4	13.8	21.9	29.2	35.6	52.3	53.4
1.8	14.9	24.7	33.1	41.0	57.5	61.1
2.3	15.6	26.6	35.9	45.3	62.3	68.3
2.7	16.7	27.7	38.2	48.8	65.5	74.5
3.2	17.3	29.2	40.6	52.1	68.6	79.7
3.6	18.2	30.4	43.0	55.1	71.0	83.8
4.1	18.7	31.4	45.1	57.6	73.6	87.9
4.5	19.3	32.8	46.6	60.0	76.1	91.3
5.0	19.7	34.1	48.5	62.3	78.0	94.3
5.4	20.0	35.1	49.9	64.4	79.8	97.4
5.9	20.5	35.8	51.2	66.3	81.6	99.5
6.3	21.0	36.7	52.2	68.1	83.1	101.9
6.8	21.3	37.4	53.3	69.4	84.6	103.6
7.2	21.6	38.1	54.0	70.6	85.9	104.9
7.7	22.0	38.8	55.1	72.1	87.2	106.4
8.1	22.2	39.4	56.2	73.5	89.0	107.7
8.6	22.6	40.2	57.0	74.4	90.2	109.0
9.0	22.8	40.8	58.1	75.4	91.9	110.4
9.5	23.1	41.3	59.1	76.2	92.9	111.9
9.9	23.4	41.7	59.6	76.8	93.9	113.0
10.4	23.6	41.8	60.0	77.2	94.7	114.1
10.8	23.8	42.2	60.1	77.7	95.8	114.9
11.3	24.0	42.3	60.5	78.2	96.6	115.1
11.7	24.2	42.5	60.9	78.8	97.0	115.6
12.2	24.4	42.6	61.7	79.4	97.9	116.4
12.6	24.5	42.8	62.5	80.0	98.2	117.3
13.1	24.7	43.1	63.0	80.8	98.7	117.9
13.5	24.9	43.4	63.6	81.5	99.2	118.2
14.0	25.1	43.8	64.4			118.3
14.4	25.3		65.1			118.3
14.9			65.5			118.5

Specimens prepared at water content of 21.1%

	Normal stress (kPa)					
	25	50	75	100	125	150
Displacement						
(mm)	Shear stress (kPa)					
0.0	1.5	2.1	1.5	1.8	1.7	2.3
0.5	8.3	11.3	12.0	30.0	25.5	37.0
0.9	10.5	17.0	20.2	37.5	37.8	49.4
1.4	12.2	20.5	26.8	43.2	46.9	59.7
1.8	13.5	23.3	31.5	47.5	53.9	67.1
2.3	14.7	25.6	35.0	51.1	59.5	73.3
2.7	16.0	27.3	37.2	53.9	63.1	77.8
3.2	16.8	29.0	40.2	56.7	67.5	81.2
3.6	17.5	30.6	42.8	59.1	70.0	83.5
4.1	18.1	31.9	44.8	61.0	72.2	84.8
4.5	17.7	33.2	46.4	62.6	73.5	85.9
5.0	18.3	34.2	48.0	64.1	74.6	86.5
5.4	18.9	35.3	49.0	65.0	75.6	86.8
5.9	19.4	36.2	50.3	66.0	76.3	87.0
6.3	19.9	36.9	51.1	66.7	77.3	87.1
6.8	20.2	37.7	51.6	67.0	77.7	87.1
7.2	20.8	38.3	52.4	67.4	77.8	86.9
7.7	21.0	38.9	52.9	67.8	77.8	87.1
8.1	21.5	39.2	53.4	68.2	78.2	87.3
8.6	21.9	39.8	53.9	68.3	78.6	87.5
9.0	22.3	40.1	54.1	68.3	78.9	86.7
9.5	22.6	40.4	54.4	68.4	79.2	86.5
9.9	22.9	40.9	54.8	68.6	79.9	86.7
10.4	23.2	41.3	55.0	68.8	80.1	86.5
10.8	23.5	41.6	55.4	69.2	80.2	86.5
11.3	24.0	42.1	55.9	69.6	80.3	86.3
11.7	24.2	42.3	56.5	70.1	80.4	86.5
12.2	24.4	42.8	56.9	70.7	80.1	86.7
12.6	24.7	43.0	57.5	71.0	80.2	86.8
13.1	24.8	43.4	57.8	71.6	80.3	86.9
13.5	25.0	43.8		71.8	80.3	87.1
14.0		43.9				87.2

Effect of speed on direct shear strength (under normal stress of 100 kPa for specimen prepared at water content of 14.6%).

		Speed (mm/min)			
0.9		0.036		0.0072	
Displacement	Shear	Displacement	Shear	Displacement	Shear
(mm)	stress (kPa)	(mm)	stress (kPa)	(mm)	stress (kPa)
0.0	1.5	0.2	1.7	0.2	1.8
0.5	1.5	0.4	2.7	0.4	14.1
0.9	21.0	0.6	21.9	0.6	24.5
1.4	31.7	0.9	29.1	0.9	25.3
1.8	38.7	1.1	34.6	1.1	33.4
2.3	44.1	1.3	38.7	1.3	38.9
2.7	48.7	1.5	42.3	1.5	43.8
3.2	52.0	1.7	45.2	1.7	47.7
3.6	55.6	1.9	47.2	1.9	50.9
4.1	58.4	2.2	49.4	2.2	53.6
4.5	60.8	2.4	51.9	2.4	56.2
5.0	62.7	2.6	54.3	2.6	58.9
5.4	64.4	2.8	56.5	2.8	60.4
5.9	66.7	3.0	57.8	3.0	61.8
6.3	68.0	3.2	59.3	3.2	64.3
6.8	69.6	3.5	60.7	3.5	65.6
7.2	70.9	3.7	62.1	3.7	66.3
7.7	72.1	3.9	63.7	3.9	67.8
8.1	73.5	4.1	64.9	4.1	68.4
8.6	74.5	4.3	65.7	4.3	68.1
9.0	75.4	4.5	66.7	4.5	69.6
9.5	76.1	4.8	67.8	4.8	70.7
9.9	76.9	5.0	68.6	5.0	71.9
10.4	77.8	5.2	69.5	5.2	72.5
10.8	78.9	5.4	70.5	5.4	73.3
11.3	79.5	5.6	71.6	5.6	73.3
11.7	80.2	5.8	72.3	5.8	73.9
12.2	80.7	6.0	73.0	6.0	74.5
12.6	81.2	6.3	73.7	6.2	75.3
13.1	81.7	6.5	74.3	6.4	76.0
13.5	82.2	6.7	75.1	6.6	76.3
14.0	82.7	6.9	75.8	6.8	76.7
14.4	83.1	7.1	76.4	7.1	77.0
14.9	83.5	7.3	77.0	7.3	77.9
15.3	84.0	7.6	77.6	7.5	78.2

Effect of speed on direct shear strength continued.

	7.8	78.2	7.7	79.2
	8.0	79.0	7.9	79.5
	8.2	79.4	8.1	79.9
	8.4	80.2	8.4	80.3
	8.6	80.7	8.6	80.8
	8.9	81.4	8.8	81.3
	9.1	81.8	9.0	81.1
	9.3	82.2	9.2	80.9
	9.5	82.5	9.4	81.0
	9.7	82.6	9.6	81.2
	9.9	82.8	9.9	81.4
	10.2	83.3	10.1	81.7
	10.4	83.2	10.3	82.2
	10.6	83.6	10.5	82.5
	10.8	84.0	10.7	83.1
	11.0	84.4	10.9	83.3
	11.2	84.9	11.2	83.3
	11.4	85.2	11.4	83.8
	11.7	85.7	11.6	84.4
	11.9	86.0	11.8	85.0
	12.1	86.4	12.0	85.3
	12.3	86.8	12.2	85.7
	12.5	87.2	12.5	86.0
	12.7	87.6	12.7	86.1
	13.0	87.8	12.9	86.3
	13.2	88.4	13.1	86.4
	13.4	88.7	13.3	86.3
	13.6	89.1	13.5	86.7
	13.8	89.4	13.8	86.9
	14.0	89.7	14.0	87.1
			14.2	87.1

Summary of variation of shear strength with normal stress and gravimetric water content in direct shear tests.

	Moisture content (%)					
Normal stress (kPa)	25.6	50.1	75.7	100.1	124.5	149.3
	Shear stress (kPa)					
3.5	24.2	42.5	63.9	79.0	93.0	116.0
9.2	24.2	44.0	63.0	82.0	102.0	118.0
14.6	24.0	45.0	65.0	84.0	97.0	114.0
16.9	25.0	42.8	65.0	81.0	98.2	118.0
21.1	18.0	40.0	54.0	68.5	78.0	87.0

Appendix D: Identification of triaxial test specimens

Test No.	Compaction water content (%) *	Test state	Shear test type
F0104	20	unsaturated	CW
F0829	20	unsaturated	CW
F0718	20	unsaturated	CW
F0721	20	unsaturated	CW
F0725	20	unsaturated	CW
F0920	20	unsaturated	CW
F0727	20	unsaturated	CW
F1102	20	unsaturated	CW
F1110	20	unsaturated	CW
G0927	20	unsaturated	CW
H0932	20	unsaturated	CW
J0405	20	unsaturated	CD
J0821	20	unsaturated	CD
J0430	20	unsaturated	CD
J0704	20	unsaturated	CD
M0811	16	unsaturated	CD
M0819	16	unsaturated	CW
M0815	16	unsaturated	CD
M0801	16	unsaturated	CW
M0805	16	unsaturated	CD
S0118	20	saturated	CU
S0905	20	saturated	CU
S0123	20	saturated	CU

* Specimens prepared at water content of 20% were about 2% wet of optimum
Specimens prepared at water content of 16% were about 2% dry of optimum

