# HYDRAULIC CONDUCTIVITY AND CHEMICAL

## COMPATIBILITY OF SOME VICTORIAN SOILS USED AS

### LINERS FOR WASTE CONTAINMENT

by

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#### **SYNOPSIS**

Wastes generated due to human activities pose a threat to the environment, and their safe and economical disposal is warranted. Landfilling of these wastes is still the most widely practised and viable alternative for their disposal. This is usually accomplished by using engineered waste containment systems in which compacted liners of predominantly clay soils are employed between waste and natural ground to act as barrier to chemical leaching. It follows that the design hydraulic conductivity of the clay is a governing parameter and many environmental regulations require this parameter to be less than  $1x10^{-v}$  m/s. Therefore, the precise assessment of hydraulic conductivity and the compatibility of the liner material with the permeant or leachate encountered, is paramount to ensure the effective performance of the clay liner.

This thesis reports the test results of the compatibility and hydraulic conductivity of some Victorian soils used for liner construction. On the basis of the physical, physicochemical and mineralogical analysis undertaken, all the soils performed adequately as suitable material for liner construction. Moisture content-hydraulic conductivity relationships were investigated from the laboratory test results and published data, and it was established that the moisture content that gives maximum wet unit weight is a closer approximation and better correlated to the moisture content giving minimum hydraulic conductivity. From the quick compatibility assessment using Atterberg limits, it was found from test results that the Atterberg limits, determined with various chemicals, is a function of mineralogical composition and the clay content of soil and physico-chemical properties of the moulding chemicals. The use of high hydraulic gradients to reduce testing time is a common practice for laboratory testing. It was found from the theoretical as well as from the laboratory investigation that hydraulic conductivity is reduced by an insignificant amount of less than a factor of 1.5 when the gradient was increased up to 300.

Hydraulic conductivity and compatibility tests were carried out in the laboratory using different concentrations of methanol and NaCl, and landfill leachate from a local landfill site. This was undertaken on the basis of a sequential testing technique, where the soil specimen was permeated first with water and subsequently with the chemical permeants, using traditional rigid wall permeameters (RWP) and flexible wall permeameters (FWP). It was found from the laboratory investigation that the type of permeameter and the testing technique used might have a significant influence on the hydraulic conductivity of the tested specimen. It was consistently found that the RWPs yielded higher hydraulic conductivity than that of the FWPs. Changes in the hydraulic conductivity on permeation of chemicals were attributed to the structural changes occurring in the soil specimen. On permeation of chemical permeants, the two types of permeameters behave differently with structural changes occurring within the soil specimen. Shrinkage of the specimen, in the case of RWPs, could lead to significant increases in hydraulic conductivity owing to the side wall leakage. In the case of FWPs, soil structural changes can be masked by specimen consolidation or shrinkage due to the effective stress used in testing, and increases in hydraulic conductivity was not observed in FWPs. On permeation of landfill leachate, increases in hydraulic conductivity were observed in the case of RWP, but in the case of FWP hydraulic conductivity showed decreases. Threshold concentration of NaCl was found to be 0.05M solution, beyond which structural changes and the hydraulic conductivity increases were observed. Methanol at high concentration was found to increase hydraulic conductivity up to 200 times in the case of RWPs, but for the same concentrations a negligible increase in hydraulic conductivity was observed in the case of FWPs.

Theoretical analysis of cracking due to the compaction stresses as well as chemicals were carried out. A new technique, no lateral strain boundary condition, reflecting the field boundary condition was proposed for hydraulic conductivity and compatibility testing. This was carried out by gluing the soil specimen with the fixed mould, and allowing the structural changes to occur in the other parts of the specimen. The validation tests of the testing technique were carried out in the laboratory. It was found that the new technique yielded hydraulic conductivity values that fall within the results of RWPs and FWPs.

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#### **STATEMENT**

This thesis contains no material, which has been previously submitted for any other degree or diploma in any University. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person, except where due reference has been made in the text.

Fashiur Rahman

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### **ABBREVIATIONS**

AAS	Atomic Adsorption Spectroscopy
ANZ	Australia New Zealand
AS	Australian Standard
ASTM	American Society for Testing and Materials
BFI	Browning Ferris Industries
BOD	Biochemical Oxygen Demand
BS	British Standard
CCL	Compacted Clay Liner
CCP	Consolidation Cell Permeameter
CEC	Cation Exchange Capacity
СН	High Plastic Clay
CL	Low Plastic Clay
CMP	Compaction Mould Permeameter
COD	Chemical Oxygen Demand
CsCl	Cesium Chloride
CSIRO	Commonwealth Scientific and Industrial Research Organisation
DMSO	Dimethyl Sulphoxide
EC	Electrical Conductivity
EPA	Environment Protection Authority
ESP	Exchangeable Sodium Percentage
esd	Equivalent Spherical Diameter
FLAC	Fast Lagrangian Algorithm of Continuua
FWP	Flexible Wall Permeameter
GCL	Geo-synthetic Clay Liners
IW	Industrial Waste
LL	Liquid Limit
М	Molar Solution
MSW	Municipal Solid Waste

N	Normal Solution
NA	Not Available
NP	Non Plastic
NTU	Natural Turbidity Unit
OMC	Optimum Moisture Content
PI	Plasticity Index
PIC	Pressure Interface Chamber
PL	Plastic Limit
pv	Pore Volume
PWM	Pacific Waste Management
RWP	Rigid Wall Permeameter
SAR	Sodium Adsorption Ratio
SC	Sandy Clay
SEM	Scanning Electron Microscope
VUT	Victoria University of Technology
XRD	X-Ray Diffraction

### NOTATIONS

А	Cross Sectional Area
Å	Angstrom
A <sub>F</sub>	Attractive Unit Force
с	Cohesion
c <sub>s</sub>	Shape Factor of the Soil Particles
c <sub>u</sub>	Undrained Cohesion
c	Effective cohesion
С	Cation Concentration
C <sub>c</sub>	Compression Index
C <sub>k</sub>	Hydraulic Conductivity Index
C <sub>r</sub> .	Recompression Index
d	Effective Particle Size
d <sub>s</sub>	Interplaner Spacing
D	Diameter
e	Void Ratio
ec	Electron Charge
e <sub>0</sub>	Void Ratio at Zero Effective Stress
E	Compression Modulus
F	Force
Gs	Specific Gravity of Soil
h	Hydraulic Head
Н	Elastic Modulus
i	Hydraulic Gradient
i eq	Equivalent Hydraulic Gradient
k	Coefficient of Hydraulic Conductivity
k <sub>B</sub>	Boltzman's Constant

1.

k <sub>field</sub>	Field Hydraulic conductivity
k <sub>i</sub>	Hydraulic Conductivity with Liquid
k <sub>lab</sub>	Laboratory Hydraulic Conductivity
k <sub>Che</sub>	Hydraulic Conductivity with Chemicals
k <sub>sat</sub>	Saturated Hydraulic Conductivity
k <sub>v</sub>	Vertical Hydraulic Conductivity
k <sub>veq</sub>	Equivalent Vertical Hydraulic Conductivity
k <sub>v0</sub>	Hydraulic Conductivity at Zero Effective Stress
k <sub>w</sub>	Hydraulic Conductivity with Water
K	Intrinsic Permeability
K <sub>a</sub>	Active Earth Pressure
K <sub>e</sub>	Coefficient of Earth Pressure at Rest from Elastic Stress Analysis
L	Length
n	Order of Diffraction
n <sub>0</sub>	Electrolyte Concentration
Ν	Rotor Speed of the Centrifuge
p'	Mean Effective Pressure
q	Flow Rate
r	Equivalent Spherical Radius of Particles to Sediment
R	Gas Constant
Rı	Initial Distance of Particles from the Axis of Rotation
$R_2$	Final distance of Particles from the Axis of Rotation
R <sub>F</sub>	Repulsive Unit Force
S	Solute Suction
S <sub>0</sub>	Average Matric Suction
t	Time
t <sub>a</sub>	Time of Acceleration of the Centrifuge to Reach at the Required Speed
t <sub>d</sub>	Time of Desceleration of Centrifuge to Stop
t <sub>s</sub>	Tensile Strength of Soil Specimen

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Т	Absolute Temperature		
T <sub>D</sub>	Diffuse Double Layer Thickness		
u	Pore Water Pressure		
u <sub>a</sub>	Pore Air Pressure		
u <sub>b</sub>	Pore Water Pressure at the Bottom		
u <sub>t</sub>	Pore Water Pressure at the Top		
U <sub>x</sub>	Horizontal Displacement		
v	Darcy Velocity		
W	Moisture Content		
w <sub>k</sub>	Moisture Content at Minimum Hydraulic Conductivity		
Wopt	Optimum Moisture Content		
Wwet	Moisture Content at Maximum Wet Unit Weight		
ε	Dielectric Constant of Pore Fluid		
ε <sub>x</sub>	Horizontal Strain		
$\Delta \epsilon_0$	Dielectric Dispersion		
φ	Angle of Internal Friction		
ф <sup>′</sup>	Effective Angle of Internal Friction		
γ	Unit Weight		
γ <sub>dry</sub>	Dry Unit Weight		
$\gamma_{sat}$	Saturated Unit Weight		
γ <sub>w</sub>	Unit Weight of Water		
Ywet	Wet Unit Weight		
γ	Submerged Unit Weight		
η	Viscosity		
$\eta_w$	Viscosity of water	÷	
$\eta_{_{\parallel}}$	Viscosity of Liquid		
λ	Wave Length of Incident Radiation		
ν	Possoin's Ratio		

ν <sub>c</sub>	Valance of Cations in the Pore Fluid
θ	Angle of Beam Incidence/Reflection
ρ <sub>s</sub>	Density of Particles
ρ <sub>s</sub>	Density of the Medium
$\sigma'_{c}$	Preconsolidation Stress of as compacted specimen
$\sigma'_{h}$	Effective Horizontal Stress
$\sigma_{_{v}}$	Vertical Stress
$\sigma'_{v}$	Effective Vertical Stress
$\sigma_{_{vb}}$	Total Stress at the Bottom
$\sigma'_{_{vb}}$	Effective Stress at the Bottom
$\sigma_{_{ m vt}}$	Total Stress at the Top
$\sigma'_{_{vt}}$	Effective Stress at the Top
$\sigma'_{_{vo}}$	Zero Effective Vertical Stress
$\tau_{c}$	Critical Shear Stress

-

### **1 INTRODUCTION**

#### **1.1 BACKGROUND**

Industralised countries generate enormous quantities of wastes during their day to day activities. Australia is the second highest producer of waste in the world behind the USA on per capita basis. Australians send 18 million tonnes of waste to landfill every year, which is equivalent to 1 Tonne for every man woman and child (Clean Up Australia, 1999). These wastes could be classified into numerous categories such as solid or liquid, and municipal, agricultural or industrial. The safe and economic disposal of these wastes is a major challenge engineers face at the present time. Although waste recycling and reduction is important in the waste management context, it is still not practised widely in Australia. Waste management in Australia has to rely solely on the disposal options available for these wastes. Out of the available disposal options open and ocean dumping, incineration and landfilling are important. The options of open and ocean dumping and incineration are unacceptable due to the environmental concerns. Therefore, the majority of the wastes end up in the ground, for example by means of landfilling. This increases the potential to degrade the natural environment by leaching of various hazardous chemicals leading to pollution of groundwater resource and the groundwater fed surface waters and ecosystems. In the current practice, engineers attempt to counter this problem by using engineered waste containment systems in which compacted liners of predominantly clay soils are employed between the waste and the natural ground to act as barriers to chemical leaching into the ground. Typical waste containment systems include municipal and industrial waste landfills, agricultural evaporation and sewage ponds and mining waste fills and ponds.

The compacted clay liner should be able to retain its integrity and retention capability by restricting the rapid advective migration of various leachates from waste containment facilities over a long period of time; at least during the design lifespan of the system. Therefore, the design hydraulic conductivity of the clay liner is a

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governing parameter, and many environmental regulations require this parameter to be less than 1 x  $10^{-9}$  m/s. But, recent research indicated that hydraulic conductivity of these compacted clay liners could increase by large amounts if the permeation is made with some chemicals specially with hydrocarbons. The increase was also found to be dependent on the mineralogy of the clay content. Therefore, proper attention is required for the selection of a suitable type of soil for an effective liner for specific types of leachate.

Over the last decade or so, clay liner technology has been the subject of on-going research in some industrialised countries, particularly in North America. Although some developments have been made, the technology is largely empirical. In relation to the current practice, one can identify a number of problems that need careful investigation. For instance, the hydraulic conductivity and the compatibility of the compacted clay are important factors for clay liner design, but there is no generally acceptable test method for its determination.

This current research will address this problem by undertaking systematic hydraulic conductivity and compatibility testing of various local soils with different permeants such as salts, leachate and methanol at different concentrations using traditional testing equipment such as fixed wall and flexible wall permeameters. On the basis of the test results a suitable test method for liner material will be suggested which will be used as a guide for the hydraulic conductivity testing, for the liner materials, and for making recommendations for safer risk assessments.

#### **1.2** AIMS OF THE PROJECT

The overall aim of the research program is to undertake a comprehensive laboratory investigation into the measurement of hydraulic conductivity of selected Victorian soils that are used in soil liner construction. Within this general aim, the following specific aims can be identified:

- a study of the mineralogy of soils used for liner construction, and chemical characteristics of leachates in waste containment systems in Victoria.
- assessment of the effects of leachates and representative chemicals on Atterberg limits and hydraulic conductivity of compacted clay, and evaluation of the possibility of using simple test techniques to assess the suitability of soils for liner construction.
- a critical assessment of available laboratory test methods for conductivity measurement, and development of appropriate test set ups.
- evaluation of the suitability of different types of permeameters and testing methods for soil liners in the laboratory, and their relevance to the field situation, and development of guidelines for hydraulic conductivity testing for waste containment systems with particular emphasis on local conditions.

#### **1.3 STRUCTURE OF THE THESIS**

The objective of the thesis is to undertake a comprehensive laboratory investigation into the hydraulic conductivity of the liner clay. The liner hydraulic conductivity is dependent on several factors and a literature synthesis of the influence of these factors will be presented in Chapter 2. Chapter 3 presents the details of soil and leachate sample collection and their characterisation. Soil samples were collected from four different landfill sites from Western region of Melbourne. Leachate samples were collected from the Lyndhurst landfill operated by the Pacific Waste Management (PWM) (the former Browning Ferris Industries (BFI)), Melbourne. Chapter 4 will give a general description of testing methodologies adopted during the entire testing program. Any variations from standard method or details related to any specific tests will be described in the individual chapters. The potential use of Atterberg limits as a quick and relatively simple method of assessment of the compatibility of clay with chemicals and leachates will be presented in Chapter 5.

Hydraulic conductivity testing is normally carried out in the laboratory using elevated hydraulic gradients to reduce the testing time. The effects of high hydraulic gradient application were examined on basis of theoretical and experimental analyses, whose details will be discussed in Chapter 6. The details related to compatibility tests involving the hydraulic conductivity measurement using various chemical permeants will be given in Chapters 7 and 8. In Chapter 9, major mechanisms involving the clay structural changes are reviewed and the theoretical analyses for cracking due to desiccation and structural changes are presented. This will also be devoted to examining any possible effects of likely residual stresses due to compaction process. This is undertaken by preliminary numerical modelling simulating the compaction process. On the basis of the analyses, and the problems of the traditional testing methods discussed in the previous chapters, a new test technique is proposed for more rational compatibility measurement using the hydraulic permeation. Chapter 9 also presents the results obtained from the tests undertaken with this technique. Finally, in Chapter 10, a summary of the work described in the above chapters and the conclusions and recommendations regarding the direction for future work will be briefly presented.

### **2 LITERATURE REVIEW**

#### 2.1 INTRODUCTION

Compacted clay liners (CCL) are commonly used to isolate wastes from natural ground in waste containment facilities. They are primarily designed as hydraulic barriers to reduce the rate of pollution migration from waste into ground water. It follows that the design hydraulic conductivity of the clay liner is a governing parameter and many environmental regulations require this parameter to be less than  $1 \times 10^{-9}$  m/s. Hence, the reliable determination of hydraulic conductivity of the clay is paramount for the design of these facilities. For CCL, laboratory conductivity testing is commonly used for the selection of suitable borrow soils, design of the liners and validation testing during construction.

As stated in Chapter 1, the main objective of the research program is to undertake a comprehensive laboratory investigation into the measurement of hydraulic conductivity of selected Victorian soils used for liner construction. The chapter gives an overview of some of the factors that might influence the effective performance of compacted clay liners. These factors include properties relating to clay and permeants, degree of compaction and moisture content, hydraulic conductivity testing variables and the interaction of clay with organic and inorganic chemicals.

#### 2.2 OVER VIEW OF CLAYEY LINERS

#### 2.2.1 Application of Clayey Liners

The earliest use of compacted clayey soils as hydraulic barriers appears to be in earthen dams for water impoundments. Since then, the specific uses of clayey liners as hydraulic barriers have grown to a wide range of applications. These applications can be found in canal linings (*eg.* Holtz, 1953), landfills and surface impoundments (*eg.* Daniel 1993a), sewerage lagoons (*eg.* Leroueil *et al.*,1992), heap leaching pads (*eg.* Van Zyl, 1982), deposits of mine tailings, chemical liquid storage ponds, leachate storage ponds, and evaporation ponds (*eg.* Daniel, 1984).

Out of the above applications, CCLs used in sanitary landfills have received the most prominent attention in recent times resulting in a vast amount of related literature. According to Daniel (1985 and 1993a), in the United States (US) where many of the recent developments took place, CCLs were not commonly used until the early 1970s, and until about 1982 nearly all of the engineered landfills in the US were constructed of clay liners using a single layer (Bouazza and Van Impe, 1998). In 1982, the Environment Protection Authority of the US (US EPA) banned their use as sole liners for landfills and surface impoundments unless the site owner could prove they were effective. The agency apparently based its ruling largely on studies showing that clay liners failed to meet EPA's criteria of zero penetration and zero leakage of contaminants into the ground (Daniel, 1993a). It appears that this decision was largely fuelled by the earlier findings that some chemicals such as concentrated organic chemicals can affect liner clays, leading to large increases in liner hydraulic conductivity (eg. Anderson, 1982) and the inferior performance of some liners due to poor construction practices (eg. Daniel, 1984). However, the current US environmental legislation allows the use of clayey liners as part of a composite liner system comprising synthetic geomembranes.

Apart from the US, CCLs are widely used in developed and developing parts of the world. Most of the developed countries (*eg.* Canada and European countries) advocate the use of CCLs in combination with geomembranes to form a composite liner system. Australia has also started to follow this approach although much of the landfills in Australia are still clay lined. In the developing countries, most of the landfills are not lined, and the sole CCLs appear to be the most preferred option in the case of new landfills. Nevertheless, with recent growth in economies and the influence of organisations such as the World Health Organisation, the practice of sanitary landfills with mutiliner systems is emerging in some developing countries (*eg.* Scott and Husain, 1996).

As shown schematically in Figure 2.1, CCLs are commonly constructed in sanitary landfills as base (bottom) and sideliners, and top cover (cap). While the primary function of these liner components is to perform as a low conductivity hydraulic barrier, their specific performance requirements can have differences. Christensen *et al.* (1994) have identified the following primary functions for these liner components:

- *Base liners* should slow and retard the leachate migration into the ground, prevent bio-gas from escaping to the environment, provide mechanical support for the waste mass, and avoid accumulation of leachate by means of filtration, drainage and collection system located above the base barrier;
- *Side liners* should slow and retard the leachate migration into the ground, provide mechanical resistance to external water pressures, provide a sufficiently stable contact for overlying materials, and prevent lateral migration of bio-gas;
- *Cover (Top)* should limit the rain and surface water infiltration into the landfill reducing the amount and rate of leachate generation, prevent bio-gas from escaping into the atmosphere, and provide support for aftercare options such as vegetation and erosion control.

In addition to the above performance requirements, liners should be able to accommodate differential settlements, which arise from the consolidation of the founding materials, without significantly compromising the barrier properties. This requirement is particularly important for cover liners because underlying wastes are likely to undergo significant settlement owing to low compaction and/or degradation with time. Nevertheless, the same level of requirement would apply for base liners if the liners are constructed on a section of an existing landfill as in the case of a landfill extension or on a compressible base (Acar and Haider, 1990).

Performance requirements for CCLs used in the majority of the other applications noted previously are similar to base and sideliners used in landfills but the requirements for the cover may be different from base and side liners. It should also be noted that some

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Figure 2.1: Clay Liners: (a) Compacted; (b), (c) and (d) Natural (source: Daniel, 1997)
of the technological advances made for compacted clayey liners are also applicable for slurry walls and cut off walls used in a variety of civil engineering applications.

## 2.2.2 Desirable Properties of Compacted Clay Liner

The primary objective of a CCL is to act as an effective barrier for contaminant migration over its design lifetime. A number of researchers have examined the desirable properties of CCLs (*eg.* Mitchell and Jaber, 1990; Daniel and Wu, 1993; Jessberger, 1995). The most important desirable properties can be summarised as follows:

• Low hydraulic conductivity - This can be regarded as the most important requirement for CCLs. In general, the hydraulic conductivity is directly proportional to the rate of seepage flow of contaminant liquid through the liner. Hence, low hydraulic conductivity will reduce the rate of contaminant liquid release and/or release of contaminants through liquid flow from the waste containment facility and, consequently, the impact on the natural environment would be lessened. The presence of an adequate percentage of clay fractions in the soils used for CCL construction is essential to achieve a low hydraulic conductivity. This property depends on a large number of variables and may feature high variability even under strict controls.

• Low diffusivity - Diffusion is the other known process by which contaminants can migrate through a CCL. This is the process whereby contaminants would migrate from a source of high concentration to points of lower concentration. In simple terms, the contaminant flux rate depends on the concentration gradient and the diffusion coefficient. Hence, as far as the liner desirable properties are concerned, low diffusivity can be achieved by maintaining a low diffusion coefficient within the liner. The diffusion of contaminants can take place without any seepage flow, meaning that achieving a low hydraulic conductivity alone may not limit the rate of contaminant release. Nonetheless, the diffusion (along with dispersion) can occur simultaneously with the seepage flow.

• High attenuation potential - It would be desirable if CCLs could reduce and

prolong the rate of contaminant release and lessen the concentration of contaminants by chemical reaction. These processes, which are generally referred to as the attenuation, include adsorption, precipitation, biological process, oxidation-reduction reactions, acid-base reactions and complex formation and chelation (Dunn, 1983). Analysis of all these processes can be significantly complex. Adsorption is commonly considered with diffusion where the contaminant migration is retarded by this reaction.

• Adequate strength and low compressibility - CCLs should feature adequate strength and low compressibility in order to effectively perform as barriers for contaminant release. These properties are essential to maintain the trafficability of construction equipment during the construction phase. These are also required to maintain the integrity of the liner against the overburden stress imposed by the material above it, and to make the liner stable when employed on slopes, for example, in the side wall of a waste containment facility.

• Long term stability - In waste containment facilities, CCLs are provided to limit the contaminant release into the natural environment, at least during its design lifetime. Hence, the long-term stability of a CCL is important where its desirable properties should not degrade significantly with time. Known mechanisms by which CCLs can degrade with time include creep and, more importantly, the chemical reaction with waste leachates and contaminants. Creep can lead to long term deformations, and if differential settlements were to occur, they can threaten the integrity of the liner. If the potential for chemical reaction leading to degradation of liner properties is low, it is then considered that the liner soils are "compatible" with the wastes to be contained. Hence, the tests of chemical compatibility are imperative in the liner design.

• **Ductility** - Ductility will allow the extension of a CCL without significant cracking due to tensile stresses being introduced. Cracks on the liner can act as preferential flow and contaminant release paths, and significantly compromise the effectiveness of the barrier. A CCL may be subjected to extension in situations such as at the crest of a slope or during uneven settlement. Uneven settlement is particularly important for cover liners, as the landfill can undergo substantial settlement with the degradation of wastes.

• Low shrink/swell potential - CCLs may display different levels of shrink/swell potential depending on a number of factors such as the plasticity and mineralogical properties of clay. It is desirable that this potential is low in order to minimise the probability of desiccation cracking due to moisture changes within the liner. The moisture changes can occur if liners were exposed to atmosphere during construction or to internal temperature gradients, or by evapo-transpiration in the case of cover liners.

• Adequate interface strength - In waste containment systems, the stability of slopes is regarded as a major design consideration. According to slope stability theory, slopes tend to fail along the zones of lowest shear strength. For example, in the well documented case of Kettlemen Hill landfill failure (Seed *et al.*, 1990), slopes are considered to have failed by sliding along the CCL surface. The low interface shear strength between the clay and the geomembrane placed above it appeared to have triggered the slip. Hence, it is important in the slope stability point of view that adequate interface strength is maintained between CCL and other adjacent structural components.

• **Constructability** - The construction of CCLs involves the mixing of soils at a moisture content generally wetter than optimum value and subsequent compaction of wet soils using heavy rollers. Hence, trafficability by various compaction equipment on wet soils is an important issue. The constructability will become a major concern if predominantly high plastic soils are used for the liner.

• Low freeze/thaw effects - In very cold climates, CCLs may undergo significant seasonal temperature changes associated with the freezing and thawing of pore liquids. This process can affect the structure of the clay liner leading to contraction cracking of the liner (Kim and Daniel, 1992). While it may be important to keep these effects to a minimum in liners constructed in very cold climates, this issue may not apply to liners constructed in countries with temperate or tropical climates (such as Australia).

# 2.3 **PROPERTIES OF LINER CLAYS**

#### 2.3.1 General

In the previous section, an overview of the desirable properties of the clay liners for its effective performance was provided. A variety of physico-chemical properties of liner soils including clay minerals will control these properties of CCLs. The important mineralogical properties of clay influencing the effective performance of a CCL will be discussed in the subsequent section. Physico-chemical properties of permeants that affect the performance of CCL are examined in some detail and will also be presented in the following sections.

#### 2.3.2 Clay Mineralogy

Clays are small crystalline particles of one or more members of a small group of minerals. Therefore, mineralogy is a primary factor controlling the size, shape, and physical and chemical properties of soil particles. Hence, it is not surprising that clay mineralogy plays an important role in compatibility assessment of clayey soils with various chemical liquids and leachates. Some countries (*eg.* Canada) undertake mineralogical analysis in the assessment of the suitability of liner clays. Therefore, it seems important that a brief review of the desirable clay mineralogical properties is presented at this point.

Clay minerals are normally formed from weathering and subsequent geological processes of parent materials such as rocks and occur in particles of small size. They are primarily hydrous silicates of aluminum, magnesium and iron, carrying a net negative charge created by either crystal structure imperfections or substitutions or from chemical reactions at the mineral surface (Goldman *et al.*, 1990). This net negative charge plays a significant role in the physico-chemical behaviour of clays because cations in the pore water can get attracted to the surfaces of clay particles creating a diffuse double layer between the clay particles. Therefore, the attractive and the

repulsive forces among the clay particles will depend on the thickness of this double layer.

On the basis of structure, clay minerals fall into a relatively small number of groups. A limited number of different minerals are found in significant abundance in the soils commonly encountered in engineering practice. In the discussion which follows, only the important clay minerals, their properties and relevance to liner clays are considered.

## 2.3.3 Clay Mineral Classification

The clay minerals commonly found in soils belong to the large family termed phyllosilicates. Most clay minerals have a sheet-like layered crystalline structure. These sheet structures consist of two different types of basic units, namely, tetrahedral units and octahedral units. Schematic diagrams of these units along with their salient features are shown in Figure 2.2. The silicon-oxygen unit is called a tetrahedral unit where silicon is tetrahedrally coordinated with four oxygens, with the silicon atom at the centre. In octahedral sheets, aluminum or magnesium ion is octahedrally coordinated with six oxygen or hydroxyl groups forming a hexagonal close packing. The oxygen atoms and hydroxyl groups lie in two parallel planes with Al or Mg atoms between these planes. As shown in Figure 2.2, the octahedral unit is slightly bigger than the tetrahedral unit.

Because of the significant similarity in crystal structure of these units, the interstratification of two or more of these unit types often leads to the formation of layered sheet which constitutes a single clay particle or mineral (Mitchell, 1976). The majority of the clay minerals can be categorised into four groups based on the height of the unit, the composition of the sheets and the kind of inter-sheet bonding (Goldman *et al.*, 1990). These groups are kaolinite, illite, smectite, and chlorite. This grouping is particularly convenient because the members of the same group have comparable behavior. A schematic diagram of different clay mineral groups is given in Figure 2.3 and the main features of these mineral characteristics are summarised in Table 2.1.