Appendix E: Triaxial specimen data

Specimens prepared wet of optimum

Test No.	F0104	F0829	F0718	F0721
<i>Initial conditions of specimens</i>				
Diameter, (cm)	7.00	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77	538.77
Wet mass, (g)	647.80	647.82	647.21	647.06
Moisture content of sample, (%)	20.2	20.30	20.2	20.1
Specific gravity of solids	2.65	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.20	1.20	1.20	1.20
Dry mass (computed) (g)	538.94	538.50	538.44	538.77
Volume of water in specimen, (ml)	108.86	109.32	108.77	108.29
Volume of solid (computed), (ml)	203.37	203.21	203.19	203.31
Void volume, (ml)	335.40	335.56	335.58	335.46
Volume of air, (ml)	226.53	226.24	226.82	227.17
Degree of saturation, (%)	32.46	32.58	32.41	32.28
Void ratio	1.65	1.65	1.65	1.65
<i>After consolidation</i>				
Volume of air removed, (ml)	0.00	16.10	29.9	57.2
Volume of water removed, (ml)	0.00	0.00	0	0.2
New diameter, D _c (cm)	7.00	6.93	6.87	6.74
New length, L _c (cm)	14.00	13.86	13.74	13.48
New volume, (ml)	538.77	522.67	508.87	481.37
New mass, (g)	647.80	647.82	647.21	646.86
New density (Mg/cu. m)	1.20	1.24	1.27	1.34
New moisture content, (%)	20.20	20.30	20.20	20.06
New void ratio	1.65	1.57	1.50	1.37
Degree of saturation, (%)	32.46	34.22	35.58	38.87
Volume of void, (ml)	335.40	319.46	305.68	278.06
<i>End of shearing (and after oven drying)</i>				
Mass of solids, (g)	537.50	534.27	539.87	539.69
Mass of water in specimen, (g)	108.86	108.30	106.38	105.88
Moisture content of specimen, (%)	20.25	20.27	19.70	19.62
<i>End of shearing (analysis)</i>				
Volume of water left, (ml)	108.86	108.30	106.38	105.88
Volume of solids, (ml)	202.83	201.61	203.72	203.66
Vol. of air removed, (ml)	89.00	129.80	120.32	116.90
Volume of water removed, (ml)	0.00	0.00	0.00	0.00
Final void ratio	1.22	0.95	0.91	0.79
Degree of saturation, (%)	44.18	57.10	57.39	65.70

Specimens prepared wet of optimum

Test No.	F0725	F0920	F0727	F1102
<i>Initial conditions of specimens</i>				
Diameter, (cm)	7.00	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77	538.77
Wet mass, (g)	647.10	647.82	647.89	643.85
Moisture content of sample, (%)	20.10	20.10	20.10	20.50
Specific gravity of solids	2.65	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.20	1.20	1.20	1.20
Dry mass (computed) (g)	538.80	539.40	539.46	534.32
Volume of water in specimen, (ml)	108.30	108.42	108.43	109.53
Volume of solid (computed), (ml)	203.32	203.55	203.57	201.63
Void volume, (ml)	335.45	335.22	335.20	337.14
Volume of air, (ml)	227.15	226.80	226.77	227.61
Degree of saturation, (%)	32.28	32.34	32.35	32.49
Void ratio	1.65	1.65	1.65	1.67
<i>After consolidation</i>				
Volume of air removed, (ml)	80.40	88.70	101.40	135.20
Volume of water removed, (ml)	0.20	0.30	0.30	4.10
New diameter, Dc (cm)	6.63	6.59	6.53	6.34
New length, Lc (cm)	13.26	13.18	13.06	12.67
New volume, (ml)	458.17	449.77	437.07	399.47
New mass, (g)	646.90	647.52	647.59	639.75
New density (Mg/cu. m)	1.41	1.44	1.48	1.60
New moisture content, (%)	20.06	20.04	20.04	19.73
New void ratio	1.25	1.21	1.15	0.98
Degree of saturation, (%)	42.42	43.91	46.31	53.29
Volume of void, (ml)	254.85	246.22	233.50	197.84
<i>End of shearing (and after oven drying)</i>				
Mass of solids, (g)	539.31	537.50	539.72	533.54
Mass of water in specimen, (g)	106.36	110.71	106.48	105.23
Moisture content of specimen, (%)	19.72	20.60	19.73	19.72
<i>End of shearing (analysis)</i>				
Volume of water left, (ml)	106.36	110.71	106.48	105.23
Volume of solids, (ml)	203.51	202.83	203.67	201.34
Vol. of air removed, (ml)	116.90	89.00	89.00	31.00
Volume of water removed, (ml)	0.00	0.00	0.00	0.00
Final void ratio	108.60	0.78	0.71	0.83
Degree of saturation, (%)	77.10	70.42	73.69	63.07

Specimens prepared wet of optimum

Test No.	F1110	G0927	H0932
<i>Initial conditions of specimens</i>			
Diameter, (cm)	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77
Wet mass, (g)	643.57	645.76	649.12
Moisture content of sample, (%)	20.50	20.50	20.50
Specific gravity of solids	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.19	1.20	1.20
Dry mass (computed) (g)	534.08	535.90	538.69
Volume of water in specimen, (ml)	109.49	109.86	110.43
Volume of solid (computed), (ml)	201.54	202.23	203.28
Void volume, (ml)	337.23	336.54	335.49
Volume of air, (ml)	227.74	226.68	225.06
Degree of saturation, (%)	32.47	32.64	32.92
Void ratio	1.67	1.66	1.65
<i>After consolidation</i>			
Volume of air removed, (ml)	162.00	73.90	86.40
Volume of water removed, (ml)	7.90	2.60	5.40
New diameter, Dc (cm)	6.17	6.65	6.58
New length, Lc (cm)	12.34	13.30	13.15
New volume, (ml)	368.87	462.27	446.97
New mass, (g)	635.67	643.16	643.72
New density (Mg/cu. m)	1.72	1.39	1.44
New moisture content, (%)	19.02	20.01	19.50
New void ratio	0.83	1.29	1.20
Degree of saturation, (%)	60.71	41.25	43.10
Volume of void, (ml)	167.33	260.04	243.69
<i>End of shearing (and after oven drying)</i>			
Mass of solids, (g)	534.13	535.68	539.56
Mass of water in specimen, (g)	101.60	105.75	102.84
Moisture content of specimen, (%)	19.02	19.74	19.06
<i>End of shearing (analysis)</i>			
Volume of water left, (ml)	101.60	105.75	102.84
Volume of solids, (ml)	201.56	202.14	203.61
Vol. of air removed, (ml)	5.00	101.40	98.70
Volume of water removed, (ml)	0.00	0.00	0.00
Final void ratio	0.81	0.79	0.71
Degree of saturation, (%)	62.59	66.66	70.93

Specimens prepared wet of optimum

Test No.	J0405	J0821	J0430	J0704
<i>Initial conditions of specimens</i>				
Diameter, (cm)	7.00	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77	538.77
Wet mass, (g)	646.50	646.64	646.5	645.90
Moisture content of sample, (%)	19.90	20.2	20.3	20.10
Specific gravity of solids	2.65	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.20	1.20	1.20	1.20
Dry mass (computed) (g)	539.20	537.97	537.41	537.80
Volume of water in specimen, (ml)	107.30	108.67	109.09	108.10
Volume of solid (computed), (ml)	203.47	203.01	202.79	202.94
Void volume, (ml)	335.30	335.76	335.98	335.83
Volume of air, (ml)	228.00	227.09	226.88	227.73
Degree of saturation, (%)	32.00	32.37	32.47	32.19
Void ratio	1.65	1.65	1.66	1.65
<i>After consolidation</i>				
Volume of air removed, (ml)	0.00	48.5	88.4	110.10
Volume of water removed, (ml)	0.00	0.2	0.8	2.00
New diameter, Dc (cm)	7.00	6.78	6.59	6.48
New length, Lc (cm)	14.00	13.56	13.18	12.95
New volume, (ml)	538.77	490.07	449.57	426.67
New mass, (g)	646.50	646.44	645.70	643.90
New density (Mg/cu. m)	1.20	1.32	1.44	1.51
New moisture content, (%)	19.90	20.16	20.15	19.73
New void ratio	1.65	1.41	1.22	1.10
Degree of saturation, (%)	32.00	37.79	43.88	47.42
Volume of void, (ml)	335.30	287.06	246.78	223.73
<i>End of shearing (and after oven drying)</i>				
Mass of solids, (g)	538.39	538.83	537.74	536.42
Mass of water in specimen, (g)	106.63	107.95	105.72	106.58
Moisture content of specimen, (%)	19.81	20.03	19.66	19.87
<i>End of shearing (analysis)</i>				
Volume of water left, (ml)	106.63	107.95	105.72	106.58
Volume of solids, (ml)	203.17	203.33	202.92	202.42
Vol. of air removed, (ml)	0.00	110.60	90.80	84.60
Volume of water removed, (ml)	0.00	0.00	0.00	0.00
Final void ratio	1.65	0.87	0.77	0.69
Degree of saturation, (%)	31.80	61.17	67.78	76.61