Figure 2.2: Clay Mineral: Tetrahedral and Octahedral Units (source: Huag, 1997)



Figure 2.3: Schematic Diagram of Different Clay Minerals (source: Huag, 1997)

Mineral	Type/	CEC *	Specific	Negative	Interlayer	Expansion Index /	Liquid	Plastic	Shrinkage	Activity	Specific	Remarks
	Layer Thickness	(meq / 1000	Surface	charge (/	Bonding	property	limit (%)	limit (%)	limit (%)	(P1/% <2 <i>U</i> m)	gravity	
				of 10 oxygen						<b>4</b>		
Kaolinite	1:1	3 - 15	10 - 20	0 - 0.015	HO - 0	0.06 - 0.20	30 - 110	25 - 40	25 - 29	0.5	2.6 -	k is usually greater than 10 <sup>-6</sup>
_	7.2A				Hydrogen strong	Very inactive					2.68	cm/s, not ideal for clay liner
Halloysite	1:1	5 -10			HO - 0		35 - 55	30 - 45		0.1 - 0.5	2.55 -	Tubular structure can not
Dehydrated	7.2 A				Hydrogen	I					2.56	compact, not ideal for clay
Hydrated	10.1 A	5 - 40	35 - 70		Suous		50 - 70	47 - 60			2.00 - 2.20	
Illite	2:1	10 - 40	65 - 100	0.6 - 1.0	K ions strong	0.21 - 0.15 no	60 - 120	35 - 60	15 - 17	0.5 - 1.0	2.60 -	k is in hetween $10^{-7}$ to
_	10 A					expansion or					3.00	
						contraction						10 <sup>-7</sup> cm/s, can be compacted or
_												consolidated, desirable clay
												mineral for liner
Vermiculite	10.5 - 14	100 -	40 - 80	1 - 1.4	Weak	Limited swelling						Difficult to compact, shrinks
	A	150	prim 870			shrinkage and						when dry , not suitable for liner
			seconda			reduction in CEC						
Smectite	2:1	- 08	50 - 120	0.5 - 1.2	0 - 0 very	0.82.5, swelling,	100 - 900	50 - 100	8.5 - 15	1 - 7	2.35 -	k is in between $10^{-9}$ to $10^{-11}$
Montmoril-	9.6 A	150	primary ,		weak	double layer					2.70	cm/s verv reactive clav _ most
lonite	complete		700 - 840		expanding	shrinkage and						desirable clav for liner
	seperation		secondary		lattice	cracking						chemical compatibility may be
										-		a problem
Chlorite	2:1:1	10 - 40	50		HO - 0	No swelling	44 - 47	36 - 40			2.60 -	Effective non-reactive barrier
	14 A				Hydrogen						2.96	clay
					situity		-					

Table 2.1: Summary of Clay Mineral Characteristics

\* Cation exchange capacity

(Sources: Mitchell , 1976 ; Goldman et al , 1990; Owesis et al, 1990 ; Quigley , 1989)

## Kaolinite Mineral

The kaolinite minerals are derived from a 1:1 arrangement of silica tetrahedral sheet and an alumina octahedral sheet. The bonding between the sheets is dominated by a fairly strong hydrogen ion bond over relatively weak van der Waal forces. This strong bonding makes kaolinite minerals relatively stable or inactive against chemical attack from leachates. Furthermore, this strong bonding can also lead to relatively large particles with a small specific surface area of  $15 \text{ m}^2/\text{g}$  and a characteristic basal spacing of 7.2Å. A small negative charge on kaolinite particles results in a cation exchange capacity of 3-15 meq per 100 gm. Consequently, they have relatively high hydraulic conductivity values (which are rarely less than  $10^{-8} \text{ m/s}$ ) and less adsorptive capacity (Rowe *et al.*, 1995). This leads to a fairly rapid advective transport and little retarding capacity, when hydraulic flow is considered. Therefore, these characteristics do not make it an ideal mineral for compacted clay liner construction, although it is relatively stable against many chemicals and moisture changes.

Halloysite is a tubular form of kaolinite and is available in both nonhydrated and hydrated form. The basal spacing for nonhydrated halloysite is about 7.2Å, which corresponds to the thickness of kaolinite, and hydrated halloysite is about 10.1Å. The difference between these values, 2.9Å, is the approximate thickness of a single layer of water molecules. The cation exchange capacity for halloysite is 5 to 40 meq/100g and the specific surface area of hydrated halloysite is in the range of 35 to 70  $m^2/g$ . Because of its tubular structure, this mineral is difficult to compact and, therefore, hydraulic conductivity values tend to be high and erratic (Quigley, 1989).

#### Illite Mineral

This mineral is derived from stacks of 2:1 three layer units with an octahedral sheet between two silica tetrahedral sheets. These units are held together by a very strong potassium ( $K^+$ ) bond to form stacks of these units. Because of the potassium bonding, only a small unbalanced charge is left in the mineral surface giving a cation exchange capacity of 25 meq/100g and making it a mineral of normal activity. Rowe *et al.* (1995)

have noted that "Illite is efficiently compacted or consolidated to form clayey soils having a hydraulic conductivity of  $10^{-9}$  to  $10^{-11}$  m/s depending on the void ratio. Also, the CEC of 25 meq/100g is adequate to permit abundant adsorption of undesirable species such as heavy metals. Finally, there is no interlayer, c-axis (unit spacing) expansion or contraction possible, so illite is often considered to be one of the most desirable clay minerals for use in engineered clay liners for municipal solid waste".

However, it should be noted that if the interlayer  $K^+$  is leached out by acidic leachates (*eg.* pH about 2 in acidic mine tailings), then the mineral structure would experience an increase in the unit spacing (*ie.* swelling) and transform to a less stable vermiculite mineral (Rowe *et al.*, 1995). The reverse is also possible where the vermiculite mineral can transform to illite by  $K^+$  fixation, where reduction in the unit spacing would occur with the possibility of significant cracking.

Vermiculite is a fairly common mineral with a poorly organised octahedral sheet between two silica tetrahedral sheets. Isomorphous substitution of aluminum for silicon is extensive in the tetrahedral sheet, resulting in a net negative charge on the crystal surface. The positive charge deficiency is larger than that of the layers of divalent cations and water. This larger charge deficiency results in vermiculite having the highest cation exchange capacity of all clay minerals. The primary surface area of vermiculite is 40 to 80 m<sup>2</sup>/g and the secondary surface may be as high as 870 m<sup>2</sup>/g.

### • Smectite Mineral

The smectite group of clay minerals includes 2:1 three layer units comprising of an octahedral sheet in between two silica tetrahedral sheets. The bonding between the layers is provided by van der Waals forces and by cations that may be present to balance charge deficiencies in the structures. These bonds are weak and are easily separated by adsorption of water or other polar liquids. Therefore, these minerals may experience significant variations in the unit spacing (c-axis expansion and contraction), leading to swelling or cracking due to change in the moisture level, as well as when exposed to some chemicals (Goldman *et al.*, 1990). The smectite mineral particles have a large specific surface area of up to 800 m<sup>2</sup>/g and have a high adsorptive capacity and can be

compacted to give very low hydraulic conductivities  $(10^{-11} \text{ to } 10^{-13} \text{ m/s})$ . These characteristics make this mineral highly desirable for compacted clay liner constructions, but one has to be careful about the swelling and cracking they might undergo when exposed to adverse conditions.

Montmorillonite and saponites are the common minerals in this group. But the montmorillonite plays a special role in clay liner construction because this is the most common mineral in natural smectite clays, as well as the mineral used in commercially available bentonite, which is commonly used as an additive for the improvement of clays and sands. Montmorillonite comes in two types: Na montmorillonite and Ca montmorillonite depending on the interlayer cation available. Na montmorillonite has the capacity to adsorb a substantial amount of interlayer water in comparison to the Ca variety. This makes Na montmorillonite significantly more reactive with the potential for a large amount of expansion or shrinkage. Because of these characteristics, Na montmorillonite (commercially bentonite) is commonly used as an additive to improve soils and sands of low hydraulic conductivity values, in the slurry cut of walls, and as a clay mat in geosynthetic clay liners. Ca montmorillonite is generally difficult to mix with soil, but its workability can be improved by activation with Na. With these clay minerals, the thing to watch out for is the potential of one mineral transforming to the other by ion exchange when exposed to leachate. Brandl (1992) has argued that the Ca montmorillonite activated with Na has a higher resistance to leachate and pure Na montmorillonite.

### • Chlorite mineral

Chlorite comprises a 2:1 layer (an octahedral sheet in between two silica tetrahedral sheets similar to Illite or Smectite) which is bonded to another octahedral sheet having Mg, Al or Fe in the central position. This sheet carries a net negative charge due to replacement of Mg by Al in the hydroxyl sheet Chlorite has similar engineering behaviour to illite and is considered to be an effective, non-reactive barrier clay (Rowe *et al.*, 1995). Nevertheless, chlorite minerals in clayey soils are almost always found in association with other clay minerals (Goldman *et al.*, 1990). Chlorites are less attractive than smectite and have a cation exchange capacity of 10 to 40 meq/100g and

## 2.4 CLAY CHEMISTRY

#### 2.4.1 Diffuse Double Layer

Clay chemistry deals with the interactions within a clay-water-electrolyte system with mobile ions. Because the clay particle surfaces generally carry a net negative charge, the cations in the pore liquid are attracted to clay surfaces by electrostatic forces. This results in a layer of cations being held at the clay particle surface (Stern layer) and a diffuse layer of cations (Gouy layer) until the cation concentration approaches that of bulk pore liquid, giving rise to what is commonly referred to as *a diffuse double layer*. As shown in Figure 2.4, the distribution of cation and anion concentrations in the diffused double layer of the bulk pore liquid are such that net negative charge at the clay surface is neutralised, and, at the other end of the layer cations and anions are balanced within the free bulk fluid. Hence, between two adjacent clay particles, the two double layers will be interacting to keep the distance between them.

The most important aspect of the interaction of a clay-water-electrolyte system (*eg.* clay and leachate) is the contraction and expansion of the double layer (Rowe *et al.*, 1995). For example, double layer contraction can cause clay particles to flocculate creating significant free void space and, therefore, leading to a substantial increase in clay hydraulic conductivity. Similarly, expansion in the double layer can cause dispersion of clay particles, thereby reducing the free void space and the hydraulic conductivity.

## 2.4.2 Factors Affecting Double Layer

Based on the Gouy-Chapman theory of the double layer, the distance to the centre of gravity of the diffused layer or the thickness of the double layer (1/K) can approximately be expressed by the following equation (Mitchell, 1976).



Figure 2.4: Diffuse Double Layer (source: Mitchell, 1976)

$$\frac{1}{K} = \sqrt{\frac{\varepsilon k_B T}{8\pi n_0 e^2 V_c^2}}$$
(2.1)

where

 $\begin{aligned} & \mathcal{E} &= \text{dielectric constant of pore fluid} \\ & k_B &= \text{Boltzman's constant (1.38 x 10^{-16} \text{ erg/}^0 \text{K})} \\ & T &= \text{absolute temperature (}^0 \text{K}) \text{ of pore fluid} \\ & n_0 &= \text{electrolyte concentration in bulk pore fluid} \\ & v_c &= \text{valence of the cations in the pore fluid.} \\ & e_c &= \text{electron charge (4.803 x 10^{-10} \text{ esu })} \end{aligned}$ 

Equation 2.1 conceptually highlights the influence of pore fluid characteristics (ion concentration and valence, dielectric constant and temperature) on double layer thickness and hence on the clay structure. These influences are discussed in some detail below.

#### Electrolyte Concentration

The double layer thickness is inversely proportional to the square root of the electrolyte concentration. The double layer is suppressed by an increase in concentration. An increase in concentration reduces the surface potential for the condition of constant surface charge. Interparticle interactions extend to a much greater particle spacing for a low electrolyte concentration than a higher concentration. Swelling of the clay layer is related to double layer interactions and hence swelling is dependent on electrolyte concentration (Mitchell, 1976). Acar *et al.* (1985) stated that a high electrolyte concentration favours flocculation which can lead to an increase in the hydraulic conductivity of compacted clay. High pore salt concentration retards full mobilization of the diffuse double layer thickness, increasing the effective void space for water which can lead to a higher hydraulic conductivity (Rao *et al.*, 1987).

## Cation Valance

Thickness of the double layer will decrease with the increase of cation valance if the solutions have the same molarity and constant surface charge causing the solution to become more flocculent. Cation valance also affects surface potential of an electrolyte. The preferential adsorption of relatively small amounts of di or trivalent cations added to clay have a significant influence on physical properties (Mitchell, 1976). If the cations are changed from monovalent to divalent or trivalent (*eg.*, from Na<sup>+</sup> to Ca<sup>++</sup>) the double layer contracts (Quigley, 1989). Acar *et al.* (1985) remarked that a high ionic valance favours flocculation.

#### • Dielectric Constant of Pore Fluid

Double layer is directly proportional to the square root of the dielectric constant. As shown in Figure 2.5, experiments have indicated that the dielectric constant of permeants can have a significant effect on hydraulic conductivity caused by the changes in the double layer thickness. Increases in hydraulic conductivity up to a million-fold have been observed depending on the type of chemicals. Similar comments were made by a number of investigators (*eg.* Mesri & Olson, 1971; Fernandez & Quigley, 1986). In the Tullamarine landfill site in Victoria, the presence of hydrocarbons is evident from previous research (Shugg, 1994) and may have similar effects on hydraulic conductivity due to changes in the dielectric constant. The low dielectric constant of the organic liquids presumably caused a contraction of the diffused double layer that surrounded the soil particles, which, in turn, caused soil flocculation of the particles. Budhu *et al.* (1991) concluded that the correlation between the dielectric constant and hydraulic conductivity is better than any other physico-chemical properties of the permeant.

### Temperature

If the temperature is increased, the dielectric constant decreases and, consequently, there is a contraction in the double layer. This is due to the effect of temperature on the dielectric constant being more paramount than on the double layer. However, Mitchell (1976) pointed out that for water, the product  $\varepsilon T$  does not decrease markedly as the



Figure 2.5: Hydraulic Conductivity vs Dielectric Constant (source: Fernandez & Quigley, 1985)

temperature is increased. Although the effect of temperature is not that significant on double layer, still the hydraulic conductivity could increase up to 6 times as the temperature is increased from  $25^{\circ}$ C to  $50^{\circ}$ C due to the changes in the viscosity and density (Airey, 1993). Therefore, it is advisable that the temperature should be maintained at a constant level during hydraulic conductivity testing.

All four of the above factors will cause the hydraulic conductivity to increase (sometimes dramatically) if the void ratio remains constant (*ie.* no overall volume change). Besides these, there are factors relating to solution properties such as pH, electrolyte type, composition and exchange phenomena as anion adsorption, selectivity of multivalent ions, and size and specific surface area of the clay particles may also affect the double layer thickness (Sparks, 1986).

**pH**: Clay particles may have hydroxyl (OH) of the chemical structure (*eg.* SiOH) exposed on their surfaces, and may have the tendency to dissociate depending on the pH of the pore fluid (Mitchell, 1976). At high pH (alkaline solutions),  $H^+$  can go into the solution increasing the net negative charge on the clay surface. It can be argued that this would lead to an increase in double layer thickness, hence clay dispersion. In addition, alumina which is exposed at the edges of particles, can ionize positively when exposed to low pH (acidic) fluids. This means that the net negative charge on the clay particles will decrease (and sometimes can become even positive), and particles tend to flocculate from positively charged edges clinging to negatively charged surfaces (Mitchell, 1976). Goldman *et al.* (1990) has pointed out that the clay minerals which derive a net negative charge mainly by chemical reactions on the surface are more likely to be affected by pH in the pore fluid. Accordingly, kaolinite, which appears to derive much of its net negative charge this way, may be more susceptible to change due to pH than other minerals.

**Hydrated Ion Size :** The thickness of the double layer depends on the hydrated ion size of cations. Therefore, from a given cation valence, the thickness of the double layer tends to increase with the increasing hydrated radii of cations (for example, hydrated radii of  $Mg^{++} > Ca^{++} > Na^{+} > K^{+}$ ).

Anion Adsorption : Descriptions given so far assume attraction of cations to the clay particle surface. There may be situations, however, where some anions are attracted to the clay particles (*eg.* edges) and thereby increase the net negative charge on the particle. This tends to increase the double layer thickness and leads to dispersion. For example, this appears to be the main mechanism by which the phosphates act as effective additives for clay dispersion or deflocculation (Mitchell, 1976).

### 2.4.3 Cation Exchange Capacity

The adsorbed cations around the clay particles can be exchanged by other cations in the pore fluid. The cation exchange capacity (CEC) of a soil is the sum of exchangeable cations, and is normally represented as meq/100g of soil. A high CEC generally represents a high clay content and a high attenuation potential for contaminants. Ion of one type can be replaced by another type (eg. Ca<sup>++</sup> for Na<sup>+</sup> or Na<sup>+</sup> for Ca<sup>++</sup>), and the replacibility of the ions depends mainly on valence, a relative abundance of the different ion types and ion size (Mitchell, 1976). For example, monovalent ions are easily replaced by divalent ions. Common cations found in soil have the following descending order of replacing power: Al<sup>+++</sup>> Ca<sup>++</sup> > Mg<sup>++</sup> > K<sup>+</sup> > Na<sup>+</sup>. However, it is possible for this replacing order to be overridden if ions of lower replacing power are present in higher concentrations.

Soils with high CECs (*eg.* Montmorillonite) are more prone to structural change than soils with less CECs (*eg.* Kaolinite). This is because cation exchange can affect the double layer thickness and, in turn, can lead to dispersion or flocculation giving rise to changes in hydraulic conductivity of soils. Replacement of monovalent cation by a divalent cation (*eg.* Na<sup>+</sup> by Ca<sup>++</sup> or Mg<sup>++</sup>) can reduce the double layer thickness. In order to account for the potential for such ion exchange, the quantity defined as Sodium Adsorption Ratio (SAR) is defined:

$$SAR = \frac{Na^{+}}{\left[(Ca^{++} + Mg^{++})/2\right]^{1/2}}$$
(2.2)

SAR is determined by a chemical analysis of pore water after saturation, and is reported in (meq/litre)<sup>1/2</sup>. Another ratio used primarily for exchange potential for Na<sup>+</sup> is the Exchangeable Sodium Percentage (ESP) which is defined as the ratio of Na<sup>+</sup> as a percentage of the total exchange capacity. Because Na<sup>+</sup> is the most common and troublesome cation found in natural soils, SAR and ESP have been found to be good indicators of the stability of clay soil structure. Soils with ESP greater than about 6 percent can disperse in water almost immediately (Gardener and Panatiotopouls, 1990). This has important implications in agricultural science, because natural soils with a high content of adsorbed Na<sup>+</sup> can undergo clogging (reduction in hydraulic conductivity), due to dispersion of clay particles when subject to leaching with water. For compacted clay liners, it is possible for hydraulic conductivity to increase arising from the replacement of Na<sup>+</sup> by cations of higher valence or concentrations in leachates and other chemical liquids.

Erosion is a complex phenomena which involves the structure and chemistry of the soil, as well as the composition of pore and eroding fluid. Erosion occurs when the shear stresses induced by fluid flow on the surface of the cracked zone or exposed zone of core material are great enough to cause the removal of aggregates or flocs of particles. On the basis of erosion properties of soil, Arulanandan and Perry (1983) developed a soil classification system involving dielectric dispersion ( $\Delta \varepsilon_0$ ) and Sodium Adsorption. Ratio (SAR) as shown in Figure 2.6(b). Dielectric dispersion ( $\Delta \varepsilon_0$ ) is a composite index which can characterise soil structure. For any soil if CEC is known, then dielectric dispersion ( $\Delta \varepsilon_0$ ) could be estimated from the relationship between CEC and  $\Delta \varepsilon_0$  as shown in Figure 2.6(a).

## 2.4.4 Interacting Forces Between Clay Particles

Physical as well as physico-chemical behaviour of clays are governed by interacting forces between clay particles. Physical interactions mainly include sliding, rolling, compaction and crushing of soil particles, whereas the physico-chemical interactions include double layer changes, ion exchange and interparticle attraction. The interacting



Figure 2.6(a): Dielectric Dispersion vs Cation Exchange Capacity (source: Arulanandan & Perry, 1983)



Figure 2.6(b): Relationship between Critical Shear Stress, Sodium Adsorption Ratio, and Dielectric Dispersion (source: Arulanandan & Perry, 1983)

forces are generated by electrostatic repulsive forces among double layers, electromagnetic attractive forces within clay particles (van der Waals forces), and forces created by self-weight and externally applied forces and potentials. In a clayey soil medium, these forces compete each other and dominating forces will produce structural changes in the soil. For silts and low plastic clays, the mechanical behaviour is governed predominantly by physical interactions rather than physico chemical factors (Bolt, 1956). Furthermore, physical interactions become more significant than physicochemical interactions when the applied pressure increases and the void ratio decreases. This effect will be evident in Chapters 7 and 8, covering compatibility of clays in which the effective stress and the initial void ratio are shown to play very significant roles in the structural changes of the compacted clays when exposed to saline water and chemical fluids. There have been attempts to combine all these interacting forces in the prediction of clay behaviour (eg. Bolt, 1956; Nagaraj and Murthy, 1983, 1986(a), 1986(b); Hueckel et al., 1997). However, they have not met with significant success, and this is not a surprise considering the complexity of interactions involved. Furthermore, although these theories can explain the clay behaviour in a conceptual sense, natural clays come in mixtures of various mineral types and derivatives. The quantitative prediction of natural clay behaviour using such unified theories appears to be a rather ambitious task at the present moment.

## 2.5 CLAY HYDRAULIC CONDUCTIVITY

Hydraulic conductivity or permeability of a material is a measure of how easy a fluid can flow through a porous media. The movement of water through saturated soil was studied by Darcy (1856), a French scientist, who showed experimentally that under laminar flow there exists a linear relationship between discharge or flow rate q and hydraulic head gradient (i) in the direction of flow, which can be expressed as:

$$q = k \, iA \tag{2.3}$$

where : k = the coefficient of proportionality and is called the "coefficient of hydraulic conductivity"; A = area of cross section normal to the direction of flow; and hydraulic

gradient is defined as i = -dh/dx. This equation is applicable for a one-dimensional condition and can be easily extended for three dimensional conditions. It can be noted that Equation 2.3 does not include the physico-chemical properties of the permeant. Hydraulic conductivity (k) can further be expressed in terms of intrinsic permeability (K) as:

$$K = k \gamma / \eta \tag{2.4}$$

where  $\gamma$  and  $\eta$  are the unit weight and viscosity of the fluid respectively, and K is a material property of the soil which is independent of the physical properties of the permeant. This definition disregards any physico-chemical effects of permeants on hydraulic conductivity.

In Equation 2.3, although k is referred to as a constant of proportionality, it is hardly a constant for clayey soils, and depends on a range of physical factors such as saturation (moisture content), grain size and distribution, the fabric, void ratio (effective stress due to applied pressures), fluid viscosity, and on some physico-chemical factors (pore fluid chemistry, adsorbed ions etc.) as well. Out of these, perhaps saturation has the most paramount effect, and if the flow is saturated, then this effect can be ignored. Based on some simplifying assumptions considering the above physical factors, Kozeny (1927) has shown that k of a saturated soil mass can be expressed as:

$$k = c_s d^2 \frac{e^3}{(1+e)} \frac{\gamma}{\eta}$$
 (2.5)

where  $c_s$  is the shape factor of the soil particles, d is the effective particle size, e is the void ratio,  $\gamma$  is the unit weight of the fluid, and  $\eta$  is the viscosity of the fluid. This equation shows the influence of some of the physical parameters including the factors which apparently allow for the tortousity of the flow path depending on the particle shapes. In some references, that part of k without the ratio  $\gamma/\eta$  is considered to be intrinsic permeability which depends entirely on the pore geometry. Based on this equation, it is possible to relate the hydraulic conductivity with water  $(k_w)$  and the hydraulic conductivity with any other fluid  $(k_l)$  as:

$$\frac{k_l}{k_w} = \frac{\eta_w}{\eta_l} \frac{\gamma_l}{\gamma_w}$$
(2.6)

where the subscripts l and w refer to pore fluid and to water respectively. Equation 2.6 indicates that if only physical factors are considered, the change in k values for pore water to another pore liquid is inversely proportional to the viscosity ratio and is directly proportional to the unit weight (density) ratio. However, the influencing factor involving viscosity ratio and unit weight ratio is usually small. Hence, it appears that only the physical factors including viscosity cannot completely explain the large changes in clay structure and the hydraulic conductivity. Therefore, one has to consider the likely changes in physico-chemical behavior of clay in assessing the change in hydraulic conductivity with other permeant.

### 2.5.1 Soil Porosity and Hydraulic Conductivity

Soil porosity is the most important factor that influences the hydraulic conductivity. According to Mitchell and Madsen (1987), hydraulic conductivity varies with the square of the pore radius. Soil fabric refers to the geometric arrangement of the particles and pore spaces in a soil and it exerts a strong influence on the physical properties of the soil system, in particular its hydraulic conductivity. Three different forms of soil fabric can be identified for fine-grained compacted soil barriers to chemical leachate flow:

- the microfabric relates to a regular aggregation of particles and very small pores within the aggregates through which little fluid will flow.
- the minifabric consists of aggregations and the large inter assemblage pores through which flows will be much greater than through intra-aggregate pores, and
- macrofabric refers to those containing joints, cracks, fissures, silt and sand seams, root holes, varves, laminations and other defects through which the flow rate will be largest in comparison with the other flows.

Fine-grained clayey soils can be described on the basis of particle arrangement as dispersed, aggregated, flocculated and deflocculated particles and are shown in Figure

2.7. Goldman *et al.* (1990) stated that dispersed soil fabric may be developed in isotropic soils whereas flocculated fabric can be created in all depositional environments. In a flocculated soil structure, the clay platelets tend to be edge-to-face oriented and can have a large porosity, leading to a very high hydraulic conductivity. In the dispersed soil, clay platelets are arranged face-to-face resulting in a deposit that is stratified on the microscale and which has a high hydraulic conductivity in horizontal direction than in a vertical direction. Hydraulic conductivity of soils with dispersed fabrics tends to be lower than the hydraulic conductivity of soils with flocculated structures. This is due to the increase in the thickness of the double layer because of the increase in dispersivity of the clay particles. Flocculated clay structures have a smaller thickness of double layer.

The fabric of the compacted clay depends on the strength of the pre-existing fabric units, the soil type and mineralogical composition, compaction effort and the moisture content of the soil. The general effects of disturbance and remoulding at constant moisture content would break down flocculated aggregations, destroy shear planes, eliminate large pores and produce a nonhomogenous arrangement (on a macro-scale) or more dispersed fabric units (Mitchell, 1976). As can be seen from Figure 2.8, moisture content controls the rearrangement of particle or particle groups under the compaction effort. For a given compaction effort and dry density, the soil tends to be more flocculated for compaction on the dry side as compared to the compaction on the wet side. It can also be seen from Figure 2.8 that for a given moulding moisture content, increasing the compactive effort tends to disperse the soil.

Increasing the moisture content tends to increase the inter-particle repulsions thereby permitting a more orderly arrangement of the soil particles to be obtained with a given amount of effort. Increasing the compactive effort at a given moisture content tends to orient the particles into a more nearly parallel arrangement.

Clod size of soil is also very important where the hydraulic conductivity is concerned. Figure 2.9 shows the suggestion made by Olsen (1962), that most of the flow of water in compacted clay occurs in relatively large pore spaces located between pads or clods of the clay rather than between particles of clay within the clods. Compaction moisture



(a) Dispersed and deflocculated



(c) Edge-to-face flocculated and aggregated







(b) Aggregated but deflocculated



(d) Edge-to-edge flocculated but dispersed



and aggregated

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(e) Edge-to-face flocculated (f) Edge-to-edge flocculated and aggregated

(g) Edge-to-face and edge-to-edge flocculated and aggregated

Figure 2.7: Basic Forms of Particle Association (source: van Olphan, 1976)



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Figure 2.8: Effect of Compaction on Soil Structure (source: Lambe, 1958)

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Figure 2.9: Flow of Permeant through Relatively Large Pores Between Clods of Soil (source: Olsen, 1962)

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content is also very important for the cloddy nature of the clay. When a soil is compacted wet of optimum, the soft and wet clods of soil are remoulded which results in smaller interclod voids and lower hydraulic conductivity.

# 2.5.2 Hydraulic Conductivity of Compacted Clay

Compaction is the application of mechanical energy to bring the soil into a dense and more stable condition. Soils in a dense state will have low porosity and a minimum flow path for the fluid to flow through them. Soil loses its natural fabrics and structure during the processing of soils for compaction. Compaction alters the soil fabric which, in turn, affects strongly the hydraulic conductivity of fine-grained cohesive soils. A number of factors are related to the hydraulic conductivity of compacted clays: basic physical properties, moulding moisture content and degree of saturation, method of compaction and compactive effort. These are all discussed in this sub-section.

## • Basic Physical Properties

Hydraulic conductivity is a function of the amount of clay fractions present in the liner material and it is reduced with an increase in the amount of fines. Higher liquid limit, plastic limit and surface activity is associated with soils having a greater quantity of clay particles (Mitchell, 1976). It can be seen from Table 2.1 that soils containing a large quantity of highly swelling minerals generally have a high plasticity index. The liquid limit & the plastic limit reflect the consistency of the structure of cohesive soils and are a good parameter in which modification in the soil structure is detectable. All other factors being equal, more plastic clays should have lower hydraulic conductivity (Day & Daniel, 1985; Mesri & Olson, 1971). As the determination of plasticity index is quite easy and fast, it can be used for a first and qualitative evaluation of clay-permeant compatibility. Change in the Atterberg limit due to the changes of moulding fluid were found by different investigators. A recent experimental study by Rahman et al. (1995) showed that the plasticity index decreases with an increase in concentration of methanol and acetic acid and this will be discussed in more detail in Chapter 5. Several investigators (eg. Ridley et al., 1984; Barbour and Yang, 1993) found that plasticity of the soil is reduced due to the presence of high salt concentration in the moulding fluid. This occurred as a result of a decrease in the liquid limit and a little change in the plastic limit. Similar results were also found in the recent testing program and was reported by Rahman *et al.* (1995). The details of these test programs and test results are given in Chapter 5. Ridley *et al.* (1984) observed a decrease in the optimum moisture content and increases in mass density owing to the presence of high concentration of saline water as a moulding fluid.

#### Moisture Content

Moisture content strongly influences the arrangement of clay particles under the compactive effort used. Lambe (1958) suggested that compacted clays are flocculated when compacted dry of optimum moisture content and dispersed when compacted wet of optimum moisture content. At optimum moisture content, soil can display a combination of flocculated and dispersed fabric. The differences in hydraulic conductivity between the dry side and wet side of optimum, which have the same dry density can increase from one to three log cycles (eg. Mitchell et al., 1965; Aughenbaugh, 1990). As shown in Figure 2.10, a six order of magnitude difference in hydraulic conductivity was found by Benson and Daniel (1990) where the tests were carried out in dry and wet side of optimum. A detailed study on the issue of moisturedensity-hydraulic conductivity relationship is presented in Chapter 3. All investigators agreed that for a good liner, water content should be on the wet side of optimum but it is uncertain how much wet of optimum it should be. The strength of compacted clay decreases with an increase in water content. Daniel & Korener (1993) suggested that the soil must not be placed at too high a water content above optimum because the shear strength may become too low. This may increase the risk of desiccation cracks forming in the drying soil, and ruts may form when construction vehicles pass over the liner.

### Compaction effort

Soil compaction is a process in which the air and, to a lesser degree, the water void volume of a soil mass is reduced. Laboratory compaction tests are used to specify the



Figure 2.10: Hydraulic Conductivity vs Moulding Moisture Content (source: Benson & Daniel, 1990)

compaction requirements for the field. Samples are compacted using a range of compaction energies and a window for moisture content and dry densities are found. The weight and type of compactor have a significant influence on hydraulic conductivity. The increasing weight of the compactor results in a greater compactive effort, more shear deformation, smaller more uniform pores and lower hydraulic conductivity. Mitchell et al. (1965) found that kneading compaction yielded hydraulic conductivity half an order of magnitude lower than static compaction at wet of optimum moisture content having the same dry unit weight. Lower hydraulic conductivity in kneading compaction attributed to larger shear stress that occurs as the foot of the kneading compactor penetrates the soil. As shown in Figure 2.11(a), experience from the laboratory has shown that the type of compactor can affect hydraulic conductivity. Benson et al. (1994) found that rubber tyre rollers produced hydraulic conductivity 4 times higher than sheep foot rollers. As shown in Figure 2.11(b), Daniel and Wu (1993) presented an acceptable zone for quality assurance on the basis of moisture content and dry density considering hydraulic conductivity, volume shrinkage and shear strength as the criteria. This could be used in the field work by plotting the curves and making sure that they fit into the acceptable zone.

### 2.5.3 Waste Leachates and Hydrocarbons

Wastes are generated in the form of solids, sludges, liquids, gases, and any combination thereof. Wastes may contain municipal solid waste (MSW), mineral waste, industrial wastes (IW), dredging waste and agricultural waste.

Leachates are the contaminated liquids generated at the base of landfills of various types of waste containment facilities. These leachates are generated due to the percolation of rain water through the solid waste which rapidly leaches out soluble salts, biodegraded organic products and other contaminants. Therefore, composition of leachates may vary depending on their parent source. It was found from the literature that domestic waste leachates contain a wide range of inorganic and organic chemicals and some of the organic chemicals, along with their properties, are listed in Table 2.3. The presence of these chemicals and hydrocarbons may cause significant changes in



Figure 2.11(a): Effect of Compaction Energy on Hydraulic Conductivity (source: Mitchell *et al.*, 1965)

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Figure 2.11(b): Acceptable Zone Based on Design Objectives for Hydraulic Conductivity, Shrinkage and Shear Strength (source: Daniel & Wu, 1993)

hydraulic conductivity. These are discussed in detail in Sections 2.5.4 and 2.5.5 and details of the test results are presented in Chapter 8.

## 2.5.4 Effect of Inorganic Chemicals on Hydraulic Conductivity

Inorganic chemicals affect the hydraulic conductivity of clay on permeation. Changes in cation valance and electrolyte concentration of the soil water account for some of these influences (Broderick *et al.*, 1990). Inorganic chemicals affect hydraulic conductivity through their effect on soil fabric and porosity owing to the changes in ion concentration, ion exchange, anion adsorption and dissociation (Mitchell and Madsen, 1987). Increases in concentration and cation valance cause flocculation of small particles, resulting in an increase in hydraulic conductivity. Inorganic chemical effects could be broadly categorised into the effects of acids, bases and salts and will be discussed separately.

Acids tend to cause flocculation and may attack crystal lattice of clays, especially the octahedral sheets of minerals by dissolution. Compacted natural clays tested by Simons *et al.* (1984) in rigid wall premeameters, using a 5% solution containing equal parts of hydrochloric acid, sulphuric acid and nitric acid, found an increase in hydraulic conductivity up to an order of magnitude after 120 days. D'Appolonia (1980) found that soil bentonite backfill and slurry wall filter cake material permeated with a 5% solution of hydrochloric acid suffered apparent dissolutioning of clay minerals and an increase in hydraulic conductivity of up to one order of magnitude. Gordon and Forrest (1981) found appreciable changes in hydraulic conductivity for sulphuric acid with pH of 1.5 on compacted clay. Lentz *et al.* (1985) concluded from their test results that dissolution taking place in any given solution is difficult to predict and should be investigated on specific soil being evaluated. Although Gibson (1985) found increases in hydraulic conductivity on acid permeation, he suggested that hydraulic conductivity tests should be conducted on site-specific soil as well as with site-specific fluid.

Inorganic bases tend to disperse and attack mainly the silica tetrahedra sheets of the minerals (Mitchell and Madsen 1987). Permeation with strong alkali increases the net particle negative charge causing particle dispersion which expanded the diffused double

layer, resulting the reduction in the hydraulic conductivity (Rao & Sridharan, 1987; Lentz *et al.*, 1985). Lentz *et al.*, (1985) tested different types of soils with sodium hydroxide and found a decrease in hydraulic conductivity by a factor of 2.5 to 13. In none of the cases did the hydraulic conductivity increase. The most significant decrease in hydraulic conductivity was in magnesium montmorillonite. They explained that the reason could be due to ion exchange where monovalent sodium ion replaces divalent magnesium ion, causing an increase in double layer thickness.

The effect of salt on hydraulic conductivity has already been discussed in Section 2.4.2 under electrolyte concentration. Alther *et al.* (1985) concluded that an increasingly more flocculated and more permeable structure would be expected with increasing aqueous salt solution. A great deal of work has not been done on saline water and further study in this area is necessary. In Australia, about 30-35% of the land is salt affected (Northcote & Skene, 1972). There is evidence of crack formation in saline soils (Hubble *et al.*, 1983) or soils permeated or treated with salt water (Alther *et al.*, 1985), which can significantly influence hydraulic conductivity. Saline soils are also present in Echuca in Victoria, and problems have been encountered regarding clay liners for the ponds. Therefore, soil from Echuca may be a point of interest in hydraulic conductivity testing. The effect of salinity on hydraulic conductivity will be discussed in more detail in Chapter 7.

## 2.5.5 Effect of Leachate on Hydraulic Conductivity

Griffin *et al.* (1976), Daniel and Liljestrand (1984) and Fernandez (1990) observed a decrease in hydraulic conductivity of the soil after permeation of domestic waste leachates. Fang and Evans (1988) found that hydraulic conductivity and the physical properties remained unchanged when the silty clay soil was permeated with landfill leachate. But for high swelling sodium montmorillonite soil, permeation with leachate resulted in significant changes in these properties. Fernandez (1989) found an increase in the amount of Na<sup>+</sup> adsorbed during leachate permeation, which could normally lead to a double layer expansion with a consequent reduction in hydraulic conductivity. He also found a decrease in CEC of the dry soil after leachate permeation. Reduction in

CEC would normally lead to an increase in hydraulic conductivity due to double layer contraction. Therefore, leachate may affect hydraulic conductivity of the clay liner in both instances depending on the clay mineralogy and the constituents of leachate.

Constituents of leachate will depend on the wastes in the landfill, age of the landfill, site surface hydrology, the local climate and the manner in which the facility is operated. Some leachate may comprise either concentrated acids or bases and as discussed earlier these acids and bases could cause dissolution of the clay minerals changing hydraulic conductivity. Results of the analysis for physical and chemical properties of collected leachate from the PWM Lyndhurst site for hydraulic conductivity testing are presented in Table 3.5. Table 2.2 presents the main parameters of the sanitary landfill leachate in a general format. On the basis of the constituents of leachate, a test program was selected and the results will be discussed in detail in Chapter 8.

Parameter	Range
COD (mg/l)	150-100,000
BOD <sub>5</sub> (mg/l)	100-90,000
рН	5.3-8.5
Alkalinity (mgCaCO <sub>3</sub> /l)	300-11,500
Hardness (mgCaCO <sub>3</sub> /l)	500-8,900
NH <sub>4</sub> (mg/l)	1-1,500
N <sub>org</sub> (mg/l)	1-2,000
N <sub>tot</sub> (mg/l)	50-5,000
NO <sub>3</sub> (mg/l)	0.1-50
NO <sub>2</sub> (mg/l)	0-25
P <sub>tot</sub> (mg/l)	0.1-30
PO <sub>4</sub> (mg/l)	0.3-25
Ca (mg/l)	10-2,500
Mg (mg/l)	50-1,150
Na (mg/l)	50-4,000
K (mg/l)	10-2,500
SO <sub>4</sub> (mg/l)	10-1,200
Cl (mg/l)	30-4,000
Fe (mg/l)	0.4-2,200
Zn (mg/l)	0.05-170
Mn (mg/l)	0.4-50
CN (mg/l)	0.04-90
AOX ( $\mu$ gCl/l)	320-3,500
Phenol (mg/l)	0.04-44

 Table 2.2: Chemical Composition of Landfill Leachate (source: Andreottola and Cannas, 1994)
#### 2.5.6 Effect of Organic Chemicals on Hydraulic Conductivity

The significant properties of some of the organic chemicals are listed in Table 2.3. Among the properties listed in Table 2.3, dielectric constant is the one which significantly affects hydraulic conductivity. As can be seen from Figure 2.5, chemicals having low dielectric constants can have very high values of hydraulic conductivity. Some of these hydrocarbons listed in Table 2.3 have very low (value of 2 compared to 80 for water) dielectric constant and can give few orders of magnitude higher values of hydraulic conductivity than with water. Dipole moment is used to express the polarity of the molecule. Organic liquids which are less polar than water, are less strongly bonded to the soil minerals and can flow more easily through the pore network (Budhu *et al.*, 1991). When the organic chemicals are allowed to pass through the soil sample, the lower dipole moment causes an increase in hydraulic conductivity. The molar volume is the ratio of molecular weight to the density of a compound and may be useful in the interpretation of test results for hydraulic conductivity.

The permeant viscosity plays an important role, leading to decreases in hydraulic conductivity for dilute to moderate concentrations of organics soluble in water. The viscosity of the solution increases with the increasing content of organics up to a certain level and then decreases again. Fernandez and Quigley (1988) found that there is a substantial increase in hydraulic conductivity if the concentration of organic chemicals is more than 70%. At low concentration, the flow of permeant through the clay is controlled by the permeant viscosity but at high concentration the effects of viscosity are reduced and double layer contraction increases hydraulic conductivity (Quigley and Fernandez, 1992). A number of researchers (*eg.* Mitchell and Madsen, 1987; Bowders and Daniel, 1987) also came to similar conclusions. It can be seen from Equation 2.5 that the viscosity of the permeant is inversely proportional to the hydraulic conductivity. Therefore, the chemicals which have lower values of viscosity may have a very high value of hydraulic conductivity. Solubility of the permeant is also important in hydraulic conductivity testing. Fernandez and Quigley (1985) found from the test results that, for water wet specimens, permeation of water insoluble liquid hydrocarbons

did not change hydraulic conductivity because of surface tension effect, even though the dielectric constant is very low.

Compound	Absolute	Density	Dielectric	Dipole	Molar	Water
Name	viscosity	(kg/m <sup>3</sup> )	constant	moment	volume	solubility
	(x10-3			(debyes)	(x10-4	$(kg/m^{3})$
	N-s/m2)				m <sup>3</sup> /mole)	
Acetic acid	1.16	1049.2	6.15	1.04; 1.74	0.572	~~~~
Acetone	0.316	789.9	20.7	1.66; 2.9	0.735	~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~
Aniline	3.71	1021.7	6.89	1.55	0.911	36
Benzene	0.65	876.5	2.274	0	1.02	0.8
Methanol	0.547	791.4	32.63	1.70	0.405	~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~~
Ethanol	0.2	789.3	24.30	1.69	0.584	~
Phenol	12.7	157.6	13.13	1.45	0.89	86
Water	1.01	997.07	80.4	1.83	0.181	997.07

**Table 2.3:** Properties of Water and Common Organic Chemicals Present in DomesticWaste Leachate (source: Mitchell and Madsen, 1987; Shackelford, 1994)

Mitchell and Madsen (1987) compiled a wide range of hydraulic conductivity test results with different organic chemicals at varying concentrations. Slurry and compacted specimens of different soil types were tested in either FWP or RWP. It could be seen from the test results that, in most cases, hydraulic conductivity increases in RWPs and the maximum increase was up to three orders of magnitude higher than with the corresponding values of water. In all cases, where an abrupt increase in hydraulic conductivity occurred, there were large shrinkage in volume, pores or cracks were observed. In case of FWPs, hydraulic conductivity decreases with the exception of specimen prepared from consolidated slurry on sand-bentonite mix where hydraulic conductivity increased up to two orders of magnitude. Due to the specimen volume shrinkage, side wall leakage might occur in RWPs causing such increases in hydraulic conductivity. Due to high effective stress applied in FWPs, any changes in volume of the specimen will be compensated by cell volume increase. Therefore, volume change should be taken into consideration during compatibility testing. Volume change characteristics of soil can be assessed from the free swell test. Ryan (1987) found from the test results that free swell is dependent on the soil type and dielectric constant of the pore fluid and the results are shown in Figure 2.12. From this figure, it is clear that montmorillonite soils display higher free swell volume with an increase in dielectric constant.

Cadwallader (1985) showed that permeation of organic fluids can cause a large increase in hydraulic conductivity due to some structural alteration which is mainly irreversible. He confirmed this by reintroducing water into the soil permeated with organic fluids. He noticed that the hydraulic conductivity did not resort to the same values as before. Bowders (1989) found, from experimental results, that there was a substantial increase in hydraulic conductivity when clay is subjected to strong acids, lower effective stresses or longer interaction times. Organic acids can dissolve carbonates and iron oxides from soil and can lead to downstream precipitation and pore clogging, resulting in a decrease in hydraulic conductivity (Uppot and Stephenson, 1989). Weak organic bases appear to have no effect on the clay particles and the effect of alkalis is not reported in the literature.

Concentrated organic chemicals may cause a change in the ionic bonding conditions in the clay minerals, modifying the diffuse double layer around each clay particle and causing the particles to draw closer together. Shrinkage of this nature could increase the hydraulic conductivity by several orders of magnitude. Although, in general, organic compounds increase the hydraulic conductivity of clay soil, certain compounds and reaction types may decrease hydraulic conductivity. A detailed study on the effect of organic chemicals on hydraulic conductivity will be presented in Chapter 8.

# 2.6 LABORATORY TESTING OF HYDRAULIC CONDUCTIVITY

Laboratory testing on hydraulic conductivity can provide quantitative estimates of the suitability of the liner in minimum time and at low cost. The factors influencing laboratory hydraulic conductivity can be classified in two categories: (a) factors associated with testing variables such as, type of test, confining pressure, saturation and hydraulic gradient, and (b) factors associated with the type of permeant, physical



Figure 2.12: Dielectric Dispersion vs Free Swell (source: Shackelford, 1994)

and chemical characteristics of the soil such as physicochemical properties of permeant, soil density, compaction method, soil water content at compaction, specimen hydraulic defects and specimen size (Karfiatis *et al.*, 1987).

Several methods of laboratory testing for hydraulic conductivity are possible, such as:

- constant head test in which the head loss is kept constant and the corresponding rate of flow is measured.
- varying head test in which the head loss declines with time in a measured manner and the rate of flow is computed from the change in water level and the area of the tube in which the head falls.
- constant rate of flow test in which the rate of flow is kept constant and the corresponding head loss is measured.

Hydraulic conductivity tests are normally carried out in the laboratory either by flexible wall permeameters or by fixed wall permeameters. Flexible wall permeameters are triaxial cells or modified triaxial cells having double drainage lines to both top and bottom of the specimen. Fixed wall permeameters are of different types. These are compaction mould permeameters, consolidation cell permeameters and fixed cylinder permeameters. Double ring and triple ring permeameters are used to identify side wall leakage in compaction mould permeameters. Each of the permeameters has advantages and disadvantages and this should be taken into account while planning the testing program. A detailed description of different types of permeameter will be presented in Chapter 4.

In the flexible wall test, the cell pressure is kept constant throughout the test and the sample can increase or decrease in volume depending on the internal structural changes. In the fixed wall cell and the consolidometer, any increase in the diameter is prevented which implies that the horizontal stresses in the sample will vary throughout the test. The only difference between the fixed wall cell and the consolidometer is that a certain effective vertical stress can be applied to the top of the specimen in the consolidometer, and perhaps the difference in specimen thickness. The flexible wall test and the consolidometer are almost identical in terms of vertical stresses (Peirce *et al.*, 1986). The variation of effective vertical stresses and pore water pressures for the three types

of tests are given in Figure 2.13 (where  $P_1$  and  $P_2$  are water pressures at influent and effluent ends and P is the applied load).



Figure 2.13: Variation of Effective Stresses and Pore Pressures for Fixed Wall, Flexible Wall and Consolidation Cell Permeameters (source: Peirce, 1986)

# 2.6.1 Permeant Liquid

Water: The important characteristics of the permeant water are the amount of dissolved air in the water, the type and concentration of electrolytes, turbidity, nutrient content, and the population of micro-organisms. Since tap water generally contains a certain amount of salt, some researchers have used 0.01N CaSO<sub>4</sub> as a standard permeant water (Brown & Anderson, 1983). Nevertheless, ASTM D5084 specifies normal tap water because distilled water leaches electrolytes from the soil water, which expands double layers and reduces hydraulic conductivity. Turbid water is not used in hydraulic conductivity testing because turbidity may clog flow paths. Nutrients can promote the growth of micro-organisms which tend to reduce hydraulic conductivity specially in long term tests. Growth of microorganisms is discouraged by use of fresh, chlorinated tap water. Table 2.4 gives an overview of the characteristics of permeant water on the basis of electrical conductance and ionic strength.

Chemicals & Waste Liquids : Permeation with chemicals or waste liquids presents a number of additional challenges, which include health and safety considerations,

material compatibility concerns, cross contamination potential from one test to the next, changes in chemistry of the influent liquid, and the monitoring of the chemistry of the effluent liquids (Daniel, 1994).

Water Type	Reference	Maximum Electrical	Maximum Ionic
		Conductance ( $\mu$ s/cm)	Strength
			(moles/litre)
Distilled	Standard	1.0	0.000013
	Methods (1985)		
Deionised-	Standard	0.2	0.0000026
distilled	Methods (1985)		
Standard 0.01N	Olson and	1538	0.02
CaSO 4	Daniel (1981)		
Standard 0.005N	ASTM D 5084	769	0.01
CaSO 4			
Tap Water	Not available	Variable	Variable

**Table 2.4:**Types of Permeant Water (source: Shackelford, 1994)

Quigley and Fernandez (1989) used the technique of adding surfactants with permeant liquids to destroy surface tension effect. They found that when cyclohexane was used as a permeant, no flow was detected even at a high hydraulic gradient and hydraulic conductivity was zero. When a 5% soap and 95% cyclohexane were used as a permeant, flow became continuous with a high hydraulic conductivity value of 1.2  $\times 10^{-9}$  cm/s. Since many types of surfactants are usually available in domestic and industrial leachates it seems reasonable to use a surfactent mixture in compatibility testing with chemical featuring high surface tension.

# 2.6.2 Test Duration

In geo-environmental engineering, duration of testing is generally expressed as the number of pore volumes (pv) passed through the specimen. Porosity is the term used to express the ratio of the volume of voids to the volume of solids, and is commonly

used by the geotechnical engineers. Pore volume is the amount of pore spaces in a soil specimen *ie.* porosity times the volume of the specimen, and depend mainly on the soil type and the compaction levels of the soil. Therefore, pore volume varies from specimen to specimen and consequently requires different time for one pore volume of permeant to flow through different specimens.

A number of factors can cause changes in hydraulic conductivity with time. It is essential in hydraulic conductivity testing that flow through the specimen be continued until stable hydraulic conductivity measurements are obtained. For testing with organic chemicals as permeant, the test should be continued until the concentrations of influent and effluent are same. In compatibility testing, at least two pore volumes of fluid should be passed through the specimen to ensure that any tendency towards structural changes effecting an increased or decreased hydraulic conductivity is observed (Bowders et al., 1986). Due to long term chemical contamination of a soil, there is a change in soil structure causing a change in mechanical properties of the soil. According to Meegoda and Rajapaske (1993), the long term condition can be simulated by premixing the soil with the permeating fluid and the hydraulic conductivity of the resulting soil chemical mixture is measured. Nevertheless, this approach can be criticised because if soil is mixed with permeating fluid prior to compaction, any structural changes in the soil would be masked due to remoulding and compaction. One suggestion could be to prepare the soil specimen and allow the specimen to reach equilibrium by diffusion for a long duration in the same environment as the testing, before the real testing is being conducted.

To reduce the testing time, another alternative may be to apply the permeant first that is the first exposure effect without applying the traditional method of water and then the permeant. According to a number of investigators (*eg.* Lo *et al.*, 1994; Shackelford, 1994), clay liners in the field may not be exposed to a few pore volumes of water before the waste liquids. It is a usual practice to compact liner materials in the field a few percent higher than optimum moisture content. Liners compacted in such conditions may be close to saturation (*ie.* 90% saturation). If direct permeation is made with some chemicals, this should also expel water from the voids before actual structural changes might occur. In such a situation, questions might arise regarding the effectiveness of the first exposure effect. On the other hand, it will be difficult to compare the variation of hydraulic conductivity with water and permeants because hydraulic conductivity can vary from specimen to specimen even under almost identical conditions.

#### 2.6.3 Sample Size and Sample Disturbance

The hydraulic defects such as cracks, fissures, and sand lenses can increase the hydraulic conductivity of fine grained soils. These defects have statistically better chance of being present in a large sample compared to a small one (Olson & Daniel, 1981). Therefore, the hydraulic conductivity of a large diameter sample would tend to increase with increasing sample diameter but for convenience the maximum diameter of laboratory samples is limited to 300 mm. As shown in Figure 2.14(a), Benson *et al.* (1994) found an increase in hydraulic conductivity with increasing diameter and the increase ceased at a diameter equal to or greater than 300 mm. According to the American Society for Testing and Materials neither the length nor the diameter of the test specimen should be less than approximately 6 times larger than the largest particle in the test specimen (ASTM D2434, ASTM D5084). This was kept in mind while considering the specimen size in the testing program.

Most rigid wall, compaction mould permeameters are used in one of 3 diameters: (1) 36 mm as in a Harvard miniature compaction mould; (2) 100 mm as in a typical proctor compaction mould; and (3) 150 mm as an alternative proctor mould diameter for materials with large particle sizes. Consolidation cell permeameters have diameters of 5 to 8 cm and a thickness of 1 to 2 cm. Flexible wall permeameters are available commercially for testing soil specimens with diameters as large as 300 mm.

As shown in Figure 2.14(b), Korfiatis *et al.* (1987) found that specimen size (height) varies linearly with the logarithm of hydraulic conductivity. Boynton and Daniel (1985) found a log linear behaviour for specimens having different diameters but the same height. For small diameter specimens the ratio of disturbance to size is much greater and can have more influence on the measured value rather than on the large diameter specimens. Decreasing L/D ratio tends to accelerate testing times. Thus, thin specimens are advantageous when testing with chemicals and waste liquids.



Figure 2.14(a): Hydraulic Conductivity vs Specimen Size Relationship (source: Benson et al. 1994)



Figure 2.14(b): Variation of Hydraulic Conductivity with Specimen Height (source: Korfiatis, 1987)

Disturbance of soil during sampling and trimming causes smearing of the soil which closes preferred seepage paths, therefore, changing hydraulic conductivity. For undisturbed samples, voids may be formed around the edges due to inadequate control of trimming and fissures may open as a result of stress relief, thus leading to high measured hydraulic conductivity values. Carpenter & Stephenson (1986) stated that the extent to which flow is impeded is a function of the trimming tool used, the properties of the soil, that is, how susceptible the soil is to smearing, and the care and skill of the operator.

### 2.6.4 Effective Overburden Stress

Soil beneath the landfill will be subjected to some overburden pressure. In the laboratory conditions, this is reflected by the confining pressure applied on the specimen. During compaction a certain level of stress is developed within the specimen and this could be measured using standard consolidation tests. The term preconsolidation pressure is used to express the stress level in a compacted specimen, which is the maximum past pressure that the specimen has been subjected to and is important as far the hydraulic conductivity testing is concerned. If the hydraulic conductivity tests are carried out at effective stress less than preconsolidation pressure, swelling of the specimen may result with a consequent increase in hydraulic conductivity. On the other hand if the test is carried out at effective stress higher than preconsolidation pressure, sample consolidation might result with the decrease in hydraulic conductivity.

Hydraulic conductivity is greatly influenced by the amount of overburden pressure applied on the clay liner. This influence is particularly significant when the permeation is made with chemicals and liquid hydrocarbons because confining pressure tend to heal any structural changes. The critical situation is the case of stabilisation ponds where the effective overburden stress is zero and the liner underneath may have a chance to swell, causing significant increases in hydraulic conductivity. Therefore, it is advisable not to use effective stress in the laboratory tests that are larger than the field values.

#### 2.6.5 Hydraulic Gradient

It is general practice to use a large hydraulic gradient to reduce testing time. But a decrease in hydraulic conductivity together with an increase in hydraulic gradient was observed by different investigators (eg. Mitchell and Younger, 1967; Olson & Daniel, 1981). Reasons for such a decrease in hydraulic conductivity are as follows: (a) the migration of particles causing pore clogging; (b) soil consolidation, and (c) changes in soil fabric and structure partly through rearrangement of soil particles and increased channelisation in the micro fissures and cracks in the soil sample. Daniel (1994) stated that an excessively large hydraulic gradient tends to wash fine soil particles downstream in the test specimen and may become trapped in the soil near the downstream, reducing hydraulic conductivity. Also the fine particles may be washed completely out of granular soils, which would cause the hydraulic conductivity to increase. ASTM D5084 recommended maximum hydraulic gradient of 30 for soils having a hydraulic conductivity of  $10^{-7}$  cm/s. Hydraulic gradients for laboratory testing in the range of 100 to 200 are also used but the use of gradients as close to those of the field value are commonly recommended. A detailed study of the gradient effect on hydraulic conductivity and specimens consolidation will be discussed in Chapter 6.

# 2.7 IN-SITU TESTING

In-situ hydraulic conductivity testing such as lysimeters, open infiltrometer, borehole tests etc. are time consuming and expensive compared to laboratory testing. These topics are beyond the scope of the current thesis and interested readers may need to refer to Daniel (1987), Gordon *et al.* (1989), Uppot & Rauser (1989) etc. Olson & Daniel (1981) found that nearly 90% of the observation of the ratio of ( $k_{jield}/k_{lab}$ ) lie in the range from 0.38 to 64. Daniel (1984) found a large difference in hydraulic conductivity between the laboratory and field results, namely, 10 to 10000 times. He explained that this could be due to the differences in the mini and macro fabric

resulting from poor construction practices. If the liners are properly constructed (*eg.* moisture content at wet of optimum) this difference can be minimised (Daniel & Benson, 1990).

# 2.8 CONCLUDING REMARKS

Environmental guidelines commonly specify that the hydraulic conductivity of a compacted clay liner should be no greater than  $1 \times 10^{-9}$  m/s during the life-time of the facility. Normally, this empirically-based criteria is implemented into design and construction of a facility by conducting laboratory hydraulic conductivity tests of compacted soil samples using water as the permeating fluid. Nevertheless, recent research indicates that hydraulic conductivity can increase significantly if testing is conducted with some chemicals representative of leachates generated in waste containment systems. This can lead to significant environmental damage due to accelerated release of toxic chemicals into the groundwater. Typical chemicals, which have led to high increases in hydraulic conductivity include some hydrocarbons, acids and bases. These chemicals have also been found to react mostly with the clay types containing montmorillonite minerals.

In Victoria, hydraulic conductivity of soils with pure water is currently used for the compatibility assessments associated with these facilities. Test methods for the determination of hydraulic conductivity in geo-environment situation have not been approved by the Australian Standard. However, overseas research indicates that the hydraulic conductivity of soils containing minerals such as montmorillonite can increase by as much as 10<sup>6</sup> times and illite by 10<sup>3</sup> times when permeated with liquid hydrocarbons (Fernandez and Quigley, 1985). Increases in hydraulic conductivity are attributed to the changes in the soil structure due to the collapse of the electrical double layer, resulting in the formation of flocculated clay structure. Therefore, it is important that the soil hydraulic conductivity be assessed rationally to ensure the effective performance of clay liners.

# 3. CHARACTERISATION OF SOILS AND CHEMICALS

# 3.1 INTRODUCTION

The chapter presents the details of characterisation of soils and chemicals used in the testing program. These details include testing techniques used and the results obtained. Soil samples were collected from four different locations of western and north-western Victoria. These soils are used for construction of clay liners for domestic waste landfills and for an evaporation pond. These collected soil samples were analysed for physical, physico-chemical and mineralogical properties. For compatibility testing for hydraulic conductivity, saline water, methanol, leachate and modified leachate were used. Leachate samples were collected from PWM Lyndhurst site and were analysed for physical and chemical properties. Different properties of saline water and methanol at different concentrations were also determined.

# **3.2** COLLECTION OF SOIL SAMPLES

Soil Samples were collected from Werribee, Echuca, Melton and Sunshine sites. These soils are available on-site and are economically used for liner construction. Werribee landfill site is located at West's Road which is approximately 35 kilometres west of the City of Melbourne and 8 kilometres south-west of Werribee. Echuca site, which is known as Terrick Road Waste Water Lagoons, is located about 205 kilometres northwest of Melbourne. Melton Municipal Landfill Site, also known as Ferris Road Tip, is located in Ferris Road (Melway Reference 117 D/E2). Sunshine site, which is known as Brooklyn Municipal Tip, is located along Jones Road (Melway Reference of 40 H8). Werribee and Echuca soils were collected near surface by using shovels. Melton soil was collected from the stock piles of soil dug from the site. Soils collected from

Sunshine site are from stock piles of borrowed material from the corner of Powlett Street and Victoria Parade (Melway 44 C7). Location maps of these sites are shown in Appendices A3.1 to A3.4. Soil samples were collected into large polythene bags, transported to the VUT laboratory, processed as discussed in Chapter 4, and stored in containers for future use.