Specimens prepared dry of optimum

Test No.	M0811	M0819	M0815	M0801	M0805
<i>Initial conditions of specimens</i>					
Diameter, (cm)	7.00	7.00	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77	538.77	538.77
Wet mass, (g)	646.30	647.21	647.06	647.10	647.10
Moisture content of sample, (%)	16.1	16	16.2	16.00	16.3
Specific gravity of solids	2.65	2.65	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.20	1.20	1.20	1.20	1.20
Dry mass (computed) (g)	556.68	557.94	556.85	557.84	556.41
Volume of water in specimen, (ml)	89.62	89.27	90.21	89.26	90.69
Volume of solid (computed), (ml)	210.07	210.54	210.13	210.51	209.96
Void volume, (ml)	328.70	328.23	328.64	328.26	328.81
Volume of air, (ml)	239.08	238.96	238.43	239.01	238.11
Degree of saturation, (%)	27.27	27.20	27.45	27.19	27.58
Void ratio	1.56	1.56	1.56	1.56	1.57
<i>After consolidation</i>					
Volume of air removed, (ml)	64.20	76.08	92.8	113.90	106.3
Volume of water removed, (ml)	2.20	1	0.2	0.20	1.1
New diameter, Dc (cm)	6.70	6.65	6.57	6.47	6.47
New length, Lc (cm)	13.40	13.30	13.14	12.93	12.93
New volume, (ml)	472.37	461.69	445.77	424.67	431.37
New mass, (g)	644.10	646.21	646.86	646.90	646.00
New density (Mg/cu. m)	1.36	1.40	1.45	1.52	1.50
New moisture content, (%)	15.70	15.82	16.16	15.96	16.10
New void ratio	1.25	1.19	1.12	1.02	1.05
Degree of saturation, (%)	33.33	35.15	38.20	41.58	40.47
Volume of void, (ml)	262.30	251.15	235.64	214.16	221.41
<i>End of shearing (and after oven drying)</i>					
Mass of solids, (g)	555.98	557.64	556.67	559.26	557.43
Mass of water in specimen, (g)	86.72	89.43	87.01	87.23	86.84
Moisture content of specimen, (%)	15.60	16.04	15.63	15.60	15.58
<i>End of shearing (analysis)</i>					
Volume of water left, (ml)	86.72	89.43	87.01	87.23	86.84
Volume of solids, (ml)	209.80	210.43	210.06	211.04	210.35
Vol. of air removed, (ml)	122.50	100.50	99.10	78.00	78.00
Volume of water removed, (ml)	0.00	0.00	0.00	0.00	0.00
Final void ratio	0.67	0.72	0.65	108.60	108.60
Degree of saturation, (%)	62.03	59.36	63.73	64.06	60.56

Specimens prepared wet of optimum

Test No.	S0118	S0905	S0123	S0224
<i>Initial conditions of specimens</i>				
Diameter, (cm)	7.00	7.00	7.00	7.00
Length, (cm)	14.00	14.00	14.00	14.00
Volume, (ml)	538.77	538.77	538.77	538.77
Wet mass, (g)	642.30	647.50	642.60	642.50
Moisture content of sample, (%)	20.50	20.40	20.50	20.60
Specific gravity of solids	2.65	2.65	2.65	2.65
Specimen density (Mg/cu. m)	1.19	1.20	1.19	1.19
Dry mass (computed) (g)	533.03	537.79	533.28	532.75
<i>After saturation</i>				
Volume of water in specimen, (ml)	337.00	335.30	334.40	335.30
Volume of solid (computed), (ml)	201.14	202.94	201.24	201.04
Void volume, (ml)	337.63	335.83	337.53	337.73
Volume of air, (ml)	0.63	0.53	3.13	2.43
Degree of saturation, (%)	99.81	99.84	99.07	99.28
Void ratio	1.68	1.65	1.68	1.68
<i>After consolidation</i>				
Volume of air removed, (ml)	0.00	0.00	0.00	0.00
Volume of water removed, (ml)	195.30	201.10	216.90	226.10
New diameter, Dc (cm)	6.02	5.99	5.90	5.84
New length, Lc (cm)	12.05	11.98	11.79	11.68
New volume, (ml)	343.47	337.67	321.87	312.67
New mass, (g)	447.00	446.40	425.70	416.40
New density (Mg/cu. m)	1.30	1.32	1.32	1.33
New moisture content, (%)	26.58	24.95	22.03	20.50
New void ratio	0.71	0.66	0.60	0.56
Degree of saturation, (%)	99.56	99.61	97.40	97.82
Volume of void, (ml)	142.33	134.73	120.63	111.63
<i>End of test</i>				
				**
Mass of solids, (g)	534.12	533.02	530.84	533.60
Mass of water in specimen, (g)	142.07	134.26	120.09	110.90
Moisture content of specimen, (%)	26.60	25.19	22.62	20.78
<i>End of shearing (analysis)</i>				
Volume of water left, (ml)	142.07	134.26	120.09	110.90
Volume of solids, (ml)	201.55	201.14	200.32	201.36
Vol. of air removed, (ml)	0.00	0.00	0.00	0.00
Volume of water removed, (ml)	0.00	0.00	0.00	0.00
Final void ratio	0.70	0.68	0.61	0.55
Degree of saturation, (%)	99.82	99.65	99.55	99.35
** test stopped during shearing due to buckling				

Appendix F: Triaxial shear test data

Specimens prepared wet of optimum

Test No.		F0104-CW		
Initial net confining stress (kPa)		0		
Initial matric suction (kPa)		50.8		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	0.00	1.20	50.80	0.00
0.13	6.22	3.17	50.70	0.00
0.27	9.57	3.89	51.30	0.00
0.35	10.59	4.33	51.00	0.00
0.90	14.10	5.50	51.20	0.00
1.16	15.07	6.02	51.00	0.00
2.30	17.61	6.87	51.00	0.00
3.41	18.84	7.78	50.40	0.00
5.04	19.63	8.54	49.70	0.00
6.74	20.12	9.11	49.30	0.00
7.98	20.95	9.48	49.00	0.04
8.14	21.14	9.55	49.10	0.05
9.75	20.84	9.45	48.50	0.18
10.44	21.32	9.31	48.70	0.20
11.76	20.67	9.39	48.60	0.20
12.73	21.04	9.51	48.70	0.21
13.18	21.12	9.34	48.80	0.21
14.73	21.06	9.42	48.60	0.21
15.87	21.12	9.54	48.40	0.22
17.01	21.17	9.06	48.90	0.22
18.15	21.42	9.14	48.90	0.22
19.30	21.44	9.15	48.90	0.22
20.44	21.45	9.15	48.90	0.22
21.58	21.65	9.22	49.00	0.22
23.87	21.61	9.20	48.90	0.22
25.26	21.86	9.19	49.00	0.22
27.29	21.64	8.91	49.70	0.22
33.16	21.93	9.21	50.00	0.22
37.67	22.56	9.12	50.10	0.22
39.96	22.25	9.02	50.40	0.22
43.71	22.47	9.49	50.20	0.22
48.28	22.62	9.84	50.70	0.22
50.57	23.10	10.20	51.00	0.22
54.00	23.39	10.30	51.60	0.25
56.28	23.91	10.47	51.90	0.27

	Test No.	F0829-CW		
Initial net confining stress (kPa)			8.7	
Initial matric suction (kPa)			49.9	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	5.83	9.34	49.90	-0.01
0.16	9.54	10.38	50.20	-0.12
0.33	11.40	10.60	50.60	-0.28
0.49	11.92	10.67	51.10	-0.44
0.84	13.25	11.32	51.10	-0.78
1.70	14.83	12.34	51.10	-1.59
2.71	16.41	13.17	51.10	-2.52
4.89	19.26	14.52	51.00	-4.44
6.54	21.57	15.49	50.60	-5.81
8.51	24.88	16.69	50.40	-7.37
10.50	27.86	17.89	50.00	-8.86
13.31	32.98	19.59	49.90	-10.80
14.27	34.38	20.36	49.60	-11.42
16.05	37.40	21.17	49.50	-12.53
18.49	41.94	22.68	49.10	-13.94
20.85	46.27	24.12	49.00	-15.20
22.54	49.65	25.15	48.50	-16.04
24.31	53.10	26.30	48.10	-16.86
26.07	56.63	27.38	48.20	-17.63
28.89	61.26	29.22	47.60	-18.75
33.27	68.55	31.55	47.00	-20.26
40.04	78.77	34.96	46.80	-22.09
42.35	81.76	35.95	46.60	-22.60
44.66	84.67	37.12	46.40	-23.05
46.97	87.07	37.92	46.20	-23.46
49.28	90.20	39.17	46.10	-23.82
51.59	91.37	39.26	45.90	-24.14
52.74	92.51	39.64	45.90	-24.29
53.90	93.81	40.17	45.60	-24.43
55.25	96.76	41.25	45.70	-24.59
56.40	95.80	40.83	45.50	-24.71
57.56	96.86	40.99	45.60	-24.83

	Test No.	F0718-CW		
Initial net confining stress (kPa)		18.8		
Initial matric suction (kPa)		49.9		
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	6.20	22.47	49.90	0.00
0.36	15.33	24.81	50.40	-0.08
1.21	18.28	25.89	50.00	-0.89
2.84	21.48	26.86	49.80	-2.38
4.24	24.38	27.93	49.30	-3.59
6.09	28.58	29.23	48.80	-5.13
8.08	32.42	30.61	48.20	-6.69
10.35	36.66	31.72	47.80	-8.29
12.43	41.07	33.19	47.50	-9.70
14.45	45.60	34.40	47.30	-10.92
17.02	51.61	36.30	47.10	-12.38
18.66	55.40	37.77	47.10	-13.25
20.29	59.07	38.79	47.00	-14.08
22.37	64.40	40.77	47.10	-15.12
24.70	70.27	42.82	47.00	-16.17
27.03	76.51	44.70	47.00	-17.15
29.55	83.09	47.10	47.40	-18.11
31.28	87.56	48.49	48.00	-18.70
35.68	98.44	52.21	48.30	-19.97
36.60	100.62	53.04	48.20	-20.27
39.59	107.16	55.32	47.70	-21.22
40.74	109.38	55.96	47.40	-21.51
43.04	113.45	57.12	47.30	-21.99
45.37	116.36	58.09	47.10	-22.36
46.54	117.40	58.03	47.50	-22.48
47.70	118.44	58.48	47.50	-22.57
48.87	119.36	58.69	47.70	-22.68
50.03	120.39	59.03	47.70	-22.85
51.20	121.21	59.20	48.10	-22.99