From the literature review in Chapter 2, it was shown that the hydraulic conductivity of the liner material is dependent on the physical as well as physico-chemical properties of the soil. Testing of these parameters was carried out in the laboratory and the details are discussed in the following sections.

#### 3.2.1 Physical and Physico-Chemical Analyses of Soils

The assessment of soil physical and physico-chemical properties was undertaken in the laboratory. The physical properties assessed included moisture content, amount of clay fraction and particle size distribution, specific gravity, compaction properties and Atterberg limits. The physico-chemical properties included pH, electrical conductivity (EC), cation exchange capacity (CEC), major cations and slaking and dispersion behaviour. The details and results of these tests are discussed here.

#### Moisture content

The as-compacted moisture content of compacted clay plays a significant role in controlling the hydraulic conductivity of the compacted clay. As described in Chapter 2, the hydraulic conductivity of compacted clay can vary as much as six orders of magnitude going from dry to the wet side of optimum. Therefore, the accurate determination of moisture content is paramount for hydraulic conductivity measurement.

The moisture content of a soil is given by the percentage of water present expressed as a percentage of dry weight and was carried out by weighing specimens before and after

the oven drying for 24 hours at  $105^{\circ}$ C. Moisture contents for individual specimens were determined before carrying out any hydraulic conductivity testing. All moisture content determinations were made in accordance with the Australian Standard AS 1289.B1.1.

## • Particle size distribution

The hydraulic conductivity of soils is significantly dependant on the particle size distribution. For example, Wang and Huang (1984) have shown that there exists a strong correlation between the hydraulic conductivity of compacted clays and their particle size distribution. For this reason, construction specifications commonly contain restrictions on the particle size distribution of liner soils. For example, Benson *et al.* (1994) have recommended the minimum requirements of particle size distribution for liner soils to achieve hydraulic conductivity less than 1 x 10<sup>-9</sup> m/s as: percentage fines  $\geq$  30%; percentage of clay  $\geq$  15%. Maximum size and percentage of gravel is also very important for effective liner material and Daniel (1993b) suggested that the gravel portion to be less than 30% and be within 25 to 50 mm size range. It should be noted that these recommendations are not absolute because they depend on other factors such as the mineralogical composition of the soils. Therefore, there is significant scope for developing more effective site specific criteria based on soil tests.

The tests for particle size distribution were carried out for the soils collected in the laboratory by sieving in combination with hydrometer analysis as described in Australian Standard AS 1289.C6.2. The sieve analysis was used for particles greater than 75  $\mu$  m, and the portion of soils passing through 75  $\mu$  m was analysed by hydrometer. The results of these analyses are presented in Appendices A3.5 to A3.8 and the results are also tabulated in Table 3.1. From the requirements for liner material as described before, all the soil samples collected are found to be suitable as liner material, although the amounts of fine and coarse fraction in collected soils vary significantly.

	Clay	Silt	Sand	Gravel
	(%)	(%)	(%)	(%)
Sunshine	24	46	29	1
Melton	62	28	9	1
Werribee	45	43	10	2
Echuca	56	35	8	1

Table 3.1: Particle Size Distribution

# • Specific Gravity

The specific gravity of the soil particles is given by the ratio of the mass of a given volume of solid soil particle to the mass of an equal volume of distilled water at any stated temperature. All specific gravity tests were performed with a specific gravity bottle for soils passing through 2.36 mm sieve and were carried out in accordance with Australian Standard AS 1289.C5.1. The results are presented in Table 3.3.

# • Compaction

The liner soils should have the potential to be compacted to achieve required liner properties, mainly the minimum hydraulic conductivity. Compaction potential is normally assessed by laboratory Proctor compaction tests. The compaction effort used in these tests should reflect those to be used in the field. Current standards allow for two levels of compaction efforts to be used, namely the standard and the modified. Compaction energy levels and related procedures are available in various standards. More recently, a reduced level of compaction has also been proposed to represent smaller equipment such as hand operated equipment (Bardon and Sides, 1970). This equipment may be particularly important for liner construction on slopes where heavy equipment is not generally convenient to use. The information which would be invaluable is the traditional compaction curves of dry density versus moisture content and the corresponding relationships of hydraulic conductivity versus moisture content obtained for different levels of compaction efforts for the same soil. With this

information in hand, it is relatively straightforward to examine the level of compaction effort and the corresponding moisture ranges that will satisfy the minimum hydraulic conductivity criteria (Daniel & Benson, 1990).

Laboratory compaction tests were carried out with soil samples collected by using the standard Proctor compaction method. From the results presented in Section 4.7, it could be found that the moisture content which gives maximum wet density is a closer approximation, and better correlated, to the moisture content giving minimum hydraulic conductivity than the traditional optimum moisture content. Summary results of the compaction tests are tabulated in Table 3.2. Graphical representations of these results are presented in Appendices A3.9 to A3.12.

 Table 3. 2:
 Summary of the Compaction Test Results

	Maximum	Optimum	Maximum	Moisture Content
	Dry Density	Moisture	Bulk Density	(%) at Maximum
	(KN/m <sup>3</sup> )	Content (%)	(KN/m <sup>3</sup> )	Bulk Density
Sunshine	17.8	15.9	20.7	17.2
Melton	14.0	31.0	18.6	32.5
Werribee	14.0	31.5	18.4	33.0
Echuca	15.5	24.2	19.4	25.9

# Hydraulic Conductivity Testing

Hydraulic conductivity testing carried out in the laboratory is discussed in detail in Chapter 4. This includes a detailed description of the equipment used for hydraulic conductivity testing and the methodologies adopted in the laboratory. Several chemicals were used for undertaking such hydraulic conductivity testing with a wide range of concentrations. On the basis of the test results and published data, a relationship was developed between the moisture content and the hydraulic conductivity for the compacted clay and this is presented in Chapter 4.

#### • Atterberg limits:

It is well accepted that Atterberg limits display the plasticity of the clayey soils, and are commonly used to make recommendations on the suitability of liner soils. Benson and Trast (1995) found experimentally that the liquid limit and plasticity index are directly related to the mineralogy of the soil and the clay content. They also found that the hydraulic conductivity generally decreases with increasing liquid limit and plasticity index. For example, Benson *et al.* (1994) has recommended that for liner soils the liquid limit  $\geq 20$  and plasticity index  $\geq 7$ .

Considering the fact that there is apparent correlation between Atterberg limits and the hydraulic conductivity of clay, tests for liquid limits were carried out in the laboratory using two standard techniques, namely: the Casagrande method as devised in AS 1289.C1.1 and the cone penetrometer method as in the British Standard BS 1377:1975. Plastic limit tests were carried out by the thread rolling method as in Australian Standard AS 1289.C2.1. Soil passing through  $425 \,\mu$  m sieve was used for the preparation of specimens for Atterberg limit tests. Soil samples were thoroughly mixed with the fluids and were allowed to cure for at least 12 hours. The results are presented in Table 3.3. Results obtained from these tests were found to be reasonable if consideration is made on the basis of mineralogy and the clay content, and the data is presented in Table 2.1.

Atterberg limits obtained from the test results were used for the classification of the above soils using Unified Soil Classification systems, and the results are presented in Table 3.3. Visual observations of the soil samples were made and are reported in Table 3.3.

#### • pH

As is evident from the literature review from Chapter 2, the soil pH has a significant influence on the double layer thickness and hence the hydraulic conductivity.

Laboratory determination of soil pH was carried out in the present study. The standard method as proposed by *Australian laboratory handbook of soil and water chemical methods (1992)* was used for such analysis. The pH value of a soil indicates the intensity of acidity which, in turn, influences soil condition. The soil pH values were determined in the laboratory by a soil/water ratio of 1:5 at  $25^{\circ}$ C. Soil samples weighing 20 gm of air-dry soil (<2 mm) and 100 gms of deionised water were used in those tests. The bottle was shaken mechanically at  $25^{\circ}$ C in a closed system for 1 hour. The suspension was then allowed to settle for 20 to 30 minutes. A buffer solution of pH 7.0 was prepared and both the buffer and the soil suspension were mechanically stirred while the electrodes were immersed in the container. The soil pH value was recorded when the meter appeared steady while the suspension was being stirred. The results are reported in Table 3.3.

#### • Electrical Conductivity (EC)

The electrical conductivity of soil suspension is used to estimate the concentration of soluble salts in the soil. It was noted in Chapter 2 that the presence of salts in the soil and permeant systems may significantly influence the hydraulic conductivity of compacted clay liners. EC tests were carried out in the laboratory on the soil samples collected. EC values were determined in the laboratory on a 1:5 soil/water extract. A suspension of 1:5 soil/water ratio was prepared by mixing 20.0 gms of air dry soil and 100 ml of deionised water. As in the case of pH measurement, the suspension as well as the reference solution was stirred while the measurements were being taken. The results are reported in Table 3.3.

## • Cation Exchange Capacity

Tests for cation exchange capacity of the collected samples were carried out in the laboratory by silver thiourea method. The standard method as proposed by *Australian laboratory handbook of soil and water chemical methods (1992)* was used for such analyses. The results are presented in Table 3.3.

	Specific	Atterberg	Classification,	pН	Electrical	Cation
	gravity	limit LL (%)	colour		conductivity	exchange
		PL (%)			(EC)	capacity
					(ms/cm)	(meq/100g)
Sunshine	2.69	36	Clayey SAND (SC)	8.2	0.85	13.4
		15	Red Brown			
Melton	2.71	83	Sandy CLAY(CH)	8.5	1.27	35.2
		34	Grey			
Werribee	2.69	74	Sandy CLAY (CH)	8.6	1.98	32
		32	Grey			
Echuca	2.63	63	Sandy CLAY (CH)	7.4	1.53	16
		25	Grey Brown			

 Table 3.3: Physical and Physico-Chemical Properties of Soil

The specific gravity values obtained for the soils are within the range of normal soils. pH values obtained from the test results show the soils are slightly alkaline for Echuca soil, and moderately alkaline for other soils. From the test results of electrical conductivity values, the soils are found to be saline. The amount of clay minerals present in different soil samples, as presented in Table 3.6, could be used to calculate the Atterberg limits and the cation exchange capacity values. Standard values presented in the summary table of the clay mineral characteristics in Chapter 2 could be used for such calculations. It could be seen that these values are within the resonable limits for the mineralogy obtained for different soils.

## Tests for Major Cations and Slaking and Dispersion

Tests for four major cations as  $Ca^{++}$ ,  $Mg^{++}$ ,  $Na^{+}$  and  $K^{+}$  for Werribee and Echuca soils were carried out at the State Chemistry Laboratory in Werribee. Measurements of these cations are necessary to assess adsorption, slaking and dispersive properties of soils as well as determining the nature of soil salinity. As stated in Chapter 2, the salinity of soil can cause cracking as well as mineralogical and structural changes in soil. These factors could eventually lead to the hydraulic conductivity to be increased. Therefore, accurate determination and analyses of these values are essential as far as the hydraulic conductivity of the soil is concerned. Exchangeable sodium percentage (ESP) was calculated on the basis of the major cations using Equation 3.1. As discussed in the Chapter 2, the SAR is the property of the saturation extract and calculation is not straightforward from the values of these major cations. An indirect method, as devised by the US Soil Salinity Laboratory (1954), was used to calculate the value of SAR from ESP values.

Slaking and dispersion are two processes that can degrade soil structure upon wetting. Slaking occurs within minutes and causes the breakdown of aggregates into smaller sizes. Dispersion, on the other hand, takes hours and causes the breakdown of aggregates into individual clay, silt and sand particles. Slaking and dispersion tests were carried out at the State Chemistry Laboratory in Werribee for Echuca and Werribee soil. Both soils tested showed considerable slaking. Dispersion tests were carried out for both dry aggregate and remoulded aggregate for 2 hours and 20 hours respectively. No significant dispersion was observed from these tests. As stated in Chapter 2, the spontaneous dispersion in soils generally becomes evident when the ESP is more than 6 percent. The reason for not having the dispersion in these soils could be the high salinity of the soil, which can suppress dispersion. In such a situation, one of the suggestions could be to leach out the soil salinity using distilled water and carry out the dispersion testing afterwards.

$$ESP = \frac{Na^{+}}{\text{Total exchange capacity}} X \ 100 \tag{3.1}$$

 Table 3.4:
 Major Cations of Soil

	Ca <sup>++</sup>	Mg <sup>++</sup>	Na <sup>+</sup>	к <sup>+</sup>	SAR	ESP
Werribee	20	15	20	1.2	28	36
Echuca	21	12	9.6	0.89	17	22

# 3.3 CLAY MINERALOGICAL ANALYSIS

As discussed in the previous chapter, liner soil should contain a significant amount of clay fraction. This clay fraction basically controls the performance of the soil liner because clay minerals display high specific surface areas and potential for reactivity with various chemicals. As revealed from the literature review of Chapter 2, recent research indicates that clay mineralogy plays a significant role in the long term integrity and retention capacity of these liners. A thorough investigation of the mineralogical analyses of the collected soil samples was carried out as part of the present research program. Clay mineralogy of coarse (2 -  $0.2 \mu$  m) and fine (<  $0.2 \mu$  m) clay fractions from all soils were established by X-ray diffractometry (XRD). A semi-quantitative analysis of the XRD results revealed the presence of both swelling minerals including smectite and vermiculite and non-swelling minerals including kaolinite, illite and chlorite. The implications of these mineral compositions for soil liner performance are discussed in the following sections.

#### 3.3.1 Materials and Methods

Mineralogical analyses of the collected soil samples from four different locations, as discussed in Section 3.2, were carried out using XRD. The technique is based on a special case of reflection-diffraction of x-rays from regularly repeated planes of ions within the surface of a mineral lattice as shown in Figure 3.1. As given in Equation 3.2, Bragg's Law describes mathematically the phenomena of diffraction and reinforcement of x-rays by atoms in crystal structure.

$$n\lambda = 2d_s \sin\theta \tag{3.2}$$

where  $\lambda$  = wave length of incident radiation (0.15478 nm for Cu-a radiation)

 $d_s = \text{interplanar spacing (Å or nm)}$ 

 $\theta$  = angle of beam incidence/reflection.

n = the order of diffraction



Figure 3.1: X-Ray Diffraction Plot for the Fine Fraction of Melton Soil After Potassium Treatment

There is a critical angle,  $\theta$ , for a given interplanar (inter lattice) spacing where n = 1. At other angles, where n = 2, 3 or greater, a higher order of diffraction occurs, however, the interplanar spacing,  $d_s$ , remains constant. The pattern of such  $d_s$  spacing is characteristic of a mineral and can be used to identify minerals by referring to standards or published tables.

In the x-ray diffractometer, specially prepared samples are rotated at an angle of  $\theta$  in the path of a x-ray beam. The reflections are measured on a detector rotating  $2\theta$  degree for every degree  $\theta$  that the sample rotates. Therefore, all the graphs plotted from the XRD tests are shown in intensity corresponding to  $2\theta$  values.

Soil passing through a 2.36 mm sieve are used for mineralogical analyses. In the past, and often even at the present time, mineralogical investigation of the bulk clay (< 2  $\mu$  m) is the common practice because this leads to substantial time saving. However, dispersed clay pipetted on to a slide leads to stratification, with results obtained only from the finer particles near the surface of the mount. It was therefore decided to investigate two sub-divisions of clays, namely coarse (2 - 0.2  $\mu$  m) and fine (< 0. 2  $\mu$  m), to obtain reliable results on clay mineralogy of the selected soils.

For clay mineralogical investigation, free iron oxides were removed from both coarse and fine fractions of all the samples by sodium dithionite. After the removal of free iron oxides the soil suspension was centrifuged and the resulting supernatant was discarded. To each treated soil sample, 50 ml 5 % calgon solution was added so that the soil was well dispersed. The soils were left overnight and the following day, the volume was increased to 1 litre by adding distilled water. The soil suspensions were stirred with a plunger and were left for 8 hours for material <2  $\mu$  m equivalent spherical diameter (e.s.d.) to sediment over 10 cm. A *J* siphon tube was then placed in the cylinder such that the tip was 10 cm below the surface of the liquid and the suspension was siphoned off, representing the clay fraction as described by Kilmer & Alexander (1949). Clay sub-divisions were accomplished by sedimentation under centrifugal force by using the integrated form of Stoke's equation as proposed by Hathaway (1956) and as is discussed in Equation 3.3. The process was repeated 5 times in order to obtain an

amount sufficient for further sub-divisions of the clay.

A Beckman JA-14 refrigerated centrifuge was used (at the centre for Bio-processing and Food Technology, Werribee Campus, Victoria University of Technology) for fractionation. The polyethylene bottles used had a capacity 250 ml, but the distance from the surface of the suspension to the rotation axis was of greater importance in the calculation. The centrifuge bottle and the fixed angle rotor is shown in the Figure 3.2. The position of the bottle to the rotor was fixed at an angle of  $25^{\circ}$ . The temperature of the suspension was maintained at  $10^{\circ}$ C. Distilled water was used as the suspending medium and the bottles were filled to a depth of 6 cm. The above information was used in Equation 3.3 to calculate the time required for the sedimentation of particles of particular diameter and specific gravity under a centrifugal force. These results are presented in Table 3.5.

$$t = \frac{\eta \log_{10} \frac{R_2}{R_1}}{3.81r^2 N^2 (\rho_s - \rho'_s)} + 2/3 (t_a + t_d)$$
(3.3)

where,

t = time taken for the larger particles in the size fraction to travel the distance from the surface of the suspension to the bottom of the centrifuge bottle, (s)

 $t_a$  = time of acceleration of the centrifuge to reach at the required speed, (s)

 $t_d$  = time of deceleration of a centrifuge to stop, (s)

 $\eta$  = viscosity of suspension, (poises)

 $R_I$  = initial distance of particles from the axis of rotation, (cm)

 $R_2$  = final distance of particles from the axis of rotation, (cm)

r = equivalent spherical radius of particle to sediment, (cm)

 $\rho_s$  = density of the particles (2.65 g cm<sup>-3</sup>)

 $\rho'_{s}$  = density of the medium (1.0 g cm<sup>-3</sup>)

N = rotary speed of the centrifuge machine, (rps)



Figure 3.2: Centrifuge Bottle in a Fixed Angle Rotor of Beckman JA-14 (source: Shahid and Jenkins, 1991)

**Table 3.5:** Centrifugation Timing for the Separation of Various Subdivisions of Clay forCalibration Curves (source: Shahid and Jenkins, 1991)

Diameter of	Temp	Sampling	R <sub>1</sub>	R <sub>2</sub>	Rotor Speed	ta	t,	Tim	e of
particles to	$(^{0}C)$	Depth (cm)	(cm)	(cm)	(rpm)	(s)	(s)	sett	ling
sediment (µm	<b>x</b> - <b>y</b>				$(10^{3})$			min	sec
>0.063	10	6	8.3	12.9	10	125	213	27	50
>0.200	10	6	8.3	13.0	4	42	114	16	56
>0.630	10	6	8.3	13.1	1.5	17	67	12	05

#### 3.3.2 Preparation of Oriented Clay Slides

About 100 ml of clay suspension from each fraction (ie., coarse and fine) was placed into two polyethylene bottles, one containing magnesium (Mg) and the other with potassium (K), using quantities of solid MgAc and KCl to make the suspension of 1N. The suspensions were then centrifuged and washed twice with distilled water and then with industrial methanol and were centrifuged again. About 40 mg of clay sediment was then redispersed in 2 ml of distilled water. Subsequently, dispersed clay was pipetted on to a slide and left to dry at room temperature. The Mg saturated clay slides were treated with ethelyne glycol to distinguish between swelling and non-swelling minerals. During this treatment some vermiculites also expand but with the glycerol only smectites undergo expansion. Therefore, a drop of glyecrol was used on the Mg treated slides. Heating the slides to 550°C for 1 hour brings smectites, and vermiculites to 10Å, while chlorite remains at 14Å. Therefore, K saturated slides were heated for 1 hour at 550°C. Heat treatment was found unsatisfactory to differentiate between chlorite and kaolinite, and This therefore, a procedure as developed by Lim et al. (1981) was used instead. procedure uses the treatment of potassium saturated soil with cesium chloride (CsCl), hydrazine hydrate and dimethyl sulfoxide (DMSO) to resolve kaolinite from chlorite. This clearly differentiates the first order peak of kaolinite (7.12Å) which normally overlaps that of the second order peak of chlorite by shifting to 11.2Å. The testing schedule is shown in Figure 3.3. For each clay fraction similar treatments were carried

out giving the total number of tests to twelve for each soil. X-ray diffraction scanning analyses were carried out at CSIRO, Division of Mineral Products, Port Melbourne, Victoria using a Philips powder diffractometer PW1710 using CuK $\alpha$  radiation. The results of the investigation are presented in the following section.



Figure 3.3: Testing Schedule for Clay Mineralogical Analysis

#### **3.3.3 Results of Mineralogical Analyses**

The results of the clay mineralogical analyses are presented in Appendices A3.13 to A3.60. On the basis of the diffraction peak area calculation as shown in Figure 3.4, the results are calculated from these Appendices and are tabulated in Table 3.6. The XRD results revealed the heterogeneous nature of the clay fraction containing both swelling and non swelling minerals. These also contain ubiquitous mineral (Quartz) which is neither sensitive to the degree of soil development nor to the specific factors of soil formation. Quartz was detected as the abundant in the tektosilicate. The phyllosilicate suite is comprised of illite, kaolinite, smectite, vermiculite and chlorite. Illite, which is normally considered as the most desirable clay mineral for soil liners (Owesis & Khera, 1990), is present in both the coarse and the fine fractions of all soils in major amounts. Within illite dioctahedral muscovite dominates over trioctahedral biotite, because 001 reflection is stronger than 002 reflection. Different clay minerals are present in variable





amounts in various subdivisions of clay. Generally kaolinite is concentrated in coarse fraction and smectite in the finer except in Sunshine soils. Vermiculite and chlorite are either present in minor quantities or in traces in both fine and coarse fractions of all soils except in Echuca soils where vermiculite is present in major amounts in finer fractions.

Soil	Size range	Illite	Kaolinite	Smectite	Vermiculite	Chlorite
Туре	$\mu$ m					
Sunshine	2 - 0.2	4	3	2	1	1
	< 0.2	3	1	1	2	1
Melton	2 - 0.2	3	4	-	1	2
	< 0.2	3	-	2	1	2
Werribee	2 - 0.2	3	3	2	1	2
	< 0.2	2	1	4	2	1
Echuca	2 - 0.2	3	4	-	2	2
	< 0.2	3	1	2	3	2

 Table 3.6: Results of Clay Mineralogical Analyses

Key:

4 = Dominant (50 - 100 %)

3 = Major (30 - 50 %)

2 = Minor (10 - 30 %)

I = Trace (< 10 %)

Sunshine soils appear to have the most desirable mineralogical composition because they predominantly contain relatively inert illite and kaolinite. Nevertheless, these soils display low plasticity, which is also, an indication of the presence of low swelling minerals, and therefore, may not have sufficient self-healing properties, should the liners experience some cracking. Melton and Werribee soils displayed sufficient plasticity, but contained higher amounts of smectite and vermiculite. As noted previously, smectite minerals can lead to large increases in hydraulic conductivity through flocculation if the leachate contains sufficiently high levels of adverse chemicals which can collapse the diffuse double layer of the minerals. These adverse

chemicals include hydrocarbons and organic acids with low dielectric constants and to a lesser extent the salts containing high electrolyte concentrations. Echuca soils contained the highest amounts of vermiculite minerals which can also pose problems of reactivity specially through  $K^+$  fixation (if present in leachates) which can lead to c-axis contraction and shrinkage.

# 3.4 COLLECTION OF LEACHATE SAMPLES

Leachate sample was collected from the PWM Lyndhust site. The samples were collected from the well by rope and bucket. The rope tied bucket was dropped into the well and the bucket full of leachate was taken out into the ground. The top of the leachate sample was disposed from the bucket to remove the floating objects collecting only the bottom portion. The leachate samples were transported to the laboratory in completely full (leaving no head space), tightly sealed black-coloured high density polyethylene bottles. Where black coloured bottles were not used the bottles were wrapped with duct tape to prevent exposure to light. The leachate samples were centrifuged (at The Centre for Bio-processing and Food Technology Laboratory, VUT, Werribee Campus) at 10,000 rpm for 30 minutes to remove most suspended solids. The centrifuged leachate was then stored in a fridge for hydraulic conductivity testing purposes.

Chemical characterisation as well as concentration of various inorganic and organic constituents of the leachate samples was carried out at the WSL Envirocon Laboratory. Some of the physical properties were tested in the Chemical Sciences Laboratory of VUT. In all the cases standard laboratory methods were followed. All the test results are presented in Table 3.7.

	pН	EC	BOD	Sulphate	Chloride	Total alkalinity as CaCO3	Total Dissolved Solids $(180^{\circ}C)$
Leachate	7.5	15.6	2600	330	4200	4800	15000

**Table 3.7:** Physical and Chemical Properties of Leachate (EC is given in milli Siemens per cm (ms), and all other concentrations presented in Table 3.7 are in mg/l.)

<u> </u>	Ammonia	Kjeldahl	Volatile	Calcium	Potassi	Magnes	Sodium
	Nitrogen	Nitrogen	Acids		um	ium	
Leachate	570	580	3900	510	650	250	1800

Petroleum Hydrocarbons	C6 - C9	>C9 - C14	>C14 - C28	>C28 - C36
Leachate	0.81	7.0	110	15

#### • Viscosity of the Permeants

As discussed in Chapter 2, viscosity of the permeant plays a significant role in hydraulic conductivity. It could be seen from Equation 2.4, that the intrinsic permeability is a multiple of hydraulic conductivity and the density to viscosity ratio of the permeant. Permeants having higher viscosity could have difficulty in penetrating the small pores of the compacted clay and should usually have low hydraulic conductivity.

When a liquid flows through a tube, the layer of liquid in contact with the wall of the tube is stationary and the liquid in the centre has the highest velocity; intermediate layers move with a gradation of velocities. The flowing tubes may therefore, be regarded as a number of concentric tubes sliding past one another. Each layer exerts a drag on the next, and work must be done to maintain the flow. Newton deduced that the internal friction, or viscosity, would produce retarding forces, proportional to the velocity gradient, dv/dx (normal to the direction of flow), times the area of contact (A) between the moving sheets of liquid. That is:

$$F \propto A \frac{dV}{dx}$$
(3.4)

$$F = \eta A \frac{dv}{dx}$$
(3.5)

where  $\eta$  is a constant - the coefficient of viscosity. The commonly used unit of viscosity is the poise and it has the dimensions of g cm<sup>-1</sup> sec<sup>-1</sup>. The viscosity of the liquid solution is measured either by moving a solid body through the liquid or by allowing the liquid to flow through the capillary.

In the laboratory, tests for viscosity of the permeants were carried out by allowing the liquid to flow through a capillary tube. Accurate measurement of absolute viscosity is difficult and in practice it is more convenient to determine the viscosity of a particular liquid by comparison with a standard liquid which is usually water. The test results for different permeants are evaluated as a percentage higher than the viscosity of water and these percentages are presented in Table 3.8. The effect of viscosity and density ratio on hydraulic conductivity will be discussed in detail in Chapter 7.

 Table 3.8: Absolute Viscosity of Permeants

Permeant	1M NaCl	2M NaCl	Leachate	Mod.	Methanol
			. (L)	Leachate	(80%)
· · · ·				(L+1M NaCl)	· · · ·
% higher than					
viscosity of	8	19	6	12	37
water					

# 3.5 CONCLUDING REMARKS

As a part of soil characterisation, soil physical and physico-chemical and mineralogical analyses were carried out for soil collected from four different locations of Victoria and the results are tabulated in the appropriate sections. Although results from particle size distribution show that all the soils have different amounts of clay, silt and sand particles, still the soils are suitable as liner material from the criteria discussed in Section 3.2.1. Sunshine soil has less clay and more silt and sand contents which gave higher hydraulic conductivity than for other soil. Melton, Werribee and Echuca soil have less sand and more clay contents. Melton soil features very high clay content compared to Echuca and Werribee.

Test results from Atterberg limits show that Sunshine soil displayed low plasticity whereas other soils displayed high plasticity. Again on the basis of the criteria discussed in Section 3.2.1, all the soil should be suitable for the liner material. The data of some selected chemical properties (EC) revealed all the soils to be saline and the soil reaction (pH) ranges from slightly alkaline (Echuca) to moderately alkaline for other soils. The soil texture is clayey sand (Sunshine) and sandy clay for other soils. Cation exchange capacity (CEC) of Melton is higher than CEC of the other soils. Higher values of four major cations in Werribee soils compared to Echuca soil also reflect the higher values of CEC in Werribee soil.

XRD results revealed the presence of both swelling (smectite and Vermiculite) and nonswelling (kaolinite, illite and chlorite) minerals in all soils. Illite and kaolinite are present in significant amounts in both coarse and fine fractions of all soils. In the finer fraction illite is the major component in Sunshine, Melton and Echuca soils. Vermiculite is also present as a major clay mineral in the finer fraction in Echuca soil. Smectite is the dominant clay mineral for Werribee soil as would be expected from the residual basaltic origin of these soils.

# 4 LABORATORY HYDRAULIC CONDUCTIVITY TESTING

# 4.1 INTRODUCTION

As discussed in Chapter 1, the primary objective of the research work is to undertake a comprehensive laboratory investigation into the hydraulic conductivity of local clay soil used in liner construction. Hence, a thorough investigation of the laboratory hydraulic conductivity measurement was carried out on the soil and leachate samples characterised in Chapter 3. This investigation of hydraulic conductivity testing was mainly undertaken by using flexible wall permeameters (FWPs). FWPs used for this research work are ELE International Tri-Flex 2 units having a main panel with one cell and an auxiliary panel having two cells. The three cells were concurrently used for the whole testing program. A few tests were also carried out using consolidation cell permeameter and compaction mould permeameter.

This chapter describes the general testing equipment and procedures followed in the laboratory measurement of the hydraulic conductivity. It is evident from the literature review in Chapter 2, that the results of laboratory hydraulic conductivity measurement are strongly dependent on the equipment, testing techniques employed and on the type and composition of the permeants used. Taking this aspect into account, a laboratory test program was designed for the measurement of hydraulic conductivity. The procedures followed in the laboratory testing program are also discussed in this chapter. Minor variations in testing, if any, for different parameters will be discussed in the respective chapters. The schedule for hydraulic conductivity testing for individual parameter and the results of these investigations are presented in subsequent chapters.

As part of characterisation and establishment of the moisture content and hydraulic conductivity relationship for compacted clay used as a liner material, laboratory
hydraulic conductivity tests were carried out. The results of this investigation are also presented in this chapter. Laboratory test results and the published data sets on relationships linking dry unit weight and hydraulic conductivity with moisture content for compacted clays were analysed and linear regression analyses were performed. The results of these analyses are also presented in this chapter.

### 4.2 HYDRAULIC CONDUCTIVITY TESTING EQUIPMENT

Hydraulic conductivity tests are normally carried out in the laboratory either by using Rigid Wall Permeameter (RWP) or Flexible Wall Permeameter (FWP). RWPs are compaction mould permeameters which can be in the forms of double or triple ring permeameters, fixed cylinder permeameters, sampling tube permeameters and consolidation cell permeameters. FWPs use triaxial cells or modified triaxial cells for conducting hydraulic conductivity testing. Out of these double ring permeameters, consolidation cell permeameters and FWPs are widely used for laboratory investigation. A general description of the above equipment with their advantages and disadvantages and laboratory procedures followed in this research work are discussed in detail in the following sub-sections.

#### 4.2.1 Rigid Wall Permeameters

#### Compaction Mould Permeameter

The compaction mould permeameter usually consists of a traditional compaction mould, two end plates and a collar placed above the mould to contain the permeant liquids. Permeation is generally carried out from top to bottom. This type of permeameter is the simplest and most economical for testing compacted clays. The disadvantages of the compaction mould device are the possibility of incomplete saturation, lack of capabilities for measuring the amount of shrinkage or swelling, lack

of complete control over stress changes and potential for side wall leakage (Daniel *et al.* 1985). Side wall leakage could be easily detected by using the modified compaction mould device known as the double ring permeameter. In this type of mould both the inner and outer rings are of the same diameter and any difference in collected effluent between the two ring sections can be attributed to side wall leakage.

During the testing program given in this thesis, a 10.2cm diameter compaction mould permeameter was modified into a double ring permeameter and was used for compatibility testing. The inflow line was connected to the Tri-Flex 2 equipment to maintain constant pressure and a pressure interface chamber was connected to the inflow line to separate chemicals from water, in the case of compatibility testing involving chemicals. The outflow was collected from inner half and the outer half of the mould separately and was compared to check the side wall leakage. This set-up is shown in Figure 4. 1.

#### Consolidation Cell Permeameter

In consolidation cell permeameters, steel consolidation rings of diameter 5 to 8 cm and a thickness of 1 to 2 cm are normally used as the mould. The soil specimen is consolidated by applying vertical pressures of the desired magnitude. The vertical deformation of the specimen can be monitored with a dial gauge or displacement transducer. Permeation is initiated by raising the hydraulic head at the base of the specimen and flowing permeant up through the soil. A main advantage of consolidation cell permeameters is the ability to apply vertical pressure that approximately simulates the field condition. The application of effective stress could suppress side wall leakage. In addition, this equipment has the ability to measure the vertical deformation. The primary disadvantages are the potential for side wall leakage, higher cost than compaction mould permeameter, difficulty for testing at low effective stresses and the thinness of the specimen. However, some researchers argue that the consolidometer test is best suited to model the full scale behaviour of a clay liner (Peirce *et al.*, 1986).



Figure 4.1: Test set-up for Compaction Mould Permeameter

The standard consolidation cell permeameter used for the research program has a sampling ring diameter of 75 cm. Two GDS pressure volume controllers are connected to the permeameter to accurately monitor the pressure and volume of the permeant flowing through the specimen. GDS controllers used in testing can read pressure up to 1 kPa and volume up to three decimal places of a millilitre. The effective stress was controlled by connecting to the Tri-Flex 2 unit and the vertical deformation was measured by a dial gauge to indicate the volume change. The volume change of the specimen was also measured from the Tri-Flex 2 burette used for effective stress application. The consolidation cell permeameter with two pressure volume controllers in a full testing condition is shown in Figure 4.2.

#### 4.2.2 Flexible Wall Permearmeter

Flexible wall permeability tests are performed in triaxial cells or modified triaxial cells. Interchangeable base pedestals and top caps permit the testing of specimens with diameters from 4 to 15 cm. Double drainage lines to both the top and the bottom of the specimen facilitate the flushing of air bubbles from hydraulic lines. Separate pressure controls maintain the cell pressure and the two pressures acting on the ends of the soil specimen. The specimen is normally back pressured prior to permeation to ensure full saturation. The advantages of flexible wall permeameters include ability to use the undisturbed specimens, the use of back pressure saturation that confirms the level of saturation, measurability of volumetric deformations of the soil and the ability to control vertical and horizontal stresses. As was discussed in Chapter 2, there is always a possibility of forming cracking or creation of higher effective pore volume in the specimen due to structural changes induced by chemicals. One major disadvantage associated with FWPs is the possibility of closing these cracks due to the all around pressure applied in the cell and this, in fact, could mask any changes in hydraulic conductivity. Nevertheless, this equipment is the most commonly used for commercial and for research compatibility testing.



Figure 4.2: Test set-up for Consolidation Cell Permeameter

The ELE International Tri-Flex 2 flexible wall permeameters used in the testing program, feature a base plate diameter of 100 min. However, traditional compaction moulds are normally 102 mm in diameter. Special mould and compaction techniques were employed for the preparation of specimens. In this technique, Proctor compaction bulk density and the corresponding predetermined moisture content was maintained at constant level and for more uniformity a 50 mm thick specimen was compacted in 5 layers. The compacted specimen with porous stones and filter paper disks on each end was wrapped with few rounds of teflon tape to provide additional resistance to chemicals attaching to the surrounding rubber membrane. The wrapped specimen was then placed on the permeameter bottom cap and the top cap was placed on top of the specimen. The whole assembly was then covered with a thick latex rubber membrane and two o-rings were placed on each end of the specimen to seal the membrane and the top and bottom cap. The cell was filled with tap water and the drainage lines were flushed until no air bubbles were visible. De-aired water was used for saturation and permeation from a deairing tank connected to the equipment to avoid entrapped air in the specimen. Hydraulic conductivity tests were carried out using constant head technique and the standard method was followed. The co-efficient of hydraulic conductivity (m/s) was calculated as:

 $\mathbf{k} = \frac{\Delta qL}{hA\Delta t}$ 

where  $\Delta q = \text{flow volume (m}^3)$ L = length of the specimen (m) h = hydraulic head (m) A = cross sectional area (m<sup>2</sup>)  $\Delta t = \text{time (s)}$ 

Pressure interface chambers (PICs), fitted with Viton diaphragm and a cylinder either of stainless steel or of epoxy material, are normally used during hydraulic conductivity testing involving toxic permeants. PIC provides a closed system for toxic liquids entering and exiting the cell. The portion of the PIC retaining toxic permeants are also fitted with stainless steel valves and connections to provide adequate protection against

(4.1)

any chemical attack. Two PICs were used in each cell and the set-up is shown in Figure 4.3.

# 4.3 TERMINATION OF HYDRAULIC CONDUCTIVITY TESTING

Researchers have put forward different arguments regarding the termination criteria of compatibility testing for hydraulic conductivity with leachates and chemicals. Pierce and Witter (1987) suggested that clay hydraulic conductivity tests be continued until: (1) at least one pore volume of permeant is passed through the clay specimen; and (2) the hydraulic conductivity does not differ significantly with the number of cumulative pore volumes time. It seems to be unrealistic to pass only one pore volume of permeant through the soil specimen because this would probably not remove all the soil water from the pore spaces and structural changes, if any, would not occur at that point. In relation to this, Bowders et al. (1986) pointed out that at least one pore volume of flow is required for testing with water. Bowders et al. (1988) presented data which indicated that a significant increase (up to 17 times) in the hydraulic conductivity of a compacted specimen of a kaolinite to acetic acid occurred suddenly and only after more than six pore volumes of flow. Unfortunately, for low conductivity soils, passing a large number of pore volumes will extend the testing time to several months or years. Bowders *et al.* (1986) suggested that compatibility testing should be continued at least for: (1) two pore volumes of the permeant to ensure that the original soil water in the test specimen is replaced by the waste liquid; (2) the influent to effluent concentration ratio approaches unity. These two criteria seem to be well accepted as the termination criteria of compatibility testing for hydraulic conductivity and were followed during the testing program. In the testing program, the emphasis was given to the duration of testing, which is expressed as the pore volumes of flow. However, the effluent concentration was also monitored in majority of the tests.



Figure 4.3: Test Set-up for Flexible Wall Permeameter

# 4.4 HYDRAULIC CONDUCTIVITY TESTING

#### 4.4.1 Specimen Preparation

Soils used in the testing program were collected from Sunshine, Werribee, Melton and Echuca. The location where the soils obtained and their other characteristics were given in Chapter 3. After collection from the field, soil samples were air dried and big lumps were broken into small pieces by rubber hammer and by soil crusher. In accordance with Australian Standard AS 1289. 1 1991 Method 1 for preparation of disturbed soil samples for laboratory testing, the crushed samples were sieved through 10 mm sieve and mixed thoroughly before splitting into sub samples. These sub samples were then stored in sealed polythene bags for future testing.

As can be found from the discussion on Section 4.7, that the hydraulic conductivity for the compacted clay is minimum when the moisture content is 2-4 percent wetter than optimum. For preparation of specimen for hydraulic conductivity testing, the required amount of tap water was mixed with dry soil to make the moisture content about 2 percent wetter than optimum, to target for the minimum hydraulic conductivity. Subsequently, the soil was allowed to cure for 24 hours in order to achieve a uniform distribution of moisture in the soil. The soil specimens were created by compacting the soil to standard Proctor dry density corresponding to the required moisture content as set out from compaction tests in Chapter 3. The soil specimens were then surrounded by several rounds of Teflon taping and a thick rubber membrane before it was placed in flexible wall permeameters. Saturation of the specimens was carried out by applying back pressure to both ends and a small gradient was applied to get rid of air bubbles from the specimen. The air present in the void spaces of the specimen is dissolved into the pore liquid due to an increase in partial pressure of the air resulting in an increase in the degree of saturation. The saturation was carried out in steps and with a small increment in cell pressure, so that the soil specimen is not disturbed. The cell pressures

were used as high as 540 kPa and effective stresses were kept constant throughout the saturation and testing period. Most of the tests were carried out at 100 kPa effective stress and a few were carried out at varying effective stresses as required. After the saturation (B value  $\geq$  95%), the permeation was started under the desired conditions.

As in the case of FWPs, the same procedure was followed for the preparation of the specimens for RWPs. Saturation as well as permeation in the compaction mould permeameter was carried out from top to bottom by increasing the influent pore pressure. Saturation and permeation in consolidation cell permeameter was carried out from bottom to top with a set pressure difference between the two.

#### 4.4.2 Leakage Test in the Tri-Flex 2 Equipment

Low flow hydraulic conductivity tests (involving clay) are usually long duration tests. Hence, a high level of precision is essential to obtain accurate results. Possible leakage from the test apparatus during testing can pose a significant impact on the precision of low gradient hydraulic conductivity test results. Leakage can be defined as a component of flow rate not associated with permeant movement through the soil specimen during testing (Huag *et al.*, 1994). Tavenas *et al.* (1983) stated that the sources of error in triaxial installation are: (1) the mechanical leakage in external fittings; (2) leakage in fittings within the cell; and (3) osmosis and diffusion through the membrane. Leakage within the cell can be reduced by careful construction and tightening of all the fittings. Osmosis and diffusion through the membrane can be reduced by using a viscous cell fluid (Tavenas *et al.*, 1983) or by vegetable oil (Hossain, 1995). Figure 4.4 shows a leakage flow pattern in a triaxial cell.

Huag *et al.* (1994) devised a technique for calculating the time required for measuring a minimum amount of leakage for different resolutions of burette. They also suggested a leakage rate of 1 x  $10^{-9}$  ml/s may be reasonably anticipated in a test setup. Leakage could easily be detected by measuring the inflow and outflow volume of the permeant.



(source: Hossain, 1994)

It may also be necessary to measure the volume change of the specimen during testing in order to evaluate the effect of chemical or physical processes. When any changes in volume of the specimen occurs, the measurement of inflow and outflow alone will not be able to explain the likely leakage.

A laboratory testing program was undertaken on the flexible wall permeameter using an impermeable aluminium block as a 'dummy' test specimen to check the leakage. The use of solid aluminium blocks have the advantages of rigidity with no consolidation or no change in volume at the effective stress used and there will be no flow through those blocks. A thin layer of high vacuum silicone grease was used on the faces of the aluminium block to seal the rubber membrane properly. The fluid pressures used during the tests were identical to those used during saturation and flexible wall hydraulic conductivity testing. Minor modification was made in the standard cell connectors to prevent leakage. One test was carried out for each cell and was continued for at least one week and the results are presented in Appendices A4.1 to A4.3. From the test results, it was found that there was no measurable leakage within the accuracy of the measuring system even after one week of testing time. Minimum measurable volume change in Tri-Flex 2 burette is 0.1 ml.

#### 4.4.3 Compatibility Testing of Equipment and Latex Membrane

As stated in previous Chapters, the present research aimed at studying how landfill liners work under the field conditions, using laboratory techniques. Landfill leachate generated from waste has to pass different stages of degradation, acetogenic and methanogenic, where acetic acid and methanol are produced (Barlaz & Ham, 1993). Taking these aspects into account, initial consideration of testing programs was given to acetic acid and methanol. Long term compatibility testing was carried out with different concentrations of acetic acid and methanol for the latex membrane and the interface chamber. These tests were carried out by submerging pieces of latex membrane in these chemicals for a significant period of time. It was found that methanol, even at 100% concentration, did not pose any problem to the latex membrane and interface chambers. Acetic acid, however, even at 60% concentration, causes deterioration of latex membrane, diaphragm and epoxy material of the interface chamber itself. Therefore, the use of acetic acid for hydraulic conductivity and compatibility testing was abandoned. As far as Echuca soil is concerned, the emphasis was on the influence of saline water on the soil. Hence, these soils were tested for saline water as the permeant. Furthermore, landfill leachates also contains varying levels of salinity. Hence, saline water was generally used as a permeant for all soils. Compatibility testing with saline water and leachate collected from PWM Lyndhust site was also carried out and it was found that latex membrane and pressure interface chambers are compatible with leachate and saline water even at very high concentrations.

# 4.5 CHEMICAL ANALYSES OF INFLUENT AND EFFLUENT SAMPLES

Effluent samples were collected at a regular interval (of about 0.5 PV), while the hydraulic conductivity testing was in progress. The effluent samples collected during testing were analysed for pH, electrical conductivity, sodium and chloride contents for NaCI solution by using atomic adsorption spectroscopy (AAS) at the Chemical Sciences Laboratory, VUT. Similar tests for influent samples were also carried out and the inflow-outflow results were compared before the termination of the hydraulic conductivity testing. Testing for turbidity of the effluent samples was carried out to monitor particle migration from soil specimen at the Water and Environmental Engineering Laboratory of the School of The Built Environment, VUT. The equipment used for such testing is HACK DR2000. Results of the analyses are presented in the subsequent chapters.

# 4.6 POST TESTING ASSESSMENT

Post testing assessment of the soil specimens was carried out by visual observation of any physical changes or cracks forming within the specimen. The thickness of the specimen could also be measured to assess the volume change occurring during chemical permeation. Results of these assessments are presented in respective chapters. Sub samples of soil may be used to obtain moisture content, mineralogical and chemical analyses, bacterial counts, pore size distribution by mercury intrusion and micro fabric studies by scanning electron microscopy (SEM) (Fernandez, 1989). Pore fluid of the tested specimen can be extracted using the mini-distillation apparatus and the organic contents of the liquid extracts could be determined using gas chromatography. These results could be used to interpret the changes in the hydraulic conductivity of the soil tested.

#### 4.7 MOISTURE CONTENT RELATIONSHIP FOR COMPACTED CLAY

As discussed before, the minimum hydraulic conductivity is the most desirable property of the compacted clay liners. It is well established from laboratory studies that for a particular soil type, this parameter is a function of the moisture content, and displays a minimum value at a certain moisture content for a given compactive effort. Hence, it is most appropriate to specify a field moisture content as practically close as possible to the moisture content that gives the minimum hydraulic conductivity value. But the laboratory hydraulic conductivity testing of compacted clays, particularly the long term tests, is very time consuming and very expensive to run. Therefore, an easy and quick method of assessing the moisture content for a particular soil is very important.

According to the current practice, however, the specification of field moisture content is relative to the traditional optimum moisture content (OMC), which gives the maximum dry unit weight, as derived from standard laboratory compaction tests. Laboratory

studies have indicated that the moisture content, which gives minimum hydraulic conductivity, falls a few percentage points wet of optimum moisture content. Hence, it is common to specify the field moisture content for compaction of soil liners, a few percentage points wet of optimum moisture content with the intention of achieving a minimum hydraulic conductivity while paying attention to other factors such as trafficability and constructability of the liners. Daniel and Benson (1990) proposed a methodology where minimum hydraulic conductivity and other considerations can be taken concurrently into account in the compaction specification. In their methodology, specifications would be given as a window of acceptable combinations of moisture content and dry unit weight that satisfy the desirable qualities of the liner. Α prerequisite to this methodology, however, is the knowledge of the variation of hydraulic conductivity with the moisture content around the minimum value of the hydraulic conductivity. In order to guide the overall testing program into hydraulic conductivity measurement, the relationships of hydraulic conductivity and moisture content were established for test soils.

However, it is clear that accurate determination of the moisture content giving minimum hydraulic conductivity is important, and this is not currently possible unless a series of laboratory hydraulic conductivity tests, at a significant expense of time and cost, are carried out. It follows then that the methods of deriving this information from relatively cheap compaction tests would be valuable. In this study, this is explored making use of laboratory test results conducted as part of this overall program and published data on relationships between dry unit weight and hydraulic conductivity with the moisture content.

#### 4.7.1 Hydraulic Conductivity Relationships for Test Soils

Experimental study was carried out to establish the relationship of hydraulic conductivity with pore volume using water as permeant. Rigid wall permeameters and flexible wall permeameters were used for such study. The preparation of specimen and

the testing techniques discussed previously were followed for the same. Flexible wall Tri Flex - 2 units, used in the testing program, have three burettes for measuring three volumes: (1) the cell volume; (2) the inflow volume; and (3) the outflow volume. The cell volume burette measures the volume change within the cell during testing. If the effective stress is greater than the pre-consolidation pressure, the specimen will undergo a reduction in the volume due to the effective stress consolidation and time dependent secondary compression. If by any means, chemicals or salinity induce a net repulsive force in the specimen, as is discussed in Chapters 7 and 8, and if this is sufficient to overcome the applied effective stress, then there will be an increase in the volume. In an equilibrium flow condition, inflow and outflow volumes should be the same. Any change in these two flows during testing might reflect either a leakage in the cell or the changes occurring within the specimen. As discussed in Section 4.4.2, no significant leakage was detected in the equipment. Therefore, the differences between the inflow and outflow volumes measured during the testing could be considered to reflect the change occurring within the specimen. Changes of these two volumes i.e. the cell volume and the (outflow - inflow) volumes, were monitored during testing and are presented along with the hydraulic conductivity calculated on the basis of inflow and outflow volumes in Figures 4.5 and 4.6.

Figures 4.5(a) and 4.6(a) present the results of the hydraulic conductivity versus pore volume where water was used as a permeant. It is clear from these figures that the reduction in hydraulic conductivity is rapid initially and this reduction slows down with the passage of time. The reason for this initial decrease could be due to the consolidation of the specimen by the effective stress and the gradient, which reduces the pore space resulting in the reduction in hydraulic conductivity. As the time passes, due to the secondary compression effect, the soil specimen continues reducing the volume at a slower rate due to the readjustment of the particles. This secondary compression continues for a long time, which eventually leads to the reduction in hydraulic conductivity. In cases of FWPs, this trend could continue for some time even after the application of chemical permeants. This type of time lag is not probably very significant in cases of compaction mould and consolidation cell permeameters because

of the side wall leakage and the thinness of the specimen. Therefore, it would be convenient to consider the minimum hydraulic conductivity for a particular specimen for compatibility assessment with FWPs and, hence, the technique is followed in Chapters 7 and 8. The differences between the inflow and outflow hydraulic conductivity values are very minimal and the results presented in these figures are from the inflow values. Therefore, one possible direction for analysis could be to consider the volume changes during testing.

From Figure 4.5(b), it can be seen that the cell volume increase is high initially followed by a more constant cell volume. This initial increase in cell volume may be due to the consolidation of the specimen. Once the consolidation is over, the cell volume should keep constant unless any change in the specimen occurs. Figure 4.6(b) also shows the initial increase in the cell volume similar to that of Figure 4.5(b). The tail portion of the curve shows relatively small increase in the cell volume. With the specimen consolidation, some of the water might be squeezed out from the specimen, and outflow volumes should be more than the inflow volumes if other mechanisms are not active. From the test results in the Figures 4.5(b) and 4.6(b), it could be seen that the specimen saturation might have overridden the consolidation effect. Inflow volumes become higher than the outflow volumes in the initial portion of the curve. This increase is higher initially and gradually reduces with time. As discussed previously, saturation of the specimen was achieved over 95%, but however, there may have been still requirement of water to get 100% saturation. As the saturation process advances, consolidation also takes place at a rapid rate and both processes go on simultaneously. During consolidation outflow might be higher than the inflow. Therefore, consolidation or saturation, depending on which mechanism dominated the process, the value of volume change given by (outflow-inflow) could be either positive or negative.

Experimental study was carried out to examine the proposed proposition made in the previous section regarding dry unit weight and hydraulic conductivity with the moisture content. Soil samples from Sunshine and Werribee were used for this study. As



Figure 4.5(a): Hydraulic Conductivity vs Pore Volume Relationship for Echuca Soil

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Figure 4.5 (b): Volume Change vs Pore Volume Relationship for Echuca Soil



Figure 4.6(a): Hydraulic Conductivity vs Pore Volume Relationship for Werribee Soil



Figure 4.6 (b): Volume Change vs Pore Volume Relationship for Werribee Soil

discussed in Chapter 3, these soils are classified as clayey soil and sandy clay respectively. Dry density – moisture content, and wet density – moisture content curves presented in Appendices A3.9 and A3.12 from standard Proctor compaction tests are used for the analysis.

As discussed in the testing technique in the previous section, hydraulic conductivity tests were carried out with water as a permeant. A wide range of moisture content was chosen from compaction test results to establish the moisture content relationship corresponding to minimum hydraulic conductivity. The results of the investigation are presented in Figure 4.7 and 4.8 respectively for Sunshine and Werribee soil.

#### 4.7.2 Theory

A number of researchers have attempted to explain theoretically why the moisture content giving minimum hydraulic conductivity occurs wet of optimum moisture content (*eg.* Lambe, 1958; Mitchell *et al.*, 1965; Daniel, 1984). These explanations include changes in clay structure at micro and macro scale. At micro scale, it is considered that clay structure changes from a flocculated to a de-flocculated (dispersed) clay structure when the moisture content is increased beyond optimum. At macro scale, there is direct experimental evidence to indicate that the compacted soil contains clods of clay with relatively large interconnected voids at and below optimum moisture content. Hence, although the dry unit weight is at a maximum (and theoretically the void ratio at a minimum) at the optimum moisture content, the hydraulic conductivity is higher than the minimum attainable. When more water is added raising the moisture content above the optimum, compaction remoulds these clods and reduces the size and the interconnectivity of the voids, thereby reducing the hydraulic conductivity. Further addition of water beyond the point when the minimum conductivity tends to increase.

In soil mechanics, the dry unit weight  $\gamma_{dry}$ , is defined by the following relationship:

$$\gamma_{dry} = \frac{G_s \gamma_w}{1+e} \tag{4.2}$$

where:  $\gamma_w$  is the unit weight of water; Gs is the relative unit weight of soil particles; and e is the void ratio of the soil.

With the moisture content w, the wet unit weight of the soil  $\gamma_{wet}$ , is expressed as:

$$\gamma_{wet} = \frac{(1+w)G_s\gamma_w}{1+e} \tag{4.3}$$

Hence, the wet unit weight and dry unit weight can be related as:

$$\gamma_{wet} = (1+w) \ \gamma_{dry} \tag{4.4}$$

As can be seen from Equations (4. 2) and (4.3), it is clear that only the wet unit weight takes into account the amount of moisture present in the soil medium. A closer inspection of the relationship given in Equation (4.4) indicates that the maximum wet unit weight should occur wet of optimum moisture content. In the current research work, it is postulated that the moisture content that gives the maximum wet unit weight would be better correlated and approximated to the moisture content giving minimum hydraulic conductivity. This proposition will be examined in the following sections making use of some laboratory test results and the published data.

#### 4.7.3 Data

The data used in the analysis include results of tests carried out in the laboratory on two local soils and the data obtained from the literature. A literature review was undertaken to search for consistent sets of data of moisture content relationships with compaction

and hydraulic conductivity. Twelve sets of data were found and the results are presented in Table 4.1. The optimum moisture content  $(w_{opt})$  and the moisture content at minimum hydraulic conductivity  $(w_k)$  were obtained from the curves provided by the authors for each data set. For the calculation of the moisture content at maximum wet unit weight  $(w_{wet})$ , only the data points given in the dry unit weight against moisture content curves were used. Using Equation (4.4), these points were re-plotted as wet unit weight against moisture content, and then,  $w_{wet}$  was estimated from a curve fitted through these points. Figures 4.7 and 4.8 illustrate the curves used for a data set of laboratory testing.

#### 4.7.4 Analysis

The data presented in Table 4.1 relate to clay soils of a reasonably wide range of plastic limits (15-35%) and plasticity indices (11-45%). Hence, it appears to be of some value in analysing this data to examine the likely correlation of compaction moisture content parameters to hydraulic conductivity typical of clay soils. Table 4.1 is arranged in such a way that: the data sets (a) to (h) are based on standard Proctor compaction and flexible wall permeameters; the data sets (i) to (k) are based on modified Proctor and flexible wall permeameter; and the data sets (l) to (n) are based on non-standard equipment and methods.

The analysed data containing data sets (a) to (k) are presented in Table 4.2. This analysis indicates that  $w_k$  is closer to  $w_{wet}$  than  $w_{opt}$  in all cases, and the ranges of  $w_k$ - $w_{opt}$  and  $w_k$ - $w_{wet}$  are 1 to 6.1% and -0.7 to 3.6% respectively. Linear regression analyses were performed to examine the correlation of  $w_k$  to  $w_{opt}$  and  $w_{wet}$ . Figures 4.9 and 4.10 illustrate the results of the linear regression and correlation analyses of data sets (a) to (h) and (a) to (k) respectively. Table 4.3 presents linear relationships established for these data sets. While recognising that the data used in the analysis are limited, it may be argued that  $w_{wet}$  is better correlated to  $w_k$  than  $w_{opt}$ . As can be seen



Figure 4.7: Hydraulic Conductivity - Density - Moisture Content Relationship for Sunshine Soil



Figure 4.8: Hydraulic Conductivity - Density - Moisture Content Relationship for Werribee Soil

from Table 4.2 the plastic limit is also close to  $w_k$  for standard compaction test results (data sets (a) to (h)), but the analyses indicated that the regression and correlation are less strong in comparison to  $w_{wet}$ .