	Test No.	F0721-CW		
Initial net confining stress (kPa)			29.5	
Initial matric suction (kPa)			49.1	
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	5.33	31.28	49.10	0.00
0.07	11.48	33.13	49.30	0.00
0.14	14.27	33.96	49.30	0.00
0.71	19.57	35.42	49.60	-0.45
1.81	22.92	36.74	49.30	-1.54
2.11	23.75	36.92	49.40	-1.83
2.80	25.69	37.36	49.10	-2.44
3.86	29.27	38.56	49.00	-3.34
4.71	31.17	39.09	49.00	-4.04
5.88	34.70	40.27	48.90	-4.98
6.93	37.65	41.35	48.70	-5.78
8.00	40.84	42.51	48.90	-6.55
9.03	43.99	43.76	48.80	-7.26
10.36	47.06	44.69	48.40	-8.17
11.43	49.84	45.61	48.70	-8.85
12.50	53.12	46.61	48.60	-9.51
13.57	56.07	47.79	48.70	-10.14
14.63	59.54	48.95	48.50	-10.79
16.40	64.81	50.80	48.50	-11.78
18.47	70.79	52.70	48.50	-12.84
20.84	77.90	54.87	48.60	-13.99
23.22	85.38	57.76	48.30	-15.06
25.59	93.13	60.04	48.80	-16.02
27.96	100.82	62.71	49.00	-16.89
30.34	108.89	65.40	48.90	-17.88
32.71	116.70	67.90	48.70	-18.88
35.09	125.07	70.59	48.80	-19.70
37.56	132.94	73.41	49.10	-20.49
41.34	143.48	76.83	49.60	-21.51
46.19	153.92	80.51	50.10	-22.49
52.12	161.50	83.03	52.40	-23.14
55.09	163.14	83.78	52.70	-23.46

	Test No.	F0725-CW		
Initial net confining stress (kPa)			37.6	
Initial matric suction (kPa)			49.4	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	4.06	41.25	49.40	0.00
0.09	12.73	44.04	49.90	0.00
0.52	17.62	45.07	50.30	-0.21
0.97	19.91	45.64	50.20	-0.57
1.95	24.18	46.76	50.40	-1.35
2.95	27.89	47.60	50.40	-2.22
3.85	30.72	48.64	50.20	-3.00
4.88	34.40	49.87	50.10	-3.89
5.92	38.05	50.98	50.10	-4.75
6.82	41.12	51.91	50.10	-5.47
7.81	44.16	52.92	49.60	-6.26
8.87	48.02	54.31	49.30	-7.10
9.78	51.03	55.41	49.20	-7.83
10.67	53.73	56.51	49.10	-8.49
11.68	56.91	57.47	48.80	-9.19
12.65	60.34	58.71	48.70	-9.85
13.74	64.26	60.02	48.60	-10.58
14.83	67.56	61.02	48.60	-11.27
16.23	72.63	62.81	48.30	-12.14
18.52	80.99	65.30	48.00	-13.45
20.83	89.25	68.45	47.70	-14.69
21.93	93.13	69.44	47.70	-15.26
23.04	97.71	71.27	47.40	-15.82
24.13	101.13	72.21	47.20	-16.34
25.13	105.57	73.79	47.20	-16.80
26.29	109.72	75.27	47.00	-17.29
27.25	113.44	76.41	46.70	-17.71
28.38	117.14	77.65	46.80	-18.18
29.48	120.98	78.93	46.80	-18.62
30.68	125.05	80.18	46.80	-19.07
32.29	131.08	82.19	46.30	-19.65
33.46	134.51	82.94	46.30	-20.05
34.65	138.91	84.50	46.10	-20.42
35.86	142.36	85.75	45.80	-20.78
37.07	145.59	87.13	45.50	-21.12
37.74	147.58	87.89	45.30	-21.31
40.76	155.36	90.49	44.70	-22.03
43.78	161.87	92.56	44.50	-22.63
46.80	166.24	93.61	44.50	-23.06
49.82	168.95	94.42	44.40	-23.34
52.84	170.66	94.69	44.40	-23.55
55.85	171.05	94.72	44.70	-23.67
58.87	170.37	94.39	44.70	-23.71

	Test No.	F0920-CW		
Initial net confining stress (kPa)			48	
Initial matric suction (kPa)			51	
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	5.85	50.45	51.00	0.00
0.17	11.67	52.39	51.10	0.00
0.63	16.90	53.73	51.30	-0.31
0.97	27.09	56.63	51.80	-0.64
1.80	32.60	58.37	51.50	-1.40
2.87	36.93	59.81	51.40	-2.36
3.72	40.08	60.86	51.30	-3.10
4.24	41.21	61.24	51.20	-3.55
5.46	46.33	62.94	51.10	-4.56
6.67	50.54	64.45	50.60	-5.55
7.89	55.28	65.93	50.50	-6.50
8.50	57.49	66.76	50.40	-6.96
9.72	62.72	68.41	50.40	-7.86
10.94	67.31	69.94	50.00	-8.73
11.55	69.72	70.84	50.10	-9.16
12.77	74.77	72.52	49.70	-9.98
13.96	79.74	74.18	49.40	-10.75
14.26	80.76	74.52	49.50	-10.94
15.75	86.32	76.47	49.20	-11.86
16.90	91.64	78.25	48.90	-12.53
18.04	95.74	79.61	48.60	-13.17
19.76	102.93	82.21	48.10	-14.09
20.90	107.85	83.75	48.10	-14.68
22.16	113.08	85.59	47.90	-15.29
23.27	118.54	87.31	47.80	-15.81
24.41	122.50	88.73	47.40	-16.31
25.56	127.11	90.27	47.10	-16.80
26.70	131.81	91.84	47.00	-17.26
27.80	136.15	93.28	46.70	-17.69
28.93	141.28	94.89	46.70	-18.11
30.36	147.04	96.91	46.30	-18.60
31.93	153.24	98.98	45.90	-19.11
33.61	159.55	101.08	45.80	-19.61
35.12	166.00	103.13	45.50	-20.02
37.17	172.40	105.37	45.00	-20.53
39.48	178.78	107.59	44.60	-21.04
41.54	183.76	109.25	44.10	-21.43
43.97	187.70	110.57	43.70	-21.82
46.40	189.57	111.19	43.30	-22.15
48.83	190.06	111.35	42.70	-22.41
51.26	189.97	111.42	42.60	-22.60

	Test No.	F0727-CW		
Initial net confining stress (kPa)			60.6	
Initial matric suction (kPa)			50.4	
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	3.88	61.79	50.40	0.00
0.09	15.51	65.47	50.50	0.00
0.26	22.64	67.25	51.20	-0.02
0.63	27.08	68.83	51.20	-0.31
0.98	28.55	69.02	51.20	-0.59
1.84	33.56	70.39	51.40	-1.31
2.84	37.97	71.86	51.70	-2.20
3.83	42.67	73.62	51.50	-3.09
4.99	48.52	75.57	51.30	-4.11
5.61	51.12	76.44	51.20	-4.62
6.84	57.49	78.46	51.30	-5.64
7.45	60.36	79.62	50.90	-6.15
8.68	66.37	82.02	50.60	-7.16
9.92	72.86	84.49	50.20	-8.08
10.53	75.63	85.31	50.20	-8.52
11.76	82.85	88.22	50.10	-9.37
12.99	88.80	89.80	50.00	-10.18
13.61	92.31	91.07	49.70	-10.57
14.84	98.93	93.38	49.60	-11.33
15.60	102.55	94.38	49.60	-11.78
16.80	108.98	96.73	49.40	-12.46
17.96	115.29	98.93	49.00	-13.10
19.17	122.00	101.27	48.80	-13.75
20.37	128.52	103.34	48.60	-14.34
21.58	134.86	105.35	48.60	-14.92
22.78	141.30	107.60	48.10	-15.47
23.98	147.77	109.86	48.10	-15.98
25.16	154.88	112.23	47.80	-16.46
26.97	163.44	115.18	47.60	-17.15
28.18	169.92	117.14	47.50	-17.58
29.31	176.78	119.63	47.10	-17.97
30.66	182.42	121.31	47.00	-18.40
32.20	190.86	124.42	46.80	-18.85
34.18	198.93	127.21	46.30	-19.38
36.63	209.20	130.33	45.60	-19.95
39.08	218.84	133.75	45.20	-20.46
41.54	226.09	136.06	45.00	-20.86
43.99	232.00	137.93	44.50	-21.13
46.44	235.01	138.84	44.00	-21.33
48.89	236.01	139.27	43.60	-21.44
51.34	235.17	138.79	43.30	-21.48

	Test No.	F1102-CW		
Initial net confining stress (kPa)		150		
Initial matric suction (kPa)		50.8		
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	7.53	151.51	50.80	0.00
0.07	11.92	152.97	50.60	-0.01
0.15	13.79	153.60	50.60	-0.01
0.82	62.35	169.68	49.40	-0.17
1.87	84.83	177.18	48.00	-0.90
3.03	96.00	181.00	46.60	-1.74
4.42	110.67	185.89	45.30	-2.72
5.79	125.75	190.92	44.10	-3.62
6.47	133.94	193.65	43.30	-4.03
7.83	149.90	198.97	42.20	-4.69
8.52	158.07	201.69	41.20	-4.84
10.56	176.34	207.78	37.90	-4.98
12.61	189.58	212.19	34.50	-5.11
13.97	197.28	214.76	32.10	-5.17
14.99	202.08	216.36	30.50	-5.22
15.72	207.44	218.15	29.40	-5.53
16.94	214.17	220.39	27.80	-5.81
17.54	218.04	221.68	27.30	-5.90
18.76	223.69	223.56	25.70	-6.10
19.98	229.13	225.38	24.50	-6.20
20.59	231.42	226.14	23.50	-6.22
22.42	238.47	228.49	21.50	-6.29
24.86	246.62	231.21	18.30	-6.41
26.69	252.37	233.12	16.40	-6.67
28.52	257.28	234.76	14.60	-6.90
30.35	260.53	235.84	12.70	-6.97
32.83	263.24	236.75	10.10	-7.04
34.07	264.50	237.17	8.80	-7.08
36.54	266.60	237.87	6.60	-7.16
39.02	267.93	238.31	5.00	-7.22
40.25	268.00	238.43	4.00	-7.24
41.49	268.21	238.40	3.40	-7.25
42.73	268.22	238.51	2.50	-7.27
43.97	268.72	238.77	1.50	-7.29
45.21	269.51	238.84	1.20	-7.31
46.44	269.62	238.87	0.40	-7.32
47.68	269.65	238.88	0.5*	-7.34
48.92	269.95	238.98	1.20	-7.38
50.16	270.09	239.03	2.10	-7.41
51.40	270.39	239.23	3.30	-7.44
* commencement of positive pore-water pressures				

	Test No.	F1110-CW		
Initial net confining stress (kPa)			250	
Initial matric suction (kPa)			50.6	
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	16.06	254.35	50.60	-0.03
0.07	21.40	256.13	50.50	-0.03
0.60	70.53	272.51	49.90	-0.07
1.57	156.12	301.04	48.50	-0.26
2.92	184.01	310.34	46.40	-0.30
3.70	197.80	314.93	44.70	-0.35
4.74	214.01	320.34	42.10	-0.40
5.91	228.90	325.30	39.20	-0.46
6.94	238.98	328.66	36.50	-0.50
7.79	245.88	330.96	34.60	-0.53
8.96	254.20	333.73	31.70	-0.58
9.54	257.80	334.93	30.30	-0.60
10.71	265.07	337.36	27.80	-0.63
11.88	272.36	339.79	25.30	-0.66
12.46	276.19	341.06	24.00	-0.67
13.63	282.77	343.26	21.50	-0.70
14.80	289.90	345.63	19.00	-0.72
15.38	292.79	346.60	17.50	-0.73
15.96	296.18	347.73	16.40	-0.74
17.26	302.82	349.94	13.80	-0.76
18.56	309.13	352.04	11.10	-0.78
19.85	315.08	354.03	8.50	-0.80
21.15	320.16	355.72	6.00	-0.83
22.45	324.90	357.30	3.40	-0.85
23.75	329.56	358.85	1.20	-0.87
25.04	333.39	360.23	1.4*	-0.89
26.34	336.90	361.30	3.80	-0.91
27.64	340.35	362.45	6.30	-0.94
28.93	342.72	363.34	8.80	-0.96
30.23	345.50	364.17	10.80	-0.99
32.18	348.49	365.16	14.10	-1.06
33.47	350.34	365.78	16.40	-1.11
34.77	351.34	366.11	18.50	-1.14
36.07	352.70	366.57	21.40	-1.18
37.36	353.95	366.98	23.30	-1.22
38.66	354.63	367.21	25.80	-1.25
39.96	355.18	367.39	27.40	-1.28
41.25	355.75	367.58	29.20	-1.30
42.55	356.18	367.73	31.20	-1.33
43.10	356.69	368.00	31.60	-1.35
44.40	357.48	368.26	32.90	-1.36
* commencement of positive pore-water pressures				

	Test No.		G0927	
Initial net confining stress (kPa)		52.8		
Initial matric suction (kPa)		100.1		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	9.79	52.76	100.10	0.00
0.09	19.85	56.02	100.30	0.00
0.17	23.57	56.86	100.70	0.00
0.26	25.56	57.42	100.80	-0.01
0.34	25.59	58.13	99.80	-0.22
0.43	26.43	58.11	100.40	-0.22
0.51	27.56	58.09	100.70	-0.22
0.79	28.41	58.47	100.40	-0.47
0.88	28.16	59.09	99.70	-0.68
0.97	28.99	58.86	100.10	-0.68
1.53	31.83	59.61	100.40	-1.12
2.12	33.85	60.98	99.40	-1.76
2.83	36.10	61.73	99.20	-2.35
3.98	41.15	63.22	99.20	-3.26
4.47	43.42	64.07	99.10	-3.70
4.94	45.60	64.10	99.40	-3.96
5.40	47.86	65.15	99.30	-4.40
6.69	52.85	67.12	98.50	-5.45
7.34	55.55	67.52	98.70	-5.87
8.51	61.30	69.43	98.70	-6.75
9.39	64.54	70.91	97.80	-7.43
10.11	69.65	72.22	98.00	-7.85
11.20	71.98	73.69	97.00	-8.72
12.28	76.67	75.16	96.80	-9.42
13.37	82.13	76.98	96.40	-10.11
14.46	85.84	78.21	96.10	-10.77
15.07	88.78	78.79	96.40	-11.03
16.19	94.30	80.93	95.80	-11.72
17.15	98.64	82.08	95.80	-12.22
18.18	102.56	83.19	95.80	-12.72
19.28	106.89	84.43	95.40	-13.25

Shear data for specimen G0927 continued.

20.24	111.00	85.90	95.20	-13.73
21.19	116.10	87.60	94.90	-14.20
22.25	119.92	88.87	94.70	-14.69
23.21	124.84	90.61	94.30	-15.17
24.18	128.73	91.71	94.30	-15.55
25.27	132.41	93.84	93.10	-16.15
26.48	138.63	95.11	93.50	-16.48
27.44	142.63	97.04	92.50	-16.94
28.54	147.49	98.06	92.80	-17.28
29.49	151.53	99.41	92.50	-17.60
30.50	155.84	100.85	92.50	-17.95
31.39	159.47	102.16	91.90	-18.24
32.47	163.82	103.51	91.60	-18.58
33.41	167.07	104.59	91.50	-18.87
34.60	170.44	106.51	90.30	-19.35
35.42	173.08	106.59	90.80	-19.43
36.38	176.72	107.71	90.60	-19.68
37.49	180.31	109.00	90.30	-19.95
37.63	181.09	109.36	89.90	-19.98
40.64	189.69	112.13	89.20	-20.62
43.65	197.54	114.75	88.40	-21.15
46.66	202.25	116.32	87.70	-21.56
49.67	205.47	117.49	87.10	-21.85
52.67	206.97	117.99	86.40	-22.05
55.68	207.34	118.11	86.00	-22.15
58.69	205.28	117.53	85.40	-22.19

	Test No.		H0932	
Initial net confining stress (kPa)		51.7		
Initial matric suction (kPa)		150.1		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	7.69	51.76	150.10	0.00
0.09	11.53	52.94	150.00	0.00
0.13	11.82	53.04	150.30	0.00
0.26	13.28	53.43	150.10	0.00
0.31	14.45	53.82	150.00	0.00
0.46	16.79	54.60	149.80	0.00
0.56	14.12	53.61	149.90	0.00
0.67	23.80	56.63	150.10	0.00
0.72	27.61	57.70	150.40	0.00
0.87	36.08	60.03	150.70	0.00
0.93	38.12	60.51	150.90	-0.05
1.40	43.96	62.55	150.50	-0.46
1.77	46.58	63.53	150.30	-0.78
1.92	48.02	63.91	150.30	-0.89
2.72	52.34	65.25	150.10	-1.56
3.82	54.29	65.90	149.50	-2.50
4.47	59.56	68.65	148.30	-3.18
5.15	63.44	68.95	149.20	-3.56
6.54	69.61	71.20	148.40	-4.66
7.95	75.37	73.02	147.90	-5.69
8.68	78.07	73.92	147.60	-6.21
9.41	81.61	75.20	147.30	-6.73
10.87	87.45	77.05	146.80	-7.73
11.97	92.45	78.72	146.10	-8.44
12.70	95.56	79.85	145.90	-8.91
13.98	100.62	81.44	145.50	-9.70
14.34	102.40	82.03	145.40	-9.91
14.71	104.44	82.71	145.00	-10.12
15.07	105.64	83.11	144.70	-10.32
16.23	110.82	84.94	144.10	-10.98
17.44	115.30	86.33	143.60	-11.66
18.57	120.53	88.18	143.20	-12.24
19.71	125.66	89.79	142.50	-12.85
20.92	130.54	91.51	142.00	-13.43

Shear data for H0932 continued.

22.08	135.58	93.09	141.70	-13.96
23.26	141.03	94.91	141.00	-14.51
24.48	145.96	96.55	140.40	-15.03
25.63	151.37	98.36	140.00	-15.54
26.81	156.57	100.09	139.50	-16.05
28.03	161.49	101.83	139.00	-16.53
29.22	166.48	103.39	138.50	-16.98
30.41	171.53	105.18	138.10	-17.41
31.63	176.47	106.82	137.80	-17.80
32.84	181.50	108.50	136.80	-18.22
34.06	186.54	110.08	136.30	-18.63
35.27	190.95	111.55	135.80	-18.96
36.49	195.58	113.19	135.30	-19.29
39.42	205.43	116.38	134.00	-19.99
42.35	213.77	119.26	132.80	-20.60
45.27	220.82	121.61	131.70	-21.10
48.20	225.45	123.15	130.50	-21.49
51.13	227.88	123.96	129.70	-21.77
54.06	228.80	124.27	128.70	-22.00
56.99	227.79	123.93	127.80	-22.12

Test No.		J0405-CD		
Initial net confining stress (kPa)			3.6	
Initial matric suction (kPa)			49.6	
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	7.80	3.60	49.60	0.00
0.07	10.39	4.36	49.60	0.00
0.17	12.45	5.45	49.80	0.00
0.29	13.47	5.59	50.50	0.00
0.51	13.18	5.49	50.10	0.00
0.67	12.91	4.80	50.30	0.00
0.79	13.15	5.58	50.00	0.00
0.95	12.61	5.30	50.20	0.00
0.98	12.61	5.10	50.10	0.00
1.04	12.60	5.20	50.50	0.00
1.13	12.85	5.58	50.30	0.00
1.18	12.84	5.28	50.30	0.00
1.20	12.84	5.48	50.20	0.00
1.21	12.83	5.28	50.20	0.00
1.24	13.34	5.45	50.20	0.00
1.26	13.34	5.55	50.30	0.00
1.27	13.08	5.36	50.20	0.00
1.28	13.08	5.36	50.30	0.00
1.31	13.08	5.46	50.50	0.00
1.33	12.82	5.37	50.20	0.00
1.34	12.82	5.37	50.20	0.00
1.35	12.82	5.27	50.40	0.00
1.37	12.81	5.27	50.20	0.00
1.41	12.30	5.10	50.30	0.00
1.50	13.82	5.51	50.30	0.00