In addition to the above analysis, the data compiled in the current paper can be used to explore the relationship of  $w_k$  to index properties of clay soils. Skempton and Northey (1953) studied the strength of remoulded saturated clays. They showed that the undrained shear strength of clay (c<sub>u</sub>) can be expressed as a function of the liquidity index of the clay. Based on this relationship, Leroueil et al. (1992) proposed the following equivalent relationship for compacted clays:

$$C_u = f\left(\frac{w - w_{opt}}{PI}\right) \tag{4.5}$$

Based on experimental results, they developed an exponential relationship for  $c_u$  with the liquidity index. It was noted previously that at  $w_k$ , it is likely that clods of clay are remoulded under the compaction hammer. This may be viewed as if soil undergoes bearing failure due to the compaction stress. For undrained clay, the bearing stress at failure is about 5 times  $c_u$ . Hence, it can be argued that at  $w_k$ ,  $c_u$  is a constant dependant on the compaction stress. It follows from Equation 4.5 that for a given compaction energy,  $(w_k - w_{opt})/PI$  is a constant. This postulation is examined by considering the data sets (a) to (h) which correspond to standard compaction tests. As shown in Table 4.2,  $(w_k - w_{opt})/PI$  values are in the range of 0.05 to 0.36 with a mean value of 0.13. Based on these limited data, it may not be fair to completely reject this postulation, and may need to be further assessed when more data are available. On the basis of current analysis, however, the following relationship may be proposed:

$$w_k = w_{out} + 0.13PI \tag{4.6}$$



Figure 4.9: Linear Regression for the Data Sets (a) to (h) - Standard Proctor



Figure 4.10: Linear Regression for the Data Sets (a) to (k) - Standard and modified Proctor

# Table 4.1

Compaction and Hydraulic Conductivity Data

		-)				
Reference	Compaction technique, permeameter used	Type of soil	Plastic limit (PL),Plasticity	w <sub>opt</sub> (%)	w <sub>wet</sub> (%)	<sup>w</sup> k (%)
(a) Sunshine Soil Figure 4.7	standard Proctor, flexible wall	Mixture of sand, silt and clay (SC/CL), Sunshine	$\frac{\text{PL} = 15}{\text{PI} = 21}$	15.9	17.6	16.9
(b) Werribee Soil	standard Proctor,	Sandy clay (CH),	PL = 32	31.5	33.0	34.5
	flexible wall	Werribee	PI = 42			
(c) Benson and Othman (1993)	standard Proctor,	low plasticity clay	PL = 18	16.7	17.3	18.8
	flexible wall	(CL)	PI = 14			
(d) Daniel and Benson (1990)	standard Proctor,	Type A	PL = 28	29.2	29.5	30.6
	flexible wall♥	Mixture of sand silt	PI = 27			
		and clay				,
(e) Daniel and Benson (1990)	standard Proctor,	Туре В	PL = 16	17.5	18.6	18.5
(1770)	flexible wall♥	Sandier than	PI = 18			
		Type A soil				
(f) Day and Daniel (1985)	standard Proctor,	Clay 1 (CL)	PL = 19	18.0	19.6	22.0
(17.02)	flexible wall♥		PI = 11			
(g) Day and Daniel (1985)	standard Proctor,	Clay 2 (CH)	PL = 27	28.4	30.9	34.5
	flexible wall♥		PI = 45			
(h) Boynton and Daniel (1985)	standard Proctor,	Kaolinite	PL = 35	31.6	32.7	35.0
	flexible wall		PI = 23			
(i) Benson and Othman (1993)	modified Proctor,	low plasticity clay	PL = 18	13.1	13.7	16.0
	flexible wall	(CL)	PI = 14			
(j) Daniel and Benson (1990)	modified Proctor,	Type A	PL = 28	24.1	25.5	26.3
	flexible wall♥	Mixture of sand silt	PI = 27			
		and clay				
(k) Daniel and Benson (1990)	modified Proctor,	Туре В	PL = 16	13.0	13.7	14.0
	flexible wall♥	Sandier than	PI = 18			
		Type A soil				
(l) Mitchell <i>et al.</i> (1965)	non standard	Not Available (NA)	NA	8.53	9.2	9.31
(m) Lambe (1958)	non standard	Jamica sandy clay	NA	14.0	14.4	15.0
(n) Lambe (1958)	non standard	Siburua clay	NA	11.0	12.0	12.0

•Other permeameters in addition to flexible wall were used. Results from all tests as given in the compaction curves were used and the results were similar.

# Table 4.2Analysed Data

_														
	Author(s)	(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)	(k)	Mean	range
	<sup>w</sup> k <sup>- w</sup> opt (%)	1.0	3.0	2.1	1.4	1.0	4.0	6.1	3.4	2.9	2.2	1.0	2.55	1 to 6.1
	wk <sup>- w</sup> wet	-0.7	1.5	1.5	1.1	-0.1	2.4	3.6	2.3	2.3	0.8	0.3	1.36	-0.7 to 3.6
	w <sub>k</sub> - PL (%)	1.9	2.5	0.8	2.6	2.5	3.0	7.5	0.0	-	-	-	2.60	0 to 7.5
	(w <sub>k</sub> - w <sub>opt</sub> ) /PI	0.05	0.07	0.15	0.05	0.06	0.36	0.14	0.15	-	-	-	0.13	0.05 to 0.36
	(w <sub>k</sub> -PL) /PI	0.09	0.06	0.06	0.10	0.14	0.27	0.17	0.0	-	-	-	0.11	0 to 0.27

TABLE 4.3Regression Relationships

Linear regression for the data sets (a) to (h);	Linear regression for the data sets (a) to (k);					
Standard Proctor compaction	Standard and modified Proctor compaction					
$\frac{W_k}{W_{wet}} = 1.061$ $\frac{W_k}{W_{opt}} = 1.116$	$\frac{W_k}{W_{wet}} = 1.059 \qquad \qquad \frac{W_k}{W_{opt}} = 1.114$					

# 4.8 CONCLUDING REMARKS

In this chapter, a general description of the testing equipment and procedures for the measurement of the hydraulic conductivity are presented. This includes the discussion of basic features of various permeameters available for hydraulic conductivity testing with their advantages and disadvantages and their suitability at different conditions. Also presented in this chapter are the hydraulic conductivity test results carried out as

part of the characterisation tests for the establishment of the relationship for dry unit weight and hydraulic conductivity with moisture content.

On the basis of the literature review and the characterisation of soil and chemicals, a series of hydraulic conductivity tests were carried out in the laboratory by using FWPs, consolidation cell permeameters and compaction mould permeameters. The testing schedule for each parameter is given in the respective chapter. The description of the equipment used and the techniques employed for undertaking such investigation were also presented in this chapter. Hydraulic conductivity and compatibility tests using various chemical permeants are discussed in Chapters 7 and 8. Any deviations from the standard methods discussed in this chapter will be presented in the respective chapters.

From the FWP test results, it is evident from the pore volume versus hydraulic conductivity test results that reduction in hydraulic conductivity is higher initially due to the consolidation of the specimen. Cell volume change and the inflow-outflow volume changes were measured during testing. Cell volume change is higher initially due to the consolidation and drops down as the time passes. Inflow and outflow volume changes show a significant variation during a full test and are discussed in this chapter. Moisture content relationships for compacted test soil were also established and the results were discussed. Laboratory test results and published laboratory data on relationships of dry unit weight and hydraulic conductivity with moisture content for compacted clays were analysed. This included re-plotting the compaction curve so that the wet unit weight of soil could be plotted against moisture content. Based on linear regression analyses, it was established that the moisture content which gives maximum wet unit weight is a closer approximation and better correlated to the moisture content giving minimum hydraulic conductivity than the traditional optimum moisture content. The linear regression relationships were also established, but it should be noted that they are based on limited data. They may need to be updated when more data become available. Nevertheless, they may provide preliminary guidance to undertake hydraulic conductivity testing required for the design of compacted clay barriers.

# 5 EFFECT OF MOULDING MOISTURE CONTENT AND ATTERBERG LIMITS ON HYDRAULIC CONDUCTIVITY

## 5.1 INTRODUCTION

The low hydraulic conductivity is the most important parameter among the desirable properties of compacted clay liners as discussed in Chapter 2. This is important because it controls the rate of release of pollutant leachates through the liners. Therefore, it is not surprising that many environmental regulations specify liner hydraulic conductivity to be less than  $1 \times 10^{-9}$  m/s as a primary requirement for liner design and construction. The long term stability of the compacted clay liners involves the retention of the desirable properties discussed in Chapter 2, over the period in which the facility is likely to pose a threat to the geo-environment. Normally, the long term stability of a soil liner is measured by assessing the compatibility of soils used for liner construction with various chemical leachates to which they would be exposed during the life time of the facility. Most importantly, this should involve the assessment of the variation of hydraulic conductivity with time when exposed to chemical leachates. The laboratory testing for hydraulic conductivity of compacted clays, particularly the long term tests, is very time consuming. Furthermore, special laboratory facilities additional to the traditional geotechnical testing equipment are required to conduct tests involving Consequently, for the purposes such as soil selection and chemical leachates. compaction specifications, the hydraulic conductivity tests are still commonly conducted with water as the permeant. It follows then that it would be advantageous if simpler and quick methods are available to assess the stability of hydraulic conductivity when exposed over a long period to various chemicals that are found in the leachates.

Atterberg limits are the indices, which are commonly used to characterise the finegrained soils. They reflect the consistency of the soil, soil particle size and clay content, the structure of the soil particles and mineralogical composition. Because of these properties, researchers have argued that Atterberg limits can be correlated to various other engineering properties including hydraulic conductivity (*eg.* Leroueil et al., 1992). Mesri and Olson (1971) found a distinct trend of decreasing hydraulic conductivity with increasing plasticity of the soil. Based on these grounds and because the tests are easy and quick to perform, some researchers have examined the possibility of using the effects of the Atterberg limits with chemical leachates as a possible method of assessing the soil compatibility. In this chapter, this approach is examined for some Victorian soils used in liner construction. Results of Atterberg limits with different chemicals are useful in planning hydraulic conductivity testing for different chemicals.

# 5.2 **PREVIOUS RESEARCH**

Researchers have used various chemicals normally found in leachates at different concentrations to determine Atterberg limits of liner soil. They have compared these results with the limits obtained with water as in the standard method (*eg.* Daniel and Liljestrand, 1984; Bowders and Daniel, 1987; Acar and Olivieri, 1989; Cancelli et al., 1994). Chemicals used in these tests include neutral-polar liquids such as methanol (found in industrial feedstocks and solvents), organic acids such as acetic acid (municipal landfill leachate), and non-polar hydrocarbons such as benzene (waste oils). Besides these, brine (salt solutions at high concentrations), which is encountered in evaporation ponds and industrial wastes was also used in assessing the changes in Atterberg limits by a number of researchers (*eg.* Ho and Pufahl, 1987; Yang and Barbour, 1992; Huag et al., 1988; Ridley et al., 1984). The results indicate that while Atterberg limits are generally changed by the chemicals, the degree of change would depend on the clay content in the soil, the types of clay minerals, the physico-chemical properties and the concentration of the moulding fluids. These properties are discussed in detail in the following sections.

#### 5.2.1 Factors Influencing the Atterberg Limit

#### • Type of Clay Mineral and Clay Content

Atterberg limits are the indices of the quality of clay size particles and mineralogical compositions. Typically, higher liquid limit and plasticity index are associated with soils having a greater quantity of clay particles and clay particles having higher surface activity (Mitchell, 1976). This is because of the soil having a greater quantity of fines and clay, and more active clay minerals generally have a thicker double layer resulting in a very high liquid limit and plasticity index. As shown in Figure 5.1, Benson and Trast (1995) found that the plasticity index is directly related to the mineralogy of the soil and the clay content. This is also evident from Table 2.1 that the liquid limits and the plastic limits for smectite soil is the highest followed by illite and kaolinite. As discussed in Chapter 2, different groups of clay minerals exhibit a wide range of properties and will react differently in the presence of various chemicals at varying concentrations. Tests carried out by Bowder and Daniel (1987) on two soils, kaolinite and illite-chlorite mixture with different chemicals at various concentrations, found that the variation in the liquid limit depends on the soil type. For kaolinite soil, they found a three times increase in the liquid limit and for illite-chlorite soil this increase is about 1.5 times when the moulding fluid is trichloroethylene. Benson and Trast (1995) also found that the hydraulic conductivity is dependent on the liquid limit and the plasticity index of the soil. The results of their findings are shown in Figure 5.2(a) and 5.2(b).

#### • The Nature of the Pore Fluid

Researchers (*eg.* Barbour & Yang, 1993; Ho and Pufahl, 1987) found a decrease in the plasticity index of soil with the increase of moulding fluid concentration. They pointed out that the decrease in the plasticity index was due to the decrease in the liquid limit, or to little change or slight increase in the plastic limit. Increases in the moulding fluid concentration change the attractive and repulsive forces in the soil structure and are discussed later.



Figure 5.1: Mineralogy, Clay Content and Plasticity Index Realtionship (source: Benson and Trast, 1995)



Liquid Limit

Figure 5.2(a): Relationship Between Hydraulic Conductivity and Liquid Limit (source: Benson and Trast, 1995)



Figure 5.2(b): Relationship Between Hydraulic Conductivity and Plasticity Index (source: Benson and Trast, 1995)

As discussed in Chapter 2, the dielectric constant of the pore fluid is the one which significantly affect the properties of different types of clays. Bowder and Daniel (1987) found that the fluids having a dielectric constant greater than 40 did not significantly alter Atterberg limits. The effect of pore fluid on the liquid limit was studied by Sridharan and Rao (1975). In their study, the calculation of the liquid limit was made on a volume basis because the unit weights of the fluids were different from one another. They found that the liquid limit of kaolinite was decreased with the increase of dielectric constant whereas a large increase in the liquid limit was observed for montmorillonite with the increase of dielectric constant. An increase in dielectric constant decreases interparticle shearing resistance and increases double layer thickness, resulting in the higher liquid limit (Sivapullaiah & Sridharan, 1987). For kaolinite, a non-expanding lattice type of clay, the liquid limit is primarily governed by the shearing resistance and hence a decrease in the liquid limit was observed. For montmorillonite, the liquid limit should be governed by the shearing resistance and because it is an expanding lattice type of clay, contribution to the diffuse double layer overrides and governs the liquid limit (Sivapullaiah & Sridharan, 1987). The results of their study are shown in Figure 5.3.

#### • Inter-particle Forces

A detail description of the attractive and repulsive forces generated within the clay particles is discussed in Chapter 7. Changes in the liquid limit and the plastic limit could also be considered on the basis of attractive and repulsive forces. Interparticle forces play a prominent role in the liquid limit. At the liquid limit soil develops a low level of shearing resistance which appears to be the result of a long range of attractive stresses between clay particles (Mitchell, 1993). In this condition, the net repulsive stress has been dissipated through full expansion of the diffused double layer. Consequently, the liquid limit varies significantly with the surface area of the clay and the pore fluid chemistry. As the pore fluid concentration or valance increases, the repulsive stress between the particles and consequently the thickness of the diffuse double layer decreases, resulting in a decrease in the liquid limit (Ridley *et al.*, 1984).



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Figure 5.3: Effect of Dielectric Constant on Liquid Limit (source: Sivapullaiah and Sridharan, 1987)
At the plastic limit the soil particles are able to slide past one another to take up new positions and sufficient cohesion between the particles exists to hold them in these new positions. Barbour & Yang (1993) concluded from the test results that the plastic limit is relatively insensitive to changes in the pore fluid chemistry and surface area, and consequently must not be sensitive to physico-chemical effects. In some cases a slight increase in the plastic limit has been observed with increases in concentration.

## • Specific Gravity of Liquid

Atterbergs limits are the moisture content corresponding to the boundaries between the states of consistency. Hence, an accurate measurement of the moisture content is important for assessing the changes in Atterberg limits. With a dilute moulding fluid, the moisture content of soil is simply determined as the ratio of the mass of the fluid to the mass of the soil solids. If significant quantities of salt are present in the moulding fluid, the measured dry mass actually represents the mass of soil solids plus the dissolved salt in dry form. The mass of the dry soil has to be corrected for the mass of dissolved salt.

Storey and Peirce (1989) found a significant decrease in the liquid limit with the increase of methanol concentration when the moisture content was expressed on a mass basis. When the results were expressed on a volume basis all the results were almost identical. Therefore, the result pointed out the importance of expressing the results in both mass and volume bases. In this current study, Atterberg limits are expressed in both mass and volume bases. Atterberg limits obtained from the tests are all in mass basis. This was converted to volume basis by the following equation.

## • Drying of Soil

Increases in salt concentration in the moulding fluid require the higher temperature and time for drying the sample. Ho (1985) found that 3-5 days of oven drying ( $204^{\circ}$ C) were

required for Regina clay samples before a constant dry mass was reached in place of 24 hours at  $105^{\circ}$ C (AS 1289.B1.1). He speculated that this delay was due to the presence of salts forming water of crystallisation, that is water trapped within the salt crystals. The delay could also be caused by the depression of the vapour pressure of water over top of the concentrated salt solutions, thus decreasing the rate of evaporation (Barbour &Yang, 1993). Taking this aspect into account, a higher drying time was used for the determination of moisture content at high salt content.

#### Electrolyte Concentration

As discussed previously, the inter-particle force plays a significant role in changing the Atterberg limits. Increases in electrolyte concentration reduce the repulsive forces and thereby reduce the liquid limit (Ridley et al., 1984). Rosenqvist (1955) found that the salt concentration in the pore fluid makes a significant contribution to the liquid limit of soil. Results presented in Figure 5.4 show the influence of NaCl on the liquid limit of natural clay. It can be seen that the liquid limit increases sharply at low concentrations and the slope drops down with the increase of salt content. Ho and Pufahl (1987) found a decrease in the liquid limit and an increase in the plastic limit thereby decreasing the plasticity index of soil with the increase in brine concentration. This in fact changed the soil into a less plastic type of material. Ridley et al. (1984) reported decreases in the liquid limit for montmorillonite dominated clays after brine contamination. They also stated that the plastic limit appears to be unaffected by brine soaking. Chassefiere & Monaco (1983) observed that the liquid limit and the plasticity index of smectite-illite marine soils dropped gradually as brine concentration increased. It appears that influence of exchangeable cations and electrolyte concentration affects interparticle forces. For montmorillonite clay, the dominant interparticle force is repulsion, which in turn determines the distance between particles (Rosenqvist, 1962). Therefore, increased electrolyte concentration reduces the repulsive forces and thereby decreases the liquid limit.



Figure 5.4: Effect of NaCl on Liquid Limit of Quick Clay (source: Rosenqvist, 1955)

## 5.3 EXPERIMENTAL PROGRAM

Soil samples collected from Sunshine, Werribee, Echuca and Melton sites were used for the determination of the Atterberg limits with different moulding fluid. The processing of collected soil samples and the characterisation and mineralogical analyses were discussed in Chapter 3.

As discussed previously, the testing involved the use of a number of chemicals including methanol, acetic acid, sodium chloride (NaCl) and landfill leachate from a local landfill for prescribed waste. These chemicals were chosen to represent a range of chemicals and leachates to which the liner soils are likely to be exposed. Since leachates from waste containment facilities can vary significantly due to waste type as well as the age of the facility, chemicals were used at a range of concentrations. Another reason for this approach is to examine how the effects vary with the variation of concentration. Table 5.1 represents some of the physico-chemical properties of these chemicals.

Chemicals	Dielectric constant	Specific Gravity
Acetic acid	6.15	1.049
Methanol	33.6	0.791
NaCl solution	~	1.14
(4M)		

**Table 5.1**: Physico-Chemical Properties of Some Chemicals Used in Testing

#### 5.3.1 Atterberg Limit Tests

For the Atterberg limit tests, dry soil samples were mixed with the moulding liquid and were kept in a constant temperature room for at least twelve hours before the tests were carried out. This was considered necessary to allow moisture equilibrium to occur. Tests for the liquid limits were carried out in the laboratory in accordance with Australian Standard AS 1289.3.9 (1991) and British Standard BS1377: 1975 (cone penetrometer) and the plastic limits by Australian Standard AS 1289.C2.1 (1977). All tests involving the use of methanol, acetic acid and landfill leachate were carried out within a fume cupboard to minimise any health risks from these chemicals. With 100% acetic acid and methanol, the soils became almost non-plastic which makes moulding and the determination of the plastic limit extremely difficult.

## 5.4 RESULTS AND DISCUSSION

Results of the tests carried out for the Atterberg limits are presented in Tables 5.2 and 5.3. As stated earlier, the results are presented in both weight basis (Table 5.2) and in volume basis (Table 5.3). The results indicate that all the chemicals and leachate give rise to a reduction of the plasticity index of the soil. This reduction in plasticity has

occurred due to the reduction in the liquid limit and increase in the plastic limit. Out of these two limits, the decrease in the liquid limit was found to be significant while only a minimal increase in the plastic limit was observed. This is in agreement with the published results from the available literature.

As discussed previously, Atterberg limits are the function of the clay mineralogy and clay content of the soil. As can be seen from Table 3.1, the Melton soil has the highest clay content followed by Werribee, Echuca and Sunshine. The results obtained for Atterberg limits are also consistent with the previous discussion that soils having higher clay content have higher Atterberg limits. Acetic acid, methanol and NaCl solution appear to have changed the Atterberg limits of soils from Melton, Werribee, Echuca and Sunshine from the highest to the lowest respectively. This appears to be due to the fact that Melton and Werribee soil contains the highest smectite percentage and considerably high clay content whereas Sunshine soil contains the lowest clay content. It is also interesting to note that for landfill leachate, the order of the effect from the highest to a combination of chemicals present in the leachate.

From these test results, it appears that plasticity has continually decreased as the concentrations of acetic acid, methanol and NaCl have increased. Although this effect is slightly more pronounced for acetic acid than methanol when the result is considered on the mass basis, one would have expected more significant reduction considering the substantial difference in the dielectric constant of the two chemicals. One explanation for this is based on the fact that the specific gravity of methanol is lower than that of acetic acid. If the results are considered on the volume basis, the reduction in plasticity index is even more in case of acetic acid. The liquid and the plastic limits are defined with respect to the weights of liquid and solids. Consequently, different limit values would result even if the same volumes of these chemicals were used. Storey and Pierce (1989) argued that it is more relevant to compare the effects after correcting for the differences in specific gravity of the chemicals. Table 5.1 represents the values of specific gravity at 100% concentrations for methanol and acetic acid. For simplicity of

calculation, specific gravity of these two chemicals was considered linearly varying with concentration. The results modified in this fashion are presented in Table 5.3.

If a comparison is made between the results in the Tables 5.2 and 5.3, it is clear that the reduction of the plasticity index is more on the basis of volume than on the basis of mass when the permeating fluid is acetic acid. In the case of methanol, this reduction is less on volume basis. With the increase in salt concentrations, the plasticity index is less with respect to volume as compared to mass. From the above discussions, it can be concluded that the variation in plasticity index on the basis of volume is dependent on the specific gravity of the liquids at different concentrations. If the specific gravity is more than water, the plasticity index on the basis of volume is less than the corresponding values on the basis of mass and if the specific gravity is less, the plasticity index would be higher.

Liquid	Conc.	Sunshine	Melton	Werribee	Echuca
	% by	Soil	Soil	Soil	Soil
	Vol.	LL	LL	LL	LL
		PL	PL	PL	PL
<u>_</u>		PI	PI	PI	PI
Water	-	36.0	83.0	74.0	63.5
		15.0	34.0	32.0	25.5
		21.0	49.0	42.0	38.0
Acetic	30	35.0	64.0	62.0	62.5
acid		16.0	36.0	33.5	25.5
		19.0	28.0	28.5	37.0
	60	35.5	61.5	54.0	60.5
		19.0	37.0	35.0	26.5
		16.5	24.5	19.0	34.0
	80	34.5	58.0	52.5	55.5
		19.5	40.5	38.0	32.0
		15.0	17.5	14.5	23.5
	100	34.5	55.0	50.5	46.5
		NP	NP	NP	NP
Methanol	30	34.0	69.0	68.5	59.0
		16.0	34.0	35.5	25.5
		18.0	35.0	33.0	33.5
	60	33.0	63.0	56.0	59.5
		16.5	35.5	34.5	25.5
		16.5	27.5	21.5	34.0
	80	32.0	53.5	50.0	51.5
		19.0	35.0	34.5	26.0
		13.0	18.5	15.5	25.5
	100	30.5	52.0	48.0	39.5
		NP	NP	NP	NP
Leachate	-	34.5	76.5	72.0	NA
		18.5	35.5	37.0	
		16.0	41.0	35.0	
Salt	1M	34.0	66.0	63.0	56.0
solution	NaCl	16.5	31.5	33.0	24.5
		17.5	34.5	30.0	31.5
	2M	31.0	60.5	58.0	51.5
	NaCl	17.0	34.5	33.0	25.5
		14.0	26.0	25.0	26.0
	4M	29.5	52.5	51.5	46.0
	NaCl	17.5	34.5	33.5	26.0
		12.0	18.0	18.0	20.0

 Table 5.2: Test Results for Atterberg Limits (mass basis)

\* NP - non plastic, NA - not available

Liquid Conc. Sunshin		Sunshine	Melton	Werribee	Echuca
	% by	Soil	Soil	Soil	Soil
	Vol.	LL	LL	LL	LL
		PL	PL	PL	PL
		PI	PI	PI	PI
Water	-	36.0	83.0	74.0	63.5
		15.0	34.0	32.0	25.5
		21.0	49.0	42.0	38.0
Acetic	30	34.5	63.0	61.0	61.5
acid		16.0	35.5	33.0	25.5
		18.5	27.5	28.0	36.0
	60	34.5	59.5	52.5	58.5
		18.5	36.0	34.0	25.5
		16.0	23.5	18.5	33.0
	80	33.0	56.0	50.5	53.5
		18.5	39.0	36.5	30.5
		14.5	17.0	14.0	23.0
	100	36.5	58.0	53.0	48.5
		NP	NP	NP	NP
Methanol	30	35.5	72.5	71.5	62.0
		16.5	36.0	37.0	27.5
		19.0	36.5	34.5	34.5
	60	36.5	69.5	62.0	66.0
		18.5	39.0	38.0	28.0
		18.0	30.5	24.0	38.0
	80	37.5	63.0	59.0	60.5
		22.5	41.0	40.5	31.0
		15.0	22.0	18.5	29.5
	100	38.5	66.5	61.0	50.0
		NP	NP	NP	NP
Salt	1M	33.0	64.0	60.5	54.0
solution	NaCl	15.5	30.0	31.5	24.0
		17.5	33.5	29.0	30.0
	2M	29.0	56.0	54.0	47.5
	NaCl	15.5	32.0	30.5	23.5
		13.5	24.0	23.5	24.0
	4M	26.0	46.0	45.0	40.5
	NaCl	15.0	30.0	29.5	23.0
		11.0	16.0	15.5	17.5

**Table 5.3:** Test Results for Atterberg Limits (volume basis)

• NP - non plastic

## 5.5 CONCLUDING REMARKS

Atterberg limits with a number of Victorian soils mixed with chemicals and leachates were determined to examine whether these limits could be used for the assessment of the compatibility of soils for liner construction. In agreement with the published data, the test results generally indicate that the soil loses the plasticity by decreasing the liquid limit and increasing the plastic limit when mixed with acetic acid, methanol, landfill leachate and NaCl solution. This effect is most prominent for soils containing higher clay content and smectite group of minerals.

Furthermore, the plasticity continually dropped with the increase in concentration of the chemicals, and this was most pronounced for liquids having a low dielectric constant. Generally, it can be concluded that the effect on Atterberg limits by various chemicals is a function of the mineralogical composition and the clay content of soils, and physico-chemical properties of the moulding chemical. Experimental evidence indicates that a reasonable correlation exists between Atterberg limits and hydraulic conductivity of compacted clays. In this regard, these limits appear to be useful for initial compatibility assessments of soils for liner construction. However, as far as the soils used in this research program are concerned, this approach may still need to be verified by conductivity testing carried out with NaCl, and leachate and methanol are discussed in chapters 7 and 8 respectively.

# 6 EFFECTS OF SPECIMEN CONSOLIDATION ON THE LABORATORY HYDRAULIC CONDUCTIVITY MEASUREMENT

## 6.1 INTRODUCTION

As discussed in Chapter 4, the termination criteria for testing for chemical compatibility requires at least 2 pore volumes of permeant to pass through the specimen before the test is completed. This requirement is important to provide sufficient resident time for any clay chemical interaction to take place (Mitchell & Madsen, 1987). In a controlled liner situation, hydraulic gradients usually encountered in the field are relatively small and are unlikely to exceed 10 to 20 in most cases. But in laboratory testing for clayey soils of low hydraulic conductivity, use of such low hydraulic gradient is generally impractical because of the excessive time involved in compatibility testing. Hence, it is common to employ an elevated hydraulic gradient (*eg.* 100 to 300) so that the test can be completed at a reduced time. Nevertheless, it is commonly believed that elevated hydraulic gradient can adversely impact on the hydraulic conductivity measured. The impact of the elevated hydraulic gradient can come in two main ways: due to specimen consolidation; and due to particle migration and piping failure. The current chapter addresses these issues, mainly the impact of specimen consolidation.

Fox (1996) presented an analysis of specimen consolidation for normally consolidated soft soils and slurries tested in rigid wall permeameters (RWPs). An extension of this analysis for overconsolidated soils including compacted soils tested in both flexible wall permeameters (FWPs) and RWPs is presented in this chapter. A comparison was made

between the theoretical and experimental results for two local soils used in testing. Different options of gradient application were studied for hydraulic conductivity testing in the FWPs. The theoretical model was used to perform parametric studies to identify the influencing variables. This chapter presents the results of these analyses.

# 6.2 SYNTHESIS OF PAST RESEARCH

Dunn and Mitchell (1984) stated that hydraulic gradients encountered in the field are generally small (around 1), and are unlikely to exceed 10 or 20 in most cases. According to US EPA regulations, the height of the leachate level (above the basal liner) in the landfill must not exceed 300 mm. Therefore, for a 1m thick liner hydraulic gradient is around 1. ASTM D5084 (1990) recommends a maximum hydraulic gradient of 30 for clay soil. However, as far as compatibility testing is considered where it is expected to pass at least 2 pore volumes of permeant, it is generally difficult to use those low gradients in laboratory testing owing to excessive testing times (*ie.* several weeks or months) involved with low conductivity soils. Figure 6.1 provides a chart for calculating the testing times to pass 2 pore volumes of permeant for different hydraulic gradient values. The calculation was based on a specimen thickness of 50 mm and a porosity value of 0.4. It should be noted that, in compatibility testing, a total of 3 pore volumes may be involved as discussed in Chapter 4; it is common practice to pass about 1 pore volume of water through the specimen before switching to the chemical permeant. Under those circumstances, the overall testing time will be even higher.