Test No.		J0821-CD		
Initial net confining stress (kPa)			27	
Initial matric suction (kPa)			49.1	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	3.56	26.99	49.10	0.00
0.07	7.66	28.65	48.80	-0.06
0.71	14.50	30.73	48.80	-0.65
0.98	16.68	31.06	49.10	-0.86
1.62	19.45	32.08	49.20	-1.67
2.63	17.01	29.87	50.30	-2.85
3.67	19.78	30.89	50.10	-3.99
4.66	22.26	31.82	50.20	-5.04
5.61	25.02	32.44	49.90	-6.01
6.93	28.31	33.84	50.00	-7.30
8.72	32.65	35.08	50.00	-8.94
9.64	35.07	36.09	50.30	-9.74
11.87	40.36	37.85	50.30	-11.54
12.62	42.73	38.54	50.30	-12.11
13.73	45.02	39.41	50.20	-12.91
14.61	47.30	40.57	50.20	-13.53
15.15	49.36	40.85	50.50	-13.89
16.15	51.57	41.59	50.40	-14.53
17.32	54.48	42.56	50.40	-15.25
18.44	57.61	43.60	50.50	-15.90
19.59	61.17	44.99	50.40	-16.52
20.75	64.65	46.15	50.50	-17.11
21.90	68.03	47.28	50.50	-17.66
23.04	71.05	48.48	50.40	-18.17
24.15	74.50	49.53	50.60	-18.62
25.23	77.61	50.77	50.30	-19.04
26.34	80.84	51.75	50.20	-19.43
27.40	84.23	52.88	50.30	-19.78
28.56	87.17	53.86	50.00	-20.13
29.67	90.98	55.23	50.20	-20.44
30.87	94.31	56.34	50.00	-20.74
32.07	97.92	57.44	50.10	-21.01
33.86	102.44	59.15	50.00	-21.35
36.09	108.80	61.27	49.70	-21.69
38.44	114.86	63.09	49.90	-21.95
40.79	120.08	64.83	50.00	-22.11
43.13	124.64	66.35	49.80	-22.19
45.48	128.71	67.60	49.70	-22.20
47.83	131.98	69.09	49.70	-22.20
50.18	135.16	70.05	49.90	-22.20
52.53	137.31	71.07	49.80	-22.20
54.87	139.09	71.56	49.80	-22.20
57.22	139.23	71.51	50.00	-22.20

Test No.		J0430-CD		
Initial net confining stress (kPa)			50.4	
Initial matric suction (kPa)			50.7	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	8.21	50.44	50.70	0.00
0.36	28.36	55.95	51.80	-0.10
0.67	30.69	56.43	51.70	-0.37
0.86	31.57	56.82	51.50	-0.58
0.97	31.57	57.12	51.40	-0.67
1.06	32.44	57.01	51.50	-0.73
1.12	33.01	56.90	51.90	-0.78
1.20	32.72	57.31	51.70	-0.84
1.26	33.30	57.60	51.90	-0.88
1.33	33.29	57.40	52.00	-0.94
1.37	33.29	57.70	51.90	-0.97
1.44	33.57	57.39	51.80	-1.02
1.46	34.15	57.58	51.90	-1.03
1.49	33.86	57.69	51.70	-1.05
2.06	36.14	58.75	51.40	-1.48
2.50	38.14	59.31	51.50	-1.81
2.99	40.12	59.87	51.20	-2.17
3.88	43.50	60.90	51.20	-2.82
4.74	47.72	62.01	51.20	-3.43
5.59	50.76	63.22	51.60	-4.02
6.73	56.30	65.07	52.00	-4.81
7.49	57.29	65.90	51.40	-5.32
8.28	59.97	66.39	51.80	-5.84
8.86	64.68	68.06	51.30	-6.22
9.43	66.23	68.48	51.20	-6.58
9.94	68.64	69.48	50.90	-6.90
10.91	72.58	70.69	51.00	-7.50
12.12	77.49	72.03	50.70	-8.24
13.09	81.59	73.70	50.90	-8.81
14.06	85.62	75.44	51.10	-9.37
14.55	88.17	75.59	51.20	-9.64
15.04	90.14	76.75	51.50	-9.91
16.04	95.12	78.41	51.60	-10.46
17.05	99.73	79.84	51.20	-10.99
17.87	102.75	80.95	51.00	-11.42
18.97	107.41	82.20	51.50	-11.97
20.01	111.47	83.26	51.40	-12.48
20.92	116.33	85.28	51.70	-12.90
22.02	120.13	86.44	51.00	-13.41
23.02	124.96	88.05	51.20	-13.85
24.04	129.39	89.63	51.50	-14.28

Shear data for J0430 continued.

25.01	135.02	91.41	51.70	-14.68
26.08	138.61	92.70	51.40	-15.10
27.21	143.00	93.97	51.00	-15.53
27.45	143.19	94.23	51.40	-15.62
28.18	147.94	95.31	50.70	-15.89
29.30	151.26	96.62	51.30	-16.27
31.02	158.54	98.95	51.70	-16.84
33.45	168.49	102.16	52.30	-17.55
35.88	177.15	105.25	52.10	-18.18
38.31	184.04	106.75	51.40	-18.72
40.74	190.90	109.53	51.90	-19.18
43.16	196.75	111.18	51.40	-19.54
45.59	200.14	112.61	51.40	-19.82
48.02	202.36	113.15	51.50	-19.95
50.45	203.55	113.95	51.50	-19.99
52.88	203.72	113.61	51.90	-20.04
55.30	203.00	113.67	51.20	-20.09
57.73	201.55	113.28	51.30	-20.13

Test No.		J0704-CD		
Initial net confining stress (kPa)			70.1	
Initial matric suction (kPa)			49.6	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	5.15	71.82	49.60	0.00
0.08	8.79	72.63	49.90	0.00
0.17	14.83	74.84	49.80	0.00
0.29	25.10	78.17	49.70	-0.01
0.60	35.28	81.86	49.20	-0.04
1.12	40.58	83.53	48.80	-0.27
2.08	46.60	85.13	48.00	-1.24
3.55	54.90	87.20	47.20	-2.52
4.54	61.03	88.44	47.60	-3.24
5.53	67.10	90.37	47.60	-3.94
6.51	73.40	92.77	47.50	-4.61
8.15	83.56	96.15	48.20	-5.68
10.27	96.22	100.57	48.90	-6.97
12.49	108.42	104.74	48.60	-8.23
16.44	132.92	113.71	49.30	-10.22
17.61	139.40	116.07	49.50	-10.75
18.34	138.66	115.52	49.90	-11.07
18.96	146.61	118.47	49.70	-11.34
20.19	152.63	120.48	49.10	-11.85
21.40	158.74	122.11	48.80	-12.32
22.60	164.92	123.47	48.40	-12.76
23.74	171.76	125.55	48.50	-13.17
24.96	178.77	127.69	48.70	-13.58
26.19	185.49	129.93	48.90	-13.97
27.34	192.09	132.23	49.30	-14.32
28.32	191.15	132.12	49.90	-14.60
28.94	200.00	134.97	49.10	-14.78
30.17	205.52	136.81	49.00	-15.11
32.02	213.61	138.70	48.90	-15.57
34.49	224.27	142.36	49.20	-16.12
35.72	228.88	144.29	49.40	-16.37
36.96	233.61	145.77	49.80	-16.61
38.19	236.20	146.83	49.10	-16.84
39.42	238.72	147.27	48.80	-17.05
40.66	240.74	147.35	48.80	-17.26
41.89	243.54	148.48	49.00	-17.45
43.12	246.01	149.80	49.60	-17.63
44.36	247.95	151.05	49.80	-17.80
45.59	248.76	151.52	49.70	-17.96
46.82	248.51	151.34	49.20	-18.12
48.06	248.39	151.40	49.00	-18.27
49.29	248.75	151.12	48.90	-18.41
50.53	248.45	150.82	48.60	-18.55
51.76	248.77	150.52	48.70	-18.68
52.99	249.30	150.70	48.70	-18.81
54.23	249.82	150.97	49.00	-18.94
55.46	249.97	151.22	48.90	-19.07

Specimens prepared dry of optimum

Test No		M0811-CD		
Initial net confining stress (kPa)		29.2		
Initial matric suction (kPa)		300.8		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	8.48	32.03	300.80	0.00
0.07	16.11	34.37	301.30	-0.03
0.56	24.60	37.00	301.40	-0.56
1.53	29.99	38.80	301.10	-1.57
2.62	35.08	40.49	301.20	-2.65
3.56	39.02	41.61	301.20	-3.55
4.51	42.95	42.92	301.10	-4.43
5.71	47.68	44.19	301.10	-5.49
6.66	51.26	45.19	301.00	-6.29
8.88	59.41	48.50	301.10	-8.04
9.84	63.15	49.65	301.00	-8.75
10.84	66.55	50.58	300.90	-9.45
11.80	69.92	51.31	300.90	-10.10
12.83	73.48	52.49	301.10	-10.77
14.00	78.57	54.29	300.90	-11.48
16.17	85.80	56.70	301.20	-12.69
18.34	93.73	59.84	300.90	-13.77
20.51	101.23	62.04	301.10	-14.73
22.68	109.03	64.64	300.90	-15.57
24.85	116.03	66.68	300.60	-16.30
27.02	122.98	69.09	301.00	-16.93
29.19	129.31	71.70	301.00	-17.46
33.04	138.66	74.72	300.90	-18.20
37.81	144.46	76.65	300.60	-18.80
42.58	145.96	77.75	299.80	-19.12
47.35	147.51	78.87	299.20	-19.25
52.12	148.59	79.93	298.20	-19.26
53.31	148.47	79.89	298.20	-19.26

	Test No	M0819-CW		
Initial net confining stress (kPa)		38.5		
Initial matric suction (kPa)		301.3		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	6.33	40.61	301.30	0.00
0.17	19.26	44.02	302.00	0.00
1.73	28.96	47.85	301.70	-1.30
2.69	34.41	50.17	301.50	-2.31
3.36	38.12	50.91	301.40	-2.92
4.25	43.48	51.99	301.80	-3.62
5.72	50.27	54.96	301.00	-4.97
6.53	54.39	55.43	302.00	-5.50
7.72	60.77	58.06	301.70	-6.44
9.35	68.67	60.29	301.60	-7.63
11.48	79.01	63.74	301.20	-9.03
13.65	90.18	67.76	301.30	-10.40
15.54	99.76	70.85	300.90	-11.52
19.63	120.30	78.90	300.00	-13.64
22.88	135.14	83.55	299.80	-15.05
24.80	144.60	86.60	300.10	-15.70
26.94	154.03	89.14	300.10	-16.43
29.15	163.58	92.63	299.70	-17.08
31.43	171.46	95.45	299.50	-17.68
34.04	179.44	97.41	299.20	-18.19
36.45	184.02	99.74	299.00	-18.57
40.64	189.14	100.95	298.50	-18.86
43.65	190.36	101.75	298.30	-19.03
46.66	189.82	101.57	298.20	-19.09
49.67	189.68	101.83	297.70	-19.10

	Test No		M0815-CD	
Initial net confining stress (kPa)			51.4	
Initial matric suction (kPa)			299.6	
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	5.90	53.37	299.60	0.00
0.07	13.56	55.62	299.90	0.00
0.56	21.20	56.97	300.40	-0.39
0.82	23.26	57.95	300.40	-0.64
1.24	26.50	59.43	300.50	-1.07
1.81	31.50	60.30	300.40	-1.63
2.37	34.45	61.88	300.70	-2.19
3.40	41.77	63.72	300.40	-3.13
4.32	48.48	65.66	300.40	-3.93
5.27	54.26	67.89	300.60	-4.74
6.79	64.31	71.04	300.50	-5.95
8.97	79.30	76.13	300.40	-7.55
11.12	93.05	81.02	300.40	-8.99
13.31	106.94	85.95	300.40	-10.32
15.55	121.98	91.26	300.20	-11.58
18.67	141.25	96.98	300.40	-13.04
21.82	161.01	103.57	300.60	-14.35
25.13	180.57	110.59	300.60	-15.62
27.39	192.69	114.23	300.60	-16.20
28.28	197.61	116.37	300.40	-16.50
30.18	207.06	119.32	300.20	-17.04
32.30	216.26	121.89	299.90	-17.54
36.79	231.99	127.53	299.70	-18.26
43.41	243.24	130.68	299.70	-18.75
47.38	245.18	132.53	299.10	-18.78

	Test No	M0801-CW		
Initial net confining stress (kPa)		60.4		
Initial matric suction (kPa)		299.9		
Axial	Deviatoric	Mean net	Matric	Volumetric
strain	stress	stress	suction	strain
(%)	(kPa)	(kPa)	(kPa)	(%)
0.00	3.96	61.72	299.90	-0.13
0.18	20.07	65.99	300.90	-0.13
0.53	25.21	67.60	300.40	-0.41
1.48	34.27	68.52	300.30	-1.20
2.86	42.94	71.81	300.50	-2.30
3.66	48.29	73.90	300.40	-2.91
4.56	54.49	76.76	300.40	-3.57
5.98	64.12	79.67	299.50	-4.57
7.60	75.05	82.72	299.30	-5.63
9.63	88.23	88.11	299.10	-6.86
11.66	102.77	91.76	298.90	-7.97
13.69	116.23	96.14	299.20	-8.98
15.79	130.50	101.90	298.80	-9.92
18.73	150.64	107.11	298.70	-11.06
21.66	169.81	113.60	297.90	-12.01
24.02	183.89	119.00	298.00	-12.66
25.47	193.34	122.95	297.90	-13.01
27.10	202.69	124.66	297.60	-13.35
28.78	212.01	128.17	297.60	-13.67
31.59	227.45	135.52	296.30	-14.09
34.68	242.42	138.51	296.90	-14.44
37.16	253.01	142.74	296.50	-14.64
40.25	262.79	145.60	296.20	-14.80
43.35	269.07	146.89	295.90	-14.89
46.44	272.07	148.79	296.10	-14.92
49.54	271.46	147.79	295.50	-14.91
52.64	270.78	149.26	295.40	-14.88
54.18	268.43	146.98	295.10	-14.86

	Test No		M0805-CD	
Initial net confining stress (kPa)		68.2		
Initial matric suction (kPa)		299.1		
Axial strain (%)	Deviatoric stress (kPa)	Mean net stress (kPa)	Matric suction (kPa)	Volumetric strain (%)
0.00	4.87	69.82	299.10	-0.02
0.15	8.81	71.14	299.00	-0.04
0.60	32.48	79.23	299.30	-0.41
0.74	34.30	79.83	299.20	-0.53
1.92	45.47	83.46	299.30	-1.60
2.74	53.29	85.46	299.30	-2.31
3.29	57.49	87.36	299.30	-2.78
4.28	65.54	89.85	299.60	-3.61
6.94	87.31	97.30	300.40	-5.66
8.05	96.53	100.28	300.50	-6.42
9.16	105.94	103.11	300.60	-7.16
10.56	118.02	107.64	300.30	-8.03
11.67	127.11	110.47	300.50	-8.69
12.79	136.92	113.54	300.70	-9.33
13.90	146.55	116.65	300.70	-9.94
14.46	151.60	118.43	300.70	-10.24
15.91	163.78	122.59	300.40	-10.99
17.15	173.90	126.17	300.40	-11.61
18.39	183.76	128.95	300.50	-12.19
19.62	193.69	132.56	299.50	-12.78
20.86	203.90	135.97	299.70	-13.35
22.10	213.48	139.26	299.40	-13.87
23.34	223.01	142.44	299.40	-14.37
24.58	232.47	145.59	299.20	-14.84
25.81	241.56	148.72	299.00	-15.27
27.05	250.75	151.58	298.80	-15.67
28.29	259.30	154.83	298.80	-16.05
29.53	267.66	157.42	298.90	-16.39
30.77	275.30	159.97	298.80	-16.70
32.00	282.41	161.54	298.70	-16.95
33.24	289.29	164.13	298.90	-17.17
34.48	295.60	166.13	299.00	-17.35
35.72	301.65	168.65	299.00	-17.49
36.95	306.44	169.95	299.40	-17.59
38.19	310.75	171.18	299.60	-17.66
41.29	318.51	174.17	300.20	-17.75
44.38	322.53	176.71	299.90	-17.76
47.47	323.24	177.85	298.80	-17.76
50.57	321.33	177.41	299.00	-17.55
53.66	318.25	176.28	299.20	-17.27
56.76	315.42	176.14	297.90	-17.23
59.85	313.94	175.25	298.00	-17.22

	Test No.		S0118-CU
Initial net confining stress (kPa)		50	
Initial matric suction (kPa)		0	
Axial	Deviator	Effective mean	Pore-water
strain	stress	stress	pressure change
(%)	(kPa)	(kPa)	(kPa)
0.00	2.77	49.92	0.00
0.15	12.81	53.27	0.30
0.20	14.88	53.76	0.40
0.47	20.02	54.57	1.30
0.69	21.69	54.33	2.00
0.77	23.74	54.91	2.30
1.16	27.76	54.85	3.60
1.55	29.69	54.00	5.20
1.71	31.01	53.54	6.00
1.81	31.66	53.25	6.40
1.99	32.28	53.16	7.00
2.72	35.75	51.92	9.40
2.95	36.00	51.10	10.10
3.21	37.25	50.82	11.00
3.94	39.30	48.60	13.70
4.20	40.19	48.00	14.80
4.73	41.62	47.07	16.20
5.53	42.91	45.90	17.80
5.79	43.11	45.37	18.50
6.59	44.05	44.38	19.70
7.12	44.44	43.41	20.80
8.70	45.26	41.59	23.00
9.23	45.63	41.01	23.80
10.48	46.24	39.81	25.20
11.01	46.59	39.33	25.70
12.07	46.95	38.65	26.40
12.60	47.57	38.56	26.70
13.13	47.59	38.26	26.90
13.66	47.90	38.17	27.10
14.19	47.90	37.77	27.50

	Test No.		S0905-CU
Initial net confining stress (kPa)			75
Initial matric suction (kPa)			0
Axial strain (%)	Deviator stress (kPa)	Effective mean stress (kPa)	Pore-water pressure change (kPa)
0.00	5.32	73.87	0.00
0.19	16.29	77.13	0.40
0.28	19.46	78.19	0.30
0.47	34.26	82.52	0.90
0.57	41.64	84.58	1.40
0.88	60.50	89.57	2.70
0.97	62.55	89.95	3.10
1.49	68.17	90.12	4.90
1.58	69.15	90.05	5.10
1.72	69.75	89.65	5.70
1.99	70.25	89.32	6.40
2.81	72.43	88.24	8.20
3.07	73.61	87.84	9.00
3.61	72.85	86.78	10.10
3.88	74.35	86.48	11.10
4.15	74.15	85.72	11.40
4.71	74.73	84.61	12.80
5.38	75.88	83.09	14.50
6.72	77.46	79.82	17.90
7.39	78.87	78.69	19.50
8.74	80.32	76.57	22.30
9.41	81.97	75.92	23.60
10.58	82.50	74.10	27.00
11.93	84.07	72.12	29.00
12.60	85.29	71.33	29.80
13.94	85.81	70.00	31.30
14.62	86.35	69.78	32.00
15.96	87.08	69.03	33.00
16.63	87.27	68.59	33.80
17.40	87.64	68.01	34.50
18.74	87.37	66.62	35.10
20.08	87.07	66.22	35.70
21.19	86.13	65.41	36.30
23.20	85.85	64.52	36.70
24.54	86.23	64.04	36.90
25.87	86.81	63.94	37.40
27.88	85.74	63.08	37.70
29.22	85.40	62.47	37.90
30.55	85.02	62.34	38.40
31.89	85.08	62.06	38.80
33.23	84.83	61.98	39.20
34.57	84.29	60.90	39.60
35.90	83.25	59.95	40.00