High hydraulic gradient can increase and/or decrease the effective stress within the testing specimen depending on the method of gradient application. High effective stress may alter the pore size distribution and the characteristics of the diffuse double layer





around clay particles and subsequently the hydraulic conductivity. Acar *et al.* (1985) found that some specimen volume changes occurred during the permeation with organic fluids. The volume change might have been initiated from structural changes caused by the changes in surface forces of interaction. The changes in the surface forces may have resulted from the variations in the attractive and repulsive forces between clay minerals induced by the permeation of organic fluids. The uses of high effective stresses much larger than in-situ conditions may restrict such possible volume changes and mask any changes in hydraulic conductivity. Foreman and Daniel (1986) performed tests on the consolidation cell permeameter with methanol as permeant and found three order magnitude lower values of hydraulic conductivity when effective stress was increased from 20 kPa to 100 kPa. For water, this change was not significant. They attributed this difference to the specimen consolidation that limited the clay structural changes due to methanol.

It is evident from the available literature that the gradient effect on hydraulic conductivity varies for different types of permeameters (eg. Foreman & Daniel, 1984; Acar et al., 1985). In the case of FWP, it is understood that the elevated hydraulic gradients can affect the conductivity testing primarily in two ways, namely, by the change of specimen volume or by particle migration. Due to the application of a hydraulic gradient, the effective stresses within the specimen will change from the initial value. Depending on the method of hydraulic gradient application, higher effective stresses generated (either at the effluent or influent end), may lead to consolidation of the specimen during permeation. At the opposite end of the specimen, however, a reduction in effective stresses normally occurs, which may give rise to specimen swelling. Hence, the overall effect of the hydraulic gradient on conductivity owing to the specimen volume change is a combination of these factors. Nevertheless, researchers have generally observed that the conductivity will decrease as the hydraulic gradient is increased (eg. Carpenter and Stephenson, 1986). For example, Edil & Erickson (1985) found that the conductivity measured in FWPs decreased as much as

58% when the hydraulic gradient was increased from 29 to 290. Similarly, Acar *et al.* (1985) and Foreman and Daniel (1984) reported decreases in the higher hydraulic conductivity as the gradient was increased. In these studies, the specimen consolidation was considered to be the primary cause for this change. Furthermore, Mitchell and Younger (1967) and Gairon and Schwartzendruber (1975) reported that the hydraulic conductivity decreased with increasing hydraulic gradient in their studies, but they suspected that this change may be due to particle migration that leads to clogging of the pore spaces at the effluent end.

In the case of RWP, a general trend of increase in hydraulic conductivity with the increase of hydraulic gradient was observed by a number of investigators. In contrast to FWPs, the side wall leakage was considered to be the major factor that led to large increases in conductivity in RWPs (*eg.* Foreman & Daniel, 1984; Acar *et al.*, 1985). The increase was higher in the case of chemicals causing shrinkage in soil specimens than due to water. Once a shrinkage crack is initiated at the wall due to chemical permeation, high gradient can cause this crack to penetrate the full depth of the specimen causing massive increases in hydraulic conductivity.

High hydraulic gradients can result in migration of fine particles which also can impact on the conductivity in two ways. If the migrating particles exit from the effluent end, then piping channels can develop within the specimen increasing the conductivity. On the other hand, if the particles get deposited or plugged at the effluent end blocking the flow paths, then even a reduction in conductivity could occur. The gradient which would initiate particle migration is a function of the gradation of the soil particles, soil cohesion and soil and permeant chemistry.

The particle migration can also lead to changes in the hydraulic conductivity of compacted clay. Owing to the high seepage velocities resulting from elevated hydraulic gradients, the particles can start to move with water. These particles can then either clog

the pores at the effluent end leading to plugging or exit at the effluent end leading to piping. The plugging may occur at low hydraulic conductivities due to high particle concentration in the seepage fluid, while piping may occur at high hydraulic conductivities due to low particle concentration. Both plugging and piping depend strongly upon the size distribution of the migrating particles relative to pore size distribution of the soil. Khilar et al. (1985) highlighted that the initial permeability and porosity of the soil, the critical shear stress  $au_c$  (ie. stress at zero sediment discharge) and the hydraulic gradient are the influencing variables for piping in earthen structures. At one extreme, very large particle movement will lead to plugging whereas at the other extreme, very small particles will wash through. In the intermediate size range the outcome also depends strongly on the concentration of dispersed particles in the seepage stream, which, in turn, depends upon the rate at which the particles erode from the pore walls. The minimum hydraulic gradient required to cause piping is very high (>200) for soils reported to be non-susceptible to piping (ie. for  $\tau_c > 10$  dynes/cm<sup>2</sup>). Large particle concentration build-ups tend to occur at hydraulic conductivities less than  $10^{-6}$  m/s for water at 20<sup>°</sup>C. On the basis of this evidence, it appears that, for compacted clay, the particle plugging is a more common mechanism than the piping failure.

# 6.3 METHODS OF APPLICATION OF HYDRAULIC GRADIENT

During the hydraulic conductivity measurement, elevating the influent pressure after the saturation of the specimen induces the flow through the specimen. Usually, the permeation of fluid through the specimen in RWPs is from top to bottom whereas in FWPs the flow is from bottom to top. Figure 6.2 schematically shows the variation of effective stress and pore water pressure for different options of gradient application commonly used. As discussed below, three different options of gradient applications





(Options 1 to Options 3) are possible for FWPs. Option 4 is usually applicable for RWPs.

**Option 1:** In this option, the hydraulic gradient is applied by increasing the influent pore pressure and decreasing the effluent pore pressure by equal amounts. In this case effective stress at the effluent end is increased and at the influent side effective stress is decreased by the same amount. Changes in effective stresses could lead to swelling and consolidation of influent and effluent ends of the specimen respectively. The researchers who followed this option include Daniel *et al.* (1984), Foreman & Daniel (1984), Acar *et al.* (1985) and Edil & Erickson (1985).

**Option 2:** Hydraulic gradient is applied by decreasing the effluent pore pressure and keeping the influent pore pressure constant. In this case, the effective stress at the effluent side is increased resulting in possible consolidation of the specimen while the influent effective stress remains constant. The main disadvantage of this method is that it needs higher effective stress compared to Option 1 for applying the same gradient. This method was reported to be used by Evans & Fang (1988), Uppot & Stephenson (1989), Yanful *et al.* (1990), Edil *et al.* (1991), Lentz *et al.* (1985), and Carpenter & Stephenson (1986).

**Option 3:** The required hydraulic gradient is applied by increasing the influent pore pressure and keeping the effluent pore pressure constant. In this option, the effective stress at the influent end is reduced with a consequent swelling of the specimen and the effluent effective stress remains constant. Similar to Option 2, this method also has the same drawback. ASTM D5084-90 recommends this as the preferred option to be used. Other reported usage can be found in publications by Lentz *et al.* (1985), Carpenter & Stephenson (1986), Evans & Fang (1988), Uppot & Stephenson (1989), Yanful *et al.* (1990), and Edil *et al.* (1991).

**Option 4:** The general way of applying hydraulic gradient in RWPs is by increasing the pore pressure at the top while keeping bottom pressure constant. In this situation, effective stress at the top of the specimen is zero whereas the highest effective stress occurs at the bottom of the specimen. The problem of having zero effective stress at the top of the specimen swelling and potential for large increases in sidewall leakage. This may be minimised by applying a surcharge load simulating the overburden pressure of the specimen prior to conducting the test (Fox and Boxter, 1997).

# 6.4 **THEORETICAL ANALYSIS**

#### 6.4.1 Analysis for RWPs





The analysis is based on the RWP concept as depicted in Figure 6.3, but an extension to FWPs will be presented in the following section. A saturated soil specimen of length *L* is contained in a RWP where pore pressures at the top and the bottom of the specimen are given by  $u_t$  and  $u_b$  respectively. Usually for RWPs,  $u_t$  is greater than  $u_b$  so that the permeation can be effected from top to bottom. The total stresses at the top and the bottom frictionless conditions at the rigid wall, the vertical total stress within the specimen can be written as:

$$\sigma_{v} = \sigma_{vt} + \int_{z}^{L} \gamma_{sat} dz$$
(6.1)

where  $\gamma_{sut}$  is the saturated unit weight of the soil. Using z = 0 as the datum for the elevation head (*h*), the pore water pressure *u* at elevation *z* can be written as  $u = \gamma_w (h-z)$ . Then, the vertical effective stress  $\sigma'_v$  at height *z* is:

$$\sigma'_{\nu} = \sigma_{\nu t} - \gamma_{w}(h-z) + \gamma_{w} \int_{z}^{L} \frac{G_{x} + e}{1+e} dz$$
(6.2)

where  $G_s$  is the specific gravity of the soil particles, and e is the void ratio. Using Darcy's law, the local hydraulic gradient *i* can be written as:

$$i = \frac{dh}{dz} = -\frac{v}{k_v} \tag{6.3}$$

where v is the Darcy velocity (upward positive) and  $k_v$  is the vertical hydraulic conductivity at height z.

The volume change characteristic of the specimen is represented by a usual *e-log*  $\sigma'_{\nu}$  curve as shown in Figure 6.4 (a). Prior to placing the specimen into the permeameter, it is assumed that the soil features a preconsolidation pressure of as compacted specimen of  $\sigma'_{c}$ . Before the application of the hydraulic gradient, the soil is assumed to be subjected to

a uniform vertical effective stress of  $\sigma'_{vo}$  at an average void ratio of  $e_o$ . The variation of vertical hydraulic conductivity with void ratio is presented in *e-log*  $k_v$  axes as shown in Figure 6.4(b). It is assumed that the gradient of this line  $C_k$  is the same for both normally consolidated and overconsolidated soil behaviour (Nagaraj *et al.*, 1994). To cater for the general form of application of hydraulic gradient, it is also assumed that  $u_t$  is increased and  $u_b$  is decreased from the initial pore pressure  $u_o$ , during the application of the hydraulic gradient.



**Figure 6.4:** Assumed Soil Behaviour: (a)  $e - log(\sigma'_v)$ ; (b)  $e - log(k_v)$ 

On the basis of these assumptions, it is possible to identify three possible conditions under which the soil specimen can undergo volume change:

Condition (a) when 
$$\sigma'_{\nu q} < \sigma'_{\nu q} \le \sigma'_{c}$$

Under this condition, the soil specimen will behave entirely in the overconsolidated region. The governing equations are given by:

$$e = e_0 - C_r \log\left(\frac{\sigma'_v}{\sigma'_{vo}}\right)$$

$$e = e_0 + C_k \log\left(\frac{k_v}{k_{vo}}\right)$$
(6.4)
(6.5)

Equations (6.4) and (6.5) give:

$$k_{\nu} = k_{\nu o} \left( \frac{\sigma_{\nu}'}{\sigma_{\nu o}'} \right)^{-C_r/C_k}$$
(6.6)

Substituting from Equations (6.2) and (6.6) into Equation (6.3), and, for simplicity, ignoring the buoyant weight of the soil specimen for compacted clay (*ie.*  $G_s=1$ ), it can be shown:

$$\frac{dh}{dz} = -\frac{v}{k_{vo}} \left[ \frac{\sigma_{vi} + \gamma_w (L-h)}{\sigma'_{vo}} \right]^{C_r/C_k}$$
(6.7)

The following boundary conditions are applicable to the problem:

At, 
$$z = L$$
,  $h = L + \frac{u_t}{\gamma_w}$  (6.8)

At, 
$$z = 0$$
,  $h = \frac{u_b}{\gamma_w}$  (6.9)

The solution of Equation (6.7) subject to these boundary conditions gives:

$$v = -\frac{k_{vo}(\sigma'_{vo})^{C_r/C_k}}{L\gamma_w(1 - C_r/C_k)} \left[ (\sigma_{vl} + L\gamma_w - u_b)^{1 - C_r/C_k} - (\sigma_{vl} - u_l)^{1 - C_r/C_k} \right]$$
(6.10)

If  $k_{veq}$  denotes the measured value of hydraulic conductivity under the applied external hydraulic gradient  $i_{eq}$ , which is given by  $[(u_t - u_b) / L\gamma_w] + 1$ , Darcy's law can be used to obtain the following expression for  $k_{veq}$  as:

$$k_{veq} = \frac{k_{vo}(\sigma'_{vo})^{C_r/C_k}}{(u_r - u_b + L\gamma_w)(1 - C_r/C_k)} \left[ (\sigma_{vt} + L\gamma_w - u_b)^{1 - C_r/C_k} - (\sigma_{vt} - u_t)^{1 - C_r/C_k} \right]$$
(6.11)

In order to simplify Equation (6.11) so that a direct conversion to the FWP situation can be made, the term  $L\gamma_w$  is ignored in comparison to the external pressure difference  $u_t - u_b$ . Furthermore, since we are dealing with stiffer compacted soils, the influence from a change in length L is also ignored. This will give:

$$k_{veq} = \frac{k_{vo}(\sigma'_{vo})^{C_r/C_k}}{(u_r - u_h)(1 - C_r/C_k)} \left[ (\sigma_{vl} - u_h)^{1 - C_r/C_k} - (\sigma_{vl} - u_l)^{1 - C_r/C_k} \right]$$
(6.12)

<u>Condition (b)</u> when  $\sigma'_{\nu b} > \sigma'_{c} > \sigma'_{\nu o}$ 

Under this condition, the effective stresses on the bottom section of the specimen will exceed  $\sigma'_c$ . Considering that this occurs at  $z = L^*$ , the volume change equations applicable to the specimen are can be written as:

$$L^* \le z \le L \qquad e = e_o - C_r \log\left(\frac{\sigma'_v}{\sigma'_{vo}}\right)$$
(6.13)

and

$$0 \le z \le L^* \quad e = e_o - C_r \log\left(\frac{\sigma'_c}{\sigma'_{vo}}\right) - C_c \log\left(\frac{\sigma'_v}{\sigma'_c}\right)$$
(6.14)  
At,  $z = L^*$ ,  $\sigma'_v = \sigma'_c$ 
(6.15)

In addition, Equation (6.5) will apply for the entire specimen. As under Condition (a), Equations (6.13), (6.14) and (6.5) can be combined, and solved subject to the boundary conditions given by Equations (6.8) and (6.9) and the internal condition given by Equation (6.15). This will give:

$$k_{veq} = \frac{k_{v0}(\sigma_{c}^{\prime})^{(C_{c}-C_{c})/C_{k}}(\sigma_{v0}^{\prime})^{C_{c}/C_{k}}}{(u_{t}-u_{b})(1-C_{c}/C_{k})} [(\sigma_{c}^{\prime})^{1-C_{c}/C_{k}} - (\sigma_{vt}-u_{t})^{1-C_{c}/C_{k}}] - \frac{k_{v0}(\sigma_{v0}^{\prime})^{C_{c}/C_{k}}}{(u_{t}-u_{b})(1-C_{c}/C_{k})} [(\sigma_{c}^{\prime})^{1-C_{c}/C_{k}} - (\sigma_{vt}-u_{b})^{1-C_{c}/C_{k}}]$$
(6.16)

# <u>Condition (c)</u> when $\sigma'_{vo} \ge \sigma'_{c}$

Under this condition, the specimen is consolidated above the pre-consolidation pressure  $\sigma'_c$  prior to permeation. Therefore, when the hydraulic gradient is applied, the bottom section of the specimen will consolidate normally, and the top section will undergo some swelling if a reduction in effective stress takes place. As under Condition (b), the applicable volume change equations are:

$$0 \le z \le L^* \qquad e = e_o - C_c \log\left(\frac{\sigma'_v}{\sigma'_{v_o}}\right)$$
and
$$(6.17)$$

$$L^{*} \leq z \leq L \qquad e = e_{o} - C_{r} \log\left(\frac{\sigma_{v}}{\sigma_{vo}}\right)$$

$$At, z = L^{*}, \quad \sigma_{v}' = \sigma_{vo}'$$

$$(6.18)$$

$$(6.19)$$

Equations 
$$(6.17)$$
,  $(6.18)$  and  $(6.5)$  can be combined, and solved subject to the boundary conditions given by Equations  $(6.8)$  and  $(6.9)$  and the internal condition given by

Equation (6.19). This will give:

$$k_{veq} = \frac{k_{v0} (\sigma'_{v0})^{C_c/C_k}}{(u_t - u_b)(1 - C_c/C_k)} [(\sigma'_{v0})^{1 - C_c/C_k} - (\sigma_{vt} - u_t)^{1 - C_c/C_k}] - \frac{k_{v0} (\sigma'_{v0})^{C_r/C_k}}{(u_t - u_b)(1 - C_r/C_k)} [(\sigma'_{v0})^{1 - C_r/C_k} - (\sigma_{vt} - u_b)^{1 - C_r/C_k}]$$
(6.20)

Therefore, Equations (6.12), (6.16) and (6.20) represent the externally measurable hydraulic conductivity which includes the effects of volume change within the specimen. It should be noted that these equations are undefined when  $C_c/C_k = 1$  or  $C_r/C_k = 1$ . Equations specifically catering for these conditions can easily be derived (see Fox, 1996). Furthermore, since the  $L\gamma_w$  term is ignored, the equations are not valid when  $(u_t/u_b) = 1$ . Nevertheless, the above equations can still be used for these conditions without introducing significant error by using values slightly different from unity.

## 6.4.2 Application to FWP

In the FWP, the vertical and lateral stresses are the same, hence, the vertical stresses must be replaced by mean pressures. Since the consolidation in the RWPs occurs under  $K_o$ conditions, the following relationship can be developed to approximate the effective mean pressure p' (Atkinson, 1993):

$$p' = \frac{1}{3}\sigma'_{v}(1+2K_{a})$$
(6.21)

where  $K_o$  is the lateral earth pressure coefficient. This depends on the overconsolidation ratio. The parameters  $C_c$ ,  $C_r$  and  $C_k$  remain unchanged in both stress systems.

In FWPs, hydraulic conductivity tests are normally conducted from bottom to top (*ie*.  $u_b > u_t$ ). In this situation, the effective stresses in the top section of the specimen will be higher than those of the bottom section. As for RWPs, equations can be re-derived to specifically cater for this situation. However, once the term  $L\gamma_w$  is ignored, the equations for  $k_{veq}$  applicable to FWPs are identical to Equations (6.12), (6.16) and (6.20) provided  $u_t$  and  $u_b$  are interchanged.

# 6.5 THEORETICAL AND EXPERIMENTAL RESULTS

In order to appreciate the applicability of the above analysis, theoretical results are compared with some experimental results obtained for soils from Sunshine and Werribee landfilling sites. On the basis of the Equations (6.12), (6.16) and (6.20), a FORTRAN computer program was developed to calculate  $(k_{veq}/k_{v0})$  vs *i* along with the state of the soil considering different options as discussed in the previous section. Laboratory testing for hydraulic and consolidation properties, required for the calculation and comparison, was carried out. The details of the testing technique and the results are presented here.

#### 6.5.1 Experimental Procedure and Results:

Soil specimens using Sunshine and Werribee soils were tested in the laboratory to investigate the effect of hydraulic gradient on hydraulic conductivity measurement by using FWPs. The same procedure discussed in Chapter 4 was adopted to carry out for specimen preparation, setting up the specimen and specimen saturation. After the required saturation level was obtained the permeation was carried out at hydraulic gradients of 28, 100, 200 and 300. The cell pressure was set to 540 kPa and the effective stress at 100 kPa. The required hydraulic gradient was applied by increasing the bottom pressure and decreasing the top pressure by equal amounts according to Option 1 in Section 6.2 of this chapter. Accuracy of the test results was verified by carrying out repeatable testing under identical conditions, using water and methanol as permeants. For water the results are presented in Appendix A5.1 and for methanol the results will be discussed in Chapter 8. As can be seen from A5.1 that the variation in hydraulic conductivity within both soils is very negligible.

The relationship between the void ratio and hydraulic conductivity was established from laboratory FWP testing. The same procedure was followed for the preparation, placement and saturation of the specimen. The specimen was then allowed to consolidate initially at low effective stress. After consolidation the permeation was started and allowed to establish a stable hydraulic conductivity. The effective stress was then increased and another few sets of testing were carried out. The results of the investigation are plotted in Figures 6.5(a) and 6.5(b), and the parametric value  $C_k$ , slope of void ratio versus logarithm of hydraulic conductivity, was calculated. The results of the investigation are tabulated in Table 6.1. The parameter  $C_c$  was also calculated from the above test results and the results are presented in Table 6.1. Traditional tests for parameters  $C_c$ ,  $C_r$  and  $\sigma'_c$  were carried out in laboratory oedometer consolidation apparatus and the results are presented in Figures 6.6(a) and 6.6(b) for Sunshine and Werribee Soils respectively. The same moisture content and bulk density, as noted in Table 6.1, was maintained for the specimen used in consolidation testing. Compaction technique, as was disused in Chapter 4 for the preparation of specimen for FWP, was employed for compacting the soil specimen in the consolidometer ring. Table 6.1 and Table 6.2 presents the results of this investigation.

	W <sub>opt</sub> (%)	w (%) (as compacted)	γ (as compacted, kN/m <sup>3</sup> )	C <sub>r</sub>	C <sub>c</sub> Oedometer (FWP)	C <sub>k</sub>	$\overline{\sigma'_c}$ ( $p'_c$ ) kPa
Sunshine	14.5	17.0	20.58	0.017	0.11	0.1	68
soil					(0.12)		(45.3)
Werribee	31.0	34.5	18.13	0.03	0.38	0.16	98
soil					(0.47)		(65)

Table 6.1: Basic Properties of Compacted Soil

 $\Psi p'_c$  is computed from  $\sigma'_c$  using Equation (6.21) and  $\phi' = 30^\circ$ .

As can be seen from Table 6.2 the hydraulic conductivity does not significantly vary up to hydraulic gradient of 300. For both types of soils, a relatively small decrease in hydraulic conductivity was observed with the increase of hydraulic gradient. From the above test results, we can conclude that the hydraulic conductivity decreases with the increase of hydraulic gradient although the variation is not significant at least up to hydraulic gradient of 300. This result is in agreement with the published data that compacted clays generally show a decrease in hydraulic conductivity with increasing hydraulic gradient, although a significant scatter of the test results was usually evident (*eg.* Dunn and Mitchell, 1984; Kallur *et al.*, 1995). From Table 6.2 it can be seen that maximum reduction for Sunshine soil and Werribee soil is 32% (reduction factor 0.67) and 21% (reduction factor 0.88) respectively when hydraulic gradient was increased from 28 to 300.

The reason for the difference in the reduction in hydraulic conductivity may be attributed to the variation in the soil particle size distribution. From Chapter 3, it can be seen that soil from Sunshine possesses a higher percentage of sand and silt particles than in the Werribee soil. The presence of higher percentages of sand and silt particles in Sunshine soil may give more space for the fine particles to migrate and at the same time, less plugging may occur for the same reason. Dunn and Mitchell (1984) found that for clay





Figure 6.5:  $e - \log k$  ( $C_k$ ) graph for: (a) Sunshine Soil; and (b) Werribee Soil



Figure 6.6(a): e - Log P Relationship for Sunshine Soil



Figure 6.6(b): e - Log P Relationship for Werribee Soil

specimens compacted to 90% Proctor maximum dry density, the hydraulic conductivity was reduced by a factor of 2.5 (or a reduction factor of 0.4) when the gradient was increased from 20 to 200 which is much higher than the percentage reductions tabulated in Table 6.2. A possible reason may be for lightly compacted clays, particle migration and the chance of clogging as well as the gradient induced consolidation is higher than for more compacted (100 % Proctor compacted) clays. These mechanisms might have contributed to the reduction of hydraulic conductivity. For denser soil, a greater percentage of particles incorporated in the load carrying framework and fewer particles would be available for migration and less pore space available for particles to migrate.

	Sunshine soil		Werribee soil		
Hydraulic	$(k_{veq}) \ge (10^{11})$	Percentage	$(k_{veq}) \propto (10^{11})$	Percentage	
gradient, i	(m/s)	reduction	(m/s)	reduction	
28	6.55	-	6.54	_	
100	5.4	17.6	6.51	0.5	
200	4.43	32.4	5.75	12.1	
300	4.64	29.2	5.16	21.1	

Table 6.2: Test Results of the Effects of Hydraulic Gradient on Hydraulic Conductivity

## 6.5.2 Theoretical Analyses

The proposed analytical model was used to examine the effects of hydraulic gradient on hydraulic conductivity using the soil properties given in Table 6.1 and the test conditions outlined above. Two sets of studies were carried out using the two sets of values of  $C_c$  for each soil obtained from oedometer and flexible wall permeameter. Theoretically and ideally, both FWP and RWP should yield same  $C_c$  and  $C_r$  values, but, in the instance, the

exact experimental values obtained were used in the analysis. As can be seen from Table 6.1,  $C_c$  (FWP) values for both the soils are higher than  $C_c$  (oedometer), because of the side wall friction in case of  $C_c$  oedometry phase. Results of the study are presented in Figures 6.7(a) and 6.7(b) using oedometer  $C_c$  for Sunshine and Werribee soils respectively. Figures 6.8(a) and 6.8(b) present the results for Sunshine and Werribee soils respectively for  $C_c$  obtained from flexible wall permeameter results. It should be noted that, for the theoretical analysis, the maximum hydraulic gradient applicable was obtained by limiting the minimum effective stress at the influent end to 10 kPa. The theoretical solutions given in Condition (a) through to Condition (c) were used to analyse the test conditions. Figures 6.7(a) and 6.7(b) also show the test results normalised with respect to  $k_{vo}$  at a unit hydraulic gradient that was obtained by scaling  $k_v$  corresponding to i=28 by the  $k_{veq}/k_{vo}$  factor determined from the theoretical model for i=28 for Sunshine soil and Werribee soil respectively. Consequently, a value of unity is obtained for  $k_{veq}/k_{vo}$ at i=28. Same approach was used for Figures 6.8(a) and 6.8(b).

From Table 6.1, it could be seen that the variation in  $C_c$  values between odeometer and FWP results are within 24%. From the analyses carried out in Section 6.5.2, it is clear that the variation in hydraulic conductivity ratio  $(k_{veg}/k_{vo})$  is not significant, which is also evident from Figures 6.7 and 6.8. It could also be seen from both the Figures 6.7 and 6.8 that the correlation between model and the test results for Werribee soil is better than Sunshine soil and the reasons for that were discussed in the previous section. The reasons could be due to the variation in the particle size, particle migration and the presence of high swelling clay in two soils. Both the theoretical model and test results show a consistent reduction of hydraulic conductivity with increasing hydraulic gradient; both results indicate that the reductions are less than a factor of 1.5 (reduction factor 0.67) in the feasible gradient range. It should be noted, however, that the current model does not take into account the effect of the particle migration.



(b) Werribee Soil

Figure 6.7: Effect of Hydraulic Gradient on Hydraulic Conductivity; Model (Oedometer Results) and Test Results



Figure 6.8: Effect of Hydraulic Gradient on Hydraulic Conductivity; Model (FWP) and Test Results.

The theoretical model was applied to simulate the likely test conditions to examine how the RWPs would have performed for these soils. Permeation in RWPs is normally carried out from top to bottom with  $u_t > u_b$ . In this case, the effective stress at the top remains at  $\sigma'_{vo}$  unless surcharge is applied and the effective stress at the bottom is increased due to the decrease in pore water pressure. Hence, it can be expected that the entire specimen will consolidate without any swelling. Two conditions of initial effective stresses, namely  $\sigma'_{vo} = 100 \text{ kPa}$  and  $\sigma'_{vo} = 1 \text{ kPa}$ , were simulated. The first condition is applicable to a consolidation cell permeameter, and the second condition is applicable to a compaction mould permeameter where no external effective stress is applied on top of the specimen. The results of these simulations for Sunshine soil are shown in Figure 6.9. For an initial effective stress of 100 kPa, the specimen behaves entirely under the Condition (c) whereas, for an initial effective stress of 1 kPa, the specimen behaves initially under the Condition (b) and then under the Condition (c). It is clear that the



**Figure 6.9:** Theoretical Evaluation of the Effects of Hydraulic Gradient on Conductivity in RWPs Under Two Effective Stress Conditions.

changes in hydraulic conductivity ratio  $(k_{veg}/k_{v0})$  are significantly higher for this form of gradient application than for the application considered with FWPs previously. The results indicate that, under hydraulic gradients as high as 1000, it is possible to get an order of magnitude reduction in hydraulic conductivity. These high hydraulic gradients can result in tests using RWPs and high constant flow rates (*eg.* Quigley and Fernandez, 1989). It is also apparent that the lower effective stress condition initially displays a more rapid rate of reduction, and a higher overall reduction in hydraulic conductivity. The Werribee Soil also displays the same initial situation but the final results are very close for both high and low effective stresses.

## 6.6 PARAMETRIC EVALUATION

Hydraulic conductivity ratio  $(k_{veq}/k_{v0})$  is dependent on how the gradient is applied for inducing the flow through the specimen. Analyses were carried out considering a wide range of geotechnical parameters for checking the suitability of different options. Sensitivity analysis of individual parameters on hydraulic conductivity was carried out to examine how each of these parameters influences the results. On the basis of these analyses, dimensionless analyses were also undertaken on the most influencing variables and the results are presented in this Section.

## 6.6.1 Effect of the Mode of Hydraulic Gradient Application

A wide range of parametric values was considered for compacted soils on the basis of experimental results and available literature for examining the suitability of different options. Theoretical model was applied for Options 1 through to Option 4, as discussed

in Section 2, for the parameters for Sunshine soil and Werribee soil as listed in Table 6.1. The results of the analyses for Sunshine soil are presented in Figure 6.10. As stated earlier, Option 4 is for RWPs where the flow is induced from top to bottom. In Option 2, top pore water pressure is reduced while the bottom pore water pressure is increased in Option 3. Increases in the bottom pore water pressure reduce the bottom effective stress. The program was made to stop if the bottom effective stress falls below 10 kPa. This is evident for Option 3 from Figure 6.10 where the program has terminated at a hydraulic gradient of 181.1. But in the case of Option 1, top pore pressure is reduced and bottom pore pressure is increased by the same amount and the program continues up to hydraulic gradient of 361.1. As was discussed before, with the reduction in effective stress swelling might have occurred with the consequent increase in hydraulic conductivity, which is also evident for Option 3 in Figure 6.10. Interchanging the values of  $u_t$  and  $u_b$ .