	Test No.		S0123-CU
Initial net confining stress (kPa)			150
Initial matric suction (kPa)			0
Axial	Deviator	Effective mean	Pore-water
strain	stress	stress	pressure change
(%)	(kPa)	(kPa)	(kPa)
0.00	8.04	152.98	0.00
0.34	25.14	158.38	0.20
0.67	68.65	172.68	0.50
0.80	74.36	174.39	0.80
1.01	80.71	175.90	1.20
1.81	94.42	178.37	3.40
2.82	106.61	178.14	7.80
3.32	111.36	177.32	10.50
3.79	115.04	176.05	12.90
4.87	122.43	172.81	18.40
5.96	128.26	169.55	23.30
6.77	132.60	167.20	26.90
7.86	137.12	164.31	31.30
8.67	140.92	161.97	35.00
9.76	145.18	159.49	39.00
10.85	148.98	156.36	43.00
11.93	153.28	154.19	46.90
12.47	154.58	153.03	48.60
13.56	158.03	150.68	52.20
14.64	161.35	148.78	55.30
15.73	163.62	146.64	58.10
16.82	166.37	144.56	61.00
18.17	168.75	142.55	63.90
19.53	171.25	140.88	66.40
20.89	173.28	139.46	68.50
22.24	175.43	138.08	70.50
23.60	176.83	136.64	71.90
24.95	178.36	136.05	73.20
26.31	179.72	135.21	74.60
27.67	180.38	134.33	75.70
29.02	181.40	133.57	76.70
30.38	181.50	132.90	77.60
31.74	181.96	132.15	78.20
33.09	182.50	131.63	78.70
34.45	182.40	131.10	79.30
35.81	182.85	130.95	80.00
37.16	182.43	130.11	80.70
38.52	182.31	130.17	81.10
39.88	181.81	128.90	81.60
41.23	181.79	128.80	81.70
43.61	181.25	127.82	82.50

Appendix G: Air volume change measurement subroutine

```

10  REM * AIR VOL. MEASUREMENT PROG. FILE : AVM
20  REM * USING THE DPV CONTROLLER
21  PRINT "ENTER DEVICE IEEE ADDRESS I5"
22  PRINT
24  INPUT "",I5
25  PRINT I5
26  IF FRAC(I5)<>0 OR I5<0 OR I5>31 OR I5=21 THEN 21
28  PRINT
31  PRINT "ENTER RESULT FILENAME WITH EXTENSION .PRN "
32  PRINT
40  F$=" "
50  INPUT " ",F$
51  PRINT F$
52  PRINT
60  CREATE F$,10
70  ASSIGN @Avmfile TO F$;FORMAT ON
71  OUTPUT @Avmfile;" Time(secs) Time(minutes) Pressure(kPa) Volume(cc) Temper
ature 'C"
72  OUTPUT @Avmfile;" "
80  ASSIGN @Controller TO 7
90  REM *
100 REM * SET TIMEOUT CONDITION
110 ON TIMEOUT 7,1 GOSUB 409
111 REM *
113 REM * Enter Controller initial volume on display
114 PRINT "INPUT CONTROLLER INITIAL VOLUME AS DISPLAYED ON PANEL"
115 INPUT "",V0
116 REM * INITIALISE O/P FILE
117 OUTPUT @Avmfile USING 401;0;0;0;V0/1000
120 REM *
153 REM * START !
154 PRINT "Select type of interval"
155 PRINT "-----"
156 PRINT "1 = Predetermined intervals"
157 PRINT "2 = Manual interval"
158 PRINT "3 = Stop now. AVM idle"
159 PRINT "Enter choice"
160 PRINT
161 Z=0
162 INPUT "",Z
163 PRINT Z
164 IF FRAC(Z)<>0 THEN 154
165 IF Z<1 OR Z>3 THEN 154
166 PRINT
167 ON Z GOTO 171,328,415
169 !-----
170 REM * Predetermined intervals
171 Id=1
173 REM * SET TO DESIRED TARGET PRESSURE
174 REM *
175 PRINT "Enter desired Target pressure"
176 INPUT "",Tp
177 PRINT Tp
178 PRINT
179 IF FRAC(Tp)<>0 THEN 175
180 IF Tp<0 OR Tp>1500 THEN 175
181 I=Tp
182 SEND @Controller;UNL UNT MTA LISTEN I5 DATA CHR$(4)&VAL$(I)&CHR$(255)
183 PRINT "Wait..."
184 WAIT I5
185 PRINT
186 REM * MEASURE VOLUME CHANGE @ PREDETERMINED INTERVALS
187 BEEP
189 PRINT "GO "

```

```

190 ! MEASURE VOLUME CHANGE IN 15secs TIME INTERVAL <----- 1
191 TO=TIMEDATE
192 Tss=TIME$(TIMEDATE)
193 FOR I1=15 TO 180 STEP 15
194 WAIT 15
195 T=TIMEDATE
196 Elptime=T-TO
197 REM * NOW READ
198 SEND @Controller;UNL UNT MLA TALK IS
199 ENTER @Controller USING "#,B,4D,6D,";S,P,V
200 SEND @Controller;UNT
201 REM * WRITE TO FILE
202 GOSUB 398
203 PRINT
204 PRINT USING "5A,X,5D.DD,4A,X,6A,X,3D.DDD";"Time=";Elptime/60;"mins";" Vol
=";(VO-V)/1000;"cc"
210 PRINT
211 NEXT I1
212 REM *
213 REM * NEW INTERVAL (1 min int) <----- 2
214 REM *
215 FOR I2=240 TO 1800 STEP 60
216 WAIT 60
217 T=TIMEDATE
218 Elptime=T-TO
219 REM * NOW READ
220 SEND @Controller;UNL UNT MLA TALK IS
221 ENTER @Controller USING "#,B,4D,6D,";S,P,V
222 SEND @Controller;UNT
223 REM * Write to file
224 GOSUB 398
225 PRINT
226 PRINT USING "5A,X,5D.DD,4A,X,6A,X,3D.DDD";"Time=";Elptime/60;"mins";" Vol
=";(VO-V)/1000;"cc"
228 PRINT
229 NEXT I2
308 REM * NEW INTERVAL (1hr int) <----- 3
309 REM *
310 FOR I3=5400 TO 86400 STEP 3600
311 WAIT 3600
312 T=TIMEDATE
313 Elptime=T-TO
314 REM * NOW READ
315 SEND @Controller;UNL UNT MLA TALK IS
316 ENTER @Controller USING "#,B,4D,6D,";S,P,V
317 SEND @Controller;UNT
318 REM * Write to file
319 GOSUB 398
320 PRINT
321 PRINT USING "5A,X,5D.DD,4A,X,6A,X,3D.DDD";"Time=";Elptime/60;"mins";" Vol
=";(VO-V)/1000;"cc"
323 PRINT
324 NEXT I3
325 !Tss=TIME$(T)
326 GOTO 345
327 !-----!
328 REM * Manually determined interval
329 Id=2
330 REM * Send command Target pressure to Controller
331 REM *
332 PRINT "Enter desired Target pressure"
333 INPUT "",Tp
334 PRINT Tp

```

```

335 PRINT
336 IF FRACT(Tp)<>0 THEN 332
337 IF Tp<0 OR Tp>1500 THEN 332
338 I=Tp
339 SEND @Controller;UNL UNT MTA LISTEN IS DATA CHR$(4)&VAL$(I)&CHR$(255)
340 PRINT "Wait..."
341 WAIT 5
342 PRINT
343 T0=TIMEDATE
344 Ts=TIME$(TIMEDATE)
345 PRINT "1 = Enter Controller reading"
346 PRINT "2 = Stop data collection and Exit"
347 PRINT "Enter choice now"
348 PRINT
349 Z=0
350 INPUT "",Z
351 PRINT Z
352 PRINT
353 IF FRACT(Z)<>0 THEN 345
354 IF Z<1 OR Z>2 THEN 345
355 ON Z GOTO 358,373
356 REM * Enter reading from Controller to screen and file
357 REM *
358 SEND @Controller;UNL UNT MLA TALK IS
359 T=TIMEDATE
360 ENTER @Controller USING "*,B,4D,6D,";S,P,V
361 SEND @Controller;UNT
362 Elptime=T-T0
363 PRINT USING "5A,X,5D.DD,4A,X,6A,X,3D.DDD";"Time=";Elptime/60;"mins";" Voi
=";(V0-V)/1000;"cc"
364 PRINT
365 PRINT "Enter cylinder temperature"
366 INPUT "",Temp
367 PRINT
368 PRINT Temp
369 PRINT
370 Id=2
371 GOSUB 398
372 GOTO 345
373 PRINT "Are you sure you want to stop ? (Y/N)"
374 Z$=""
375 INPUT "",Z$
376 PRINT Z$
377 PRINT
378 IF Z$<>"Y" AND Z$<>"N" THEN 373
379 IF Z$="N" THEN 345
380 Te$=TIME$(T)
381 !-----!
382 BEEP
383 IF Id=2 THEN 386
384 PRINT "Enter cylinder temperature"
385 INPUT "",Temp
386 OUTPUT @Avmfile;" "
387 OUTPUT @Avmfile;"Result file is..";" ";F$
388 OUTPUT @Avmfile;" "
389 OUTPUT @Avmfile;"Start time is..";Ts$;" Endtime is..";" ";Te$
390 OUTPUT @Avmfile;" "
391 IF Id=2 THEN 393
392 OUTPUT @Avmfile;"Temperature is..";" ";Temp;"deg.C"
393 PRINT
394 PRINT "TEST COMPLETED !! RESULT IN FILE...";F$
395 STOP
396 !-----!

```

```

397 REM *
398 REM * Write data to file
399 IF Id=1 THEN 400
400 OUTPUT @Avmfile USING 401;Elptime;Elptime/60;P;(VO-V)/1000
401 IMAGE X,6D,3X,5D.DD,3X,4D,8X,3D.DDD
402 GOTO 407
404 IF Id=2 THEN 405
405 OUTPUT @Avmfile USING 406;Elptime;Elptime/60;P;(VO-V)/1000;Temp
406 IMAGE X,6D,3X,5D.DD,3X,4D,8X,3D.DDD,8X,2D.DD
407 RETURN
408 REM *
409 REM Timeout subroutine
410 ABORT 7
411 CLEAR 7
412 BEEP
413 PRINT "TIMEOUT !"
414 RETURN
415 END

```

IMAGE EVALUATION TEST TARGET (QA-3)

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