**Figure 6.10:** Effect of the Mode of Hydraulic Gradient Application on Hydraulic Conductivity for Sunshine Soil.
same results could be obtained for both Options 2 and 3. From Figure 6.10, it could be seen that the reduction in hydraulic conductivity ratio is less in case of Option 1 than in Option 2. Similar results were also obtained for Werribee soil.

As shown in Figure 6.2, higher effective stresses are required to apply the same levels of hydraulic gradients in Options 2 and 3 than in Option 1. High effective stresses applied are again associated with specimen consolidation and consequent decrease in hydraulic conductivity. On the basis of the above analysis, Option 1 is found to be the most suitable for FWP hydraulic conductivity testing, and is, therefore, used for the following parametric analysis.

#### 6.6.2 Effect of Individual Parameters

From Equations (6.12), (6.16) and (6.20) the hydraulic conductivity ratio could be considered as:

• 。

$$(k_{veq}/k_{v0}) = \Psi(i, C_{c}, C_{r}, C_{k}, \sigma'_{c}, \sigma_{vt})$$
(6.22)

Sensitivity analysis of these individual parameters was carried out for both normally consolidated and over consolidated soils considering Option 1. The parametric values for these variables were assumed in typical ranges that can be found in the literature. During the analysis, each of the individual parameters were increased and decreased separately by 50% from their mean values, keeping the other variables constant. Percentage changes in the values of hydraulic conductivity ratios were calculated with respect to their mean values at  $C_c$ ,  $C_r$ ,  $C_k$  at i=361.1,  $\sigma'_{v0}$  at i = 161.1, and  $\sigma'_c$  at i = 261.1. Comparisons were made at different hydraulic gradients because the program terminates at different hydraulic gradients after reaching the least effective stress of 10 kPa within the specimen. Results of these analyses are presented in Table 6.3.

From Table 6.3, it is evident that different parameters have a varying influence on the hydraulic conductivity ratio. For the normally consolidated soil, the following sequence  $C_c$ ,  $C_k$ ,  $\sigma'_{v0}$  and  $C_r$  may be considered as the order from higher to lower in which the variables influence the hydraulic conductivity ratio. The parameters  $\sigma'_c$  and  $\sigma_{vr}$  are found to be least sensitive to changes in the hydraulic conductivity ratio for normally consolidated soil. For over-consolidated soil, the sequence appears to be  $\sigma'_{v0}$ ,  $C_r$ ,  $C_c$  and  $C_k$ . The parameter  $\sigma_{vr}$  is also found to be insensitive for over-consolidated soils as well. Although the changes in hydraulic conductivity ratio due to the changes in  $\sigma'_c$  was found to be very low, still it was found that the changes are dependent on the degree of the overconsolidation ratio.

	Over consolidated soils				Normally consolidated soils				
	$(k_{veq}/k_{v0})$ (%)				$(k_{veq}/k_{v0})$ (%)				
	-50(%)	-25%	25%	50(%)	-50(%)	-25%	25%	50(%)	
C <sub>c</sub>	1.38	0.67	-0.64	-1.25	8.68	4.03	-3.51	-6.58	
$C_k$	2.08	0.5	-0.2	-0.3	-4.71	-2.4	1.93	3.44	
C <sub>r</sub>	-1.80	-0.93	1.0	2.06	-3.07	-1.56	1.63	3.32	
$\sigma'_{v0}$	1.62	0.35	-0.96	-6.34	-4.19	-1.9	1.39	2.43	
$\sigma_{c}^{\prime}$	-2.80	0	0	0	0	0	0	0	
$\sigma_{_{vt}}$	0	0	0	0	0	0	0	0	

 Table 6.3: Changes in Hydraulic Conductivity Ratio with Respect to the Changes in the

 Individual Parameter

#### 6.6.3 Non-Dimensional Analyses

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In the previous section, a dimensional analyses of  $k_{veq}/k_{v0}$  ratio was presented considering a wide range of parametric values. The relative influence of parameters affecting hydraulic conductivity was identified on the basis of this analysis. However, it is considered useful to present these influences in non-dimensional fashion. For instance, the hydraulic conductivity ratio can be given in non-dimensional form as:

$$k_{veq}/k_{v0} = \Psi(\sigma'_{c}/\sigma'_{v0}, \sigma'_{v0}/L_{0}\gamma_{w}, C_{c}/C_{r}, C_{c}/C_{k})$$
(6.23)

For non-dimensional analyses to be undertaken, the likely ranges applicable to individual non-dimensional parameters need to be identified. Experimental evidence shows that the value of  $C_c/C_r$  is about 10 and, hence, was kept constant during the analyses. Reasonable ranges for other non-dimensional quantities were considered as  $\sigma'_c/\sigma'_{v0} = 0.5$  to 1.5,  $\sigma'_{v0}/L_0\gamma_w = 50$  to 200, and  $C_c/C_k = 0.1$  to 5.0. Based on these ranges, more analyses were conducted and the results are presented in Figures 6.11 to 6.15.

Figure 6.11 presents the results for the analyses for different values of  $C_c/C_k$  where  $\sigma'_c/\sigma'_{v0} \leq 1$  (normally consolidated specimen) and  $C_c/C_r = 10$ . It is evident from the figure that there is a general trend of decrease in hydraulic conductivity with the increase of hydraulic gradient. If  $C_k$  is high, the hydraulic conductivity ratio is also high, i.e reduction in hydraulic conductivity ratio is low. For low values of  $C_k$  (*ie.*  $C_c/C_k = 2.5$  or 5) the hydraulic conductivity ratio featured a tendency to increase after reaching a minimum. This increase is dependent on the  $C_c/C_k$  ratio and  $\sigma'_{v0}$ . The reasons for this could be explained from  $C_c$  and  $C_k$  plots as shown in Figures 6.5 and 6.6. Here  $C_c/C_k$  ratio was varied by keeping  $C_c$  constant and varying  $C_k$ . For a constant  $C_c$  value, the change in the void ratio is fixed but with the decrease in  $C_k$  large increases in hydraulic

conductivity are possible. For over-consolidated soil ( $\sigma'_c / \sigma'_{v0} \ge 1$ ),  $C_c / C_t$  ratio behaves differently than that of normally-consolidated soil and the results are shown in Figure 6.12. The initial portion of the curves of this figure show an increase in hydraulic conductivity because of swelling and the soil behaves under Condition 1. The other parts of the curves are similar to the shape of the normally consolidated soil.

Figure 6.13 shows the results for different values of  $\sigma'_c / \sigma'_{v0}$ . For the situation  $\sigma'_c / \sigma'_{v0} \leq 1$  (*ie.* within the normally consolidated region) hydraulic conductivity is independent of the  $\sigma'_c / \sigma'_{v0}$  ratio. All the curves are identical and there is a general trend of decrease in hydraulic conductivity with the increase in hydraulic gradient. For the condition  $\sigma'_c / \sigma'_{v0} > 1$  (*ie.* effective stress is less than preconsolidation pressure) the situation is different and the hydraulic conductivity starts increasing as the gradient is increased. However, as the ratio approaches 1, hydraulic conductivity ratio decreases after a certain hydraulic gradient. It can also be seen from the results that there are small increases in hydraulic conductivity up to a certain ratio of  $\sigma'_c / \sigma'_{v0}$  and beyond which no change in the hydraulic conductivity is evident. Similar results were obtained for both types of soil used.

For  $\sigma'_{v_0}/L_0\gamma_{w_1}$  both normally consolidated and over consolidated regions were considered and the results are presented in Figures 6.14 and 6.15. The analyses were carried out considering  $L_0\gamma_w = 0.50$  and keeping  $\sigma'_c/\sigma'_{v_0} = 0.50$  for normally consolidated and 1.50 for over-consolidated soils. As is evident from Table 6.3, there is no change in hydraulic conductivity ratio with the changes in  $\sigma'_c$  for normally consolidated soil but for over consolidated soil minimal change was observed at a very low  $\sigma'_c$ . Therefore, it could be considered from the results presented in Figures 6.14 and 6.15 are not influenced by changes in  $\sigma'_c$ . It can be seen from Figure 6.14 that, for normally consolidated soil, there is a decrease in hydraulic conductivity with increase in



**Figure 6.11:** Effect of  $C_c/C_k$  on Hydraulic Conductivity Ratio of Normally Consolidated Soil.



**Figure 6.12:** Effect of  $C_c/C_k$  on Hydraulic Conductivity Ratio of Over-Consolidated Soil.



**Figure 6.13:** Effect of  $(\sigma'_{\nu c}/\sigma'_{\nu 0})$  on Hydraulic Conductivity Ratio  $(\sigma'_{\nu 0} = 100)$ .

hydraulic gradient. This decrease in hydraulic conductivity is found to be higher with the decrease in  $\sigma'_{\nu 0}/L_0 \gamma_w$  ratio. As can be seen for over-consolidated soil from Figure 6.15, increases in hydraulic conductivity is higher as the  $\sigma'_{\nu 0}/L_0 \gamma_w$  ratio is decreased.

# 6.7 CONCLUDING REMARKS

A theoretical analysis was presented to estimate the effect of specimen volume change resulting from the use of elevated hydraulic gradients in the laboratory hydraulic conductivity measurement. The theoretical results were compared with experimental results obtained for two local soils. It was found from both the results that hydraulic conductivity decreases with the increasing gradient. For particular soil and the test conditions used with water as permeant, a reduction of less than a factor of 1.5 was observed when the gradient was increased up to 300. This variation is very insignificant compared to the variations mentioned in Chapter 2 (up to  $10^{6}$  times).

Different options of gradient application were discussed and their suitability for FWP was judged on the theoretical basis. It was found that Option 1, application of hydraulic gradient by reducing the top pore pressure and increasing the bottom pore pressure by the same amount, is most suitable for FWP hydraulic conductivity testing. A wide range of parametric values was considered for the evaluation and it was found that different parameters have a varying influence on hydraulic conductivity. It was also found that the effect is different from normally consolidated to over-consolidated soils. The order of influence of the parameters was also identified for both normally consolidated and over consolidated soils.



**Figure 6.14:** Effect of  $(\sigma'_{\nu 0} / L_0 \gamma_w)$  on Hydraulic Conductivity Ratio for Normally Consolidated Soil ( $L_0 \gamma_w = 0.5$ ).



Figure 6.15: Effect of  $(\sigma'_{v0} / L_0 \gamma_w)$  on Hydraulic Conductivity Ratio for Over-Consolidated Soil ( $L_0 \gamma_w = 0.5$ ).

Non-dimensional analyses for the parameters were carried out in the present study. A general trend of decreasing hydraulic conductivity ratio with increasing hydraulic gradient was observed. Normally, increases in  $C_c/C_k$  ratio decrease hydraulic conductivity ratio. Within the normally consolidated region, hydraulic conductivity ratio is independent of  $\sigma'_c/\sigma'_{v0}$  ratio and there is a general trend of decrease in hydraulic conductivity with the increase in hydraulic gradient. For over-consolidated soils, hydraulic conductivity starts increasing with the increase of hydraulic gradient, but for different  $\sigma'_{v0}/L_0\gamma_w$  ratios, a decrease in hydraulic conductivity was observed with the increase of hydraulic gradient. This decrease in hydraulic conductivity is found to be higher with the decrease of  $\sigma'_{v0}/L_0\gamma_w$  ratio. Overall, however, the variation in hydraulic gradient of 360. It should be noted, however, in this study, the likely effect of piping or particle migration was not modeled. This may generate increasing or decreasing effect depending on the operating mechanism.

# 7. CLAY HYDRAULIC CONDUCTIVITY AND PERMEANT ELECTROLYTE CONCENTRATION

## 7.1 INTRODUCTION

As discussed in Chapter 2, electrolyte concentration in the permeating fluid may have significant influence on hydraulic conductivity of compacted clays. Previous research on the effects of electrolyte concentration on clay conductivity was undertaken mainly on two fronts, namely geotechnical and geo-environmental engineering, and agricultural and soil science. Early work in geotechnical engineering involved the study of clay dispersion, erodibility and potential piping failure of earth dams resulting from the variations in electrolyte concentration in impounded water (*eg.* Aitchison and Wood, 1965). The work in agricultural and soil science mainly targeted the control of decrease in soil hydraulic conductivity and subsequent water logging resulting from dispersion of clay particles. This was considered to occur when water with low salt concentration permeates into sodic soils initially (*eg.* Quirk and Schofield, 1955). In geo-environmental engineering, research was focused on clayey barriers for waste containment where the issue is primarily the potential for increase of hydraulic conductivity due to high electrolyte concentrations prevalent in some waste leachates.

Several reviews of the research on chemical effects on clay hydraulic conductivity have been published (*eg.* Shackelford, 1994; Mitchell and Madsen, 1987). However, these reviews contained only limited coverage of the effects of electrolytes on clay hydraulic conductivity. In recent times, several investigators have carried out a substantial amount of research work (*eg.* Yang and Barbour, 1992; Barbour and Yang, 1993). In this chapter, a detailed literature synthesis is presented on this issue and experimental results on hydraulic conductivity obtained for a laboratory compacted local soil are presented.

# 7.2 MECHANISMS OF CLAY-ELECTROLYTE INTERACTION

It is well established that the soil hydraulic conductivity is primarily controlled by the clay fraction, and can increase or decrease when clay particles interact with the electrolytes in the permeating liquid. Clay particles are built of clay minerals, which mainly consist of two basic crystal structures; namely a tetrahedral sheet of silicon and oxygen and an octahedral sheet of magnesium and aluminum and hydroxyl. The basic clay mineral is a unit formed by stacking these sheet structures with different bonding between them. Owing to broken bonds or isomorphous substitution within clay mineral structures, clay particles develop a net negative charge on the surface (Mitchell, 1993). This net negative charge creates a repulsive force between the clay particles, which operates against the van der Waal attractive force, which operates between clay particles. However, the cations as electrolytes in the soil-pore water system (Na<sup>+</sup>, Ca<sup>++</sup>, Mg<sup>++</sup>, K<sup>+</sup> etc.) are attracted to the clay particle surface and neutralise some of the negative charges on the clay surface. If the concentration of electrolytes within the system changes, an unbalanced force would result, which has the potential to change the clay structure. For example, a net attractive force can lead to flocculation of clay particles while a net repulsive force can lead to dispersion of clay particles. In the following sections, the primary mechanisms, which can give rise to clay structural changes in this manner, are discussed.

## 7.2.1 Changes in Diffuse Double Layer

Because of the negative charge at the clay surface, the cation concentration is a maximum at the clay particle surface and, away from the particle surface, diffuses to that of the bulk pore solution which is in electric neutrality with solution anions. The zone over which this happens is known as the diffuse double layer. The thickness of this layer is a measure of the separation distance of clay particles, and is normally

described by the Gouy-Chapman equation. This is discussed in more detail in Chapter 2.

According to the Gouy-Chapman equation, it is clear that for a certain electrolyte solution at a certain temperature, increasing electrolyte concentration or cation valence will lead to a decrease in the double layer thickness or vice versa. In other words, permeating with liquids containing higher electrolyte concentrations or cations with higher valence than already in the original pore liquid (for example, Ca<sup>++</sup> replacing Na<sup>+</sup>) can lead to flocculation of clay particles, which has the potential to increase the hydraulic conductivity. Similarly, the opposite condition will lead to the dispersion of clay particles which has the tendency to decrease the hydraulic conductivity by reducing the effective pore space available for fluid passage. Mesri and Olson (1971) stated that the higher pore salt concentration retards the full mobilisation of the diffuse double layer thickness, increasing the effective void space and leading to a higher hydraulic conductivity.

## 7.2.2 C-axis Expansion and Contraction

The c-axis is the central axis of unit layers in a mineral, and the distance between the beginning of one layer to the next is referred to as the c-axis spacing (Mitchell, 1976). For more stable minerals such as kaolinite and illite, unit layers are bonded by relatively strong ionic bonds ( $H^+$  in kaolinite and  $K^+$  in illite) and this distance is usually fixed. However, some minerals in the smectite group such as montmorillonite do not have any special bonding between the unit layers. The water molecules and other ions can go in and out of the inter-unit layer spaces, making the c-axis spacing highly variable. Hence, these minerals show high swell/shrink properties with water. Similarly, the size of the hydrated cation within the inter-unit layers influences the c-axis spacing. For example, replacement of  $Mg^{++}$  (hydrated ion radius = 10.8Å) with  $K^+$  (hydrated ion radius = 3.8 to 5.3Å) will decrease this spacing and can lead to volumetric shrinkage and hydraulic conductivity increase.

### 7.2.3 Fixation

Normally, in smectite minerals, there is insufficient negative charge in the unit layer to bond cations between them. However, high charge smectite and vermiculite (which has a similar structure to smectite with hydrated interlayer  $Mg^{++}$ ) can undergo complete c-axis collapse to illite structure when  $K^{+}$  ions get fixed at interlayer spacing. This is associated with decreases in both the mineral volume (about 28%) and the double layer thickness, and usually features a significant decrease in cation exchange capacity (Quigley *et al.*, 1988).  $K^{+}$  fixation can result in soil shrinkage and potential for cracking and large increases in hydraulic conductivity.

#### 7.2.4 Ion Exchange

Cations held at the clay double layer as well as at interlayer spacing contribute to ion exchange sites where one ion can be replaced by another. Common cations found in soil have the following descending order of replacing power:  $AI^{+++} > Ca^{++} > Mg^{++} > K^{+}$ > Na<sup>+</sup>. However, it is possible for this replacing order to be overridden if ions of lower replacing power are present in higher concentrations. Ion exchange can affect the clay structure by double layer changes due to changes in valence, ion size and other mechanisms such as K<sup>+</sup> fixation discussed above. The cation exchange capacity (CEC) of a soil, as discussed in Chapter 2, is a measure of this property and a high CEC in soil generally represents high clay content and a high potential for clay structural changes due to electrolytes. It is established that, to some extent, anions too can exchange in the clay-water-electrolyte system, but the underlying mechanisms and effects are not clearly stated. There may be situations that some anions are attracted to the clay particles (*eg.* edges) and thereby increase the net negative charge on the particle. This tends to increase the double layer thickness and leads to dispersion. For example, this appears to be the main mechanism by which the phosphates act as effective additives for clay dispersion or deflocculation (Mitchell, 1976).

## 7.2.5 Change in pH

A change of pH in electrolyte solution can also affect the clay structure and conductivity. High pH (alkaline) liquids tend to disperse the clays whereas low pH (acidic) liquids lead to flocculation of clays. However, the effects of pH are of greatest importance in kaolinite and of lesser importance in illite and montmorillonite (Mitchell, 1976). If the interlayer  $K^+$  is leached out by acidic leachates (*eg.* pH about 2 in some acidic mine tailings), illite would experience an increase in the unit spacing (*ie.* swelling) and transform to the less stable vermiculite mineral, which is the reverse of  $K^+$  fixation, as discussed previously.

# 7.3 EFFECTS ON HYDRAULIC CONDUCTIVITY

## 7.3.1 General

In the previous section, the major mechanisms governing the interaction of clay-pore water-electrolyte system were discussed. Increases in electrolyte concentrations can lead to flocculation of the clay structure: volumetric shrinkage of clay and, in the extreme, cracking of the clay coupled with dramatic increases in hydraulic conductivity. A decrease in electrolyte concentration has the tendency to disperse clay particles, cause volumetric swelling of clay and, hence, has the potential to decrease the hydraulic conductivity. However, the degree of change would depend on a number of variables including the externally applied and residual stresses within the clay medium. In the following discussion, a literature synthesis is presented on the effects of hydraulic conductivity by the major influencing variables.

## 7.3.2 Density and Viscosity

The hydraulic conductivity of clay is also a function of the density and viscosity of the permeant. As discussed in Chapter 2, the "intrinsic permeability" is used to refer to the component of conductivity which depends entirely on clay pore geometry and structure. As shown in Equation 2.4, the hydraulic conductivity is then obtained by multiplying by  $\gamma/\eta$ , where  $\gamma$  and  $\eta$  are respectively the permeant density and viscosity. Therefore, the change in conductivity depends on the ratio of  $\gamma/\eta$  of permeant to water. Table 7.1 shows how this ratio typically changes for increasing NaCl concentrations. Hence, a small reduction in conductivity can be expected with increasing NaCl concentration.

**Table 7.1:** Unit Weight-Viscosity of NaCl at Different Concentrations at  $25^{\circ}$ C (source: Ridley *et al.*, 1984)

NaCl	η	γ	γ/η	$(\gamma / \eta)_{_{NaCl}}$
%	Pa s	kN/m <sup>3</sup>	m²/s	$(\gamma / \eta)_{water}$
	x 10 <sup>-4</sup>		x 10 <sup>6</sup>	
0	8.9	9.77	10.98	1.00
1.0	9.06	9.84	10.88	0.99
5.0	9.62	10.11	10.51	0.96
10.0	10.48	10.44	9.96	0.91
20.1	12.98	11.05	8.52	0.78
30.0	16.92	11.65	6.89	0.63

#### 7.3.3 Mineral Type and Electrolyte Concentration

Electrolyte effects on hydraulic conductivity of soils depend primarily on their mineral composition and particle size distribution. Mesri and Olson (1971) provided void ratio – hydraulic conductivity relationships for smectite, illite and kaolinite slurry samples and

showed that the effects on hydraulic conductivity by NaCl and CaCl<sub>2</sub> solutions were largest for smectite, and moderate for illite and negligible for kaolinite. Ouirk and Schofield (1955) used lightly packed silty loam (40% illite, 40% kaolinite, 20% vermiculite) initially saturated with 1M NaCl, 0.2M KCl, 0.0316 M MgCl<sub>2</sub>, 0.01M CaCl<sub>2</sub>, and then permeated with successively dilute solutions to study the effects of clay dispersion and reduction in hydraulic conductivity as applicable to irrigation. The threshold concentrations, which gave 10 to 15% reduction in soil hydraulic conductivity at pH around 5.2 to 5.4, were 0.25 M for NaCl, 0.067 for KCl, 0.001 MgCl<sub>2</sub> and 0.0003 for CaCl<sub>2</sub>. These results indicate that NaCl is the most troublesome where substantially high salt concentration is needed in the irrigation water to maintain the clay in the flocculated state. Hardcastle and Mitchell (1974) studied the dispersion of clays for illite and crushed basalt slurry mixtures initially consolidated with 0.6M NaCl and then permeated with 0.1M and 0.05M solutions. They observed hydraulic conductivity decreases of 0.4 to 0.2 respectively, but found that these effects were reversible when permeants with higher electrolyte concentrations were reintroduced.

For compacted clayey barriers as in waste containment, the effect of increase in electrolyte concentration is more relevant. Figure 7.1 schematically illustrates the typical effects of electrolyte concentration on clay conductivity as well as the double layer for both agriculture and waste containment problems.

Alther *et al.* (1985) measured the hydraulic conductivity of salt saturated (0 to 5% weight or 50,000 mg/L chlorides, sulphates and carbonates of K, Na, Mg, Ca) slurry filter cakes of bentonite in rigid wall columns. They explained their results using the Gouy-Chapman theory, and noted that  $K^+$  showed the most pronounced and most linear effect (with respect to concentration) on bentonite conductivity. Cl<sup>-</sup> was also associated with significant conductivity increases, and apparently controlled the rate of conductivity increase with increased ion concentration. Divalent cations (Ca<sup>++</sup> and Mg<sup>++</sup>) showed a rapid initial increase, but at about 1% concentration, reached a reasonably stable conductivity which is 3 to 4 times higher than that with water. In contrast, the concentration increase of KCl and NaCl solutions (*ie.* K<sup>+</sup> and Na<sup>+</sup>) featured a relatively slow but steady increase (3 to 4 times), and did not reach stable conductivities within 0 to 5% concentration. They also observed progression of cracking of bentonite slurry by 0%, 3%, 5% and 10% KCl solutions.



**Electrolyte concentration** 



It should be noted that while the above observations may reflect the general effects of the increase in electrolyte concentration, they were obtained with high moisture content slurries. Direct data in relation to compacted clay with electrolytes at low concentrations are not readily available. One could expect that higher electrolyte concentrations are necessary to create hydraulic conductivity changes to compacted clay because of the over-consolidated nature of the soil and, perhaps, due to the residual stresses generated during compaction.

Several studies were conducted on the effects of hydraulic conductivity of compacted clay with brine (saturated NaCl) solutions, and were summarised by Barbour and Yang (1993). Many of these tests were conducted on glacial soils containing significant quantities of Ca-montmorillonite. Mostly rigid wall permeameters with vertically unconfined conditions were utilised. Increases in hydraulic conductivity up to 10 times were recorded. Ridley *et al.* (1984) reported increases of 100 and 1000 times of hydraulic conductivity for compacted montmorillonitic clayey soils permeated with brine in rigid wall permeameters. They attributed this to flocculation (or coagulation) of clay particles and opening of larger flow pore channels. However, they did not rule out the possibility of side wall leakage due to volumetric shrinkage of soils.

#### 7.3.4 Mixed Ion Solutions

The electrolyte concentration in waste liquids results from the concentration of various ions. The effect of mixed ion solutions on clay conductivity is more complex and will be a combined effect of all ions. The ion exchange will be a controlling mechanism, but will be influenced by relative concentrations and replacing power and the valence of the individual ions. Quigley *et al.* (1988) noted that compacted silty clay permeated with a domestic waste leachate produced a slight reduction in hydraulic conductivity. They reported the retardation of Na<sup>+</sup> and K<sup>+</sup> ions by adsorption on to clay particles and displacement of Mg<sup>++</sup> and Ca<sup>++</sup> ions to maintain electrical neutrality. This is considered to be responsible for the "hardness halo" effect observed below some landfills where Mg<sup>++</sup> and Ca<sup>++</sup> concentrations in pore liquid increased with depth.

Considerable research work is reported on compositions of Na<sup>+</sup>, Ca<sup>++</sup> and Mg<sup>++</sup> on the dispersivity of clayey soils. Since Na<sup>+</sup> is the most easily exchangeable ion, the Sodium Adsorption Ratio (SAR), discussed in Chapter 2, is commonly used as a measure of dispersivity and change in hydraulic conductivity. Aitchison and Wood (1965) showed that the cation concentration of the permeant required to change the clay structure from a dispersed to a flocculated state increased with the SAR of the soil. A number of researchers have shown that in clay compacted after mixing with high salt solutions the conductivity increase resulting from the subsequent permeation with high strength salt solutions is markedly reduced (*eg.* Huag *et al.*, 1988).

#### 7.3.5 Effect of Confining Stress and Hydraulic Gradient

Effects of effective stress and hydraulic gradient were discussed in the previous chapters. Effects of electrolyte concentration on the confining stress and hydraulic conductivity is discussed in this section. There is significant experimental evidence to suggest that the effective stress generated by confinement decreases the clay structural changes by electrolytes. Yang and Barbour (1992) showed experimentally that the application of a vertical confining stress of about 75 kPa is sufficient to bring down the conductivity increase of compacted clay from 6 times to 2 times when permeated with brine solutions. They also found a 50 times increase in hydraulic conductivity compared to water after brine permeation at 20 kPa effective stress and hydraulic gradient of 60 for the slurry sample. As can be seen from Figure 7.2, with the increase in effective stress from 20 kPa to 40 kPa hydraulic conductivity decreased almost back to the original value. These two results are important to express the effect of effective stress on hydraulic conductivity of the soil specimen.

# 7.4 VOLUME CHANGE AND OSMOTIC CONSOLIDATION

The mechanisms outlined in Section 7.2 give rise to volume change within clay when exposed to electrolytes. The resulting volume change is normally referred to as osmotic consolidation driven by osmotic pressure due to electrolytes irrespective of the mechanism. The dilemma associated with FWPs and RWPs can only be sorted out by analysing resultant volume changes arising from mechanical effective stresses and osmotic pressures. In this chapter, basic theoretical concepts associated with this process and some experimental evidence are presented.



Figure 7.2: Hydraulic Conductivity Variation with Time for a Constant Head Test of a Kneading Compacted Sample (source: Yang and Barbour, 1992).

## 7.4.1 Effective Stress Principle

In traditional geotechnical areas, effective stress ( $\sigma$ ') is considered to relate with total stress ( $\sigma$ ) and pore water pressure (u). However, in situations where the physicochemical force fields originated from the diffused double layers of ions surrounding the clay particles are prevalent, the effective stress principle needs to be modified to account for these force fields. The major forces can be considered as an attractive unit force ( $A_F$ ) and a repulsive unit force ( $R_F$ ). The attractive force between clay particles arises from van der Waals forces and pulls particles together and acts with other body forces leading to decrease of the specimen volume. Repulsive force between particles is due to electrostatic repulsion between the negatively charged clay surfaces arising from Coulomb repulsion and osmotic forces.

At physico-chemical equilibrium of clay-porewater-electrolyte system, the repulsive force will balance the attractive force. However, when the clay is exposed to an increase/decrease in electrolyte concentration, there will be an imbalance in these forces. The true effective stress change ( $\Delta \sigma'$ ) in the clay medium can then be expressed as (Barbour and Yang, 1993):

$$\Delta \sigma' = \Delta \sigma - \Delta u - (R_F - A_F) \tag{7.1}$$

where  $\Delta\sigma$  is the total stress change;  $\Delta u$  is the externally measured water pressure change; and  $(R_F - A_F)$  is the imbalance stress due to attractive and repulsive forces. If  $\Delta\sigma$  and  $\Delta u$ are zero, then  $\Delta\sigma' = (R_F - A_F)$ . This means that the effective stress will increase/decrease leading to sample shrinkage (consolidation)/swelling with higher/lower electrolyte concentrations. However, as the clay-electrolyte system approaches equilibrium,  $(R_F - A_F) \rightarrow 0$ , and consequently,  $\Delta\sigma' \rightarrow 0$ . This means that the preconsolidation pressure of the clay can increase with higher electrolyte concentrations.

As the physicochemical stress  $(R_F - A_F)$  approaches zero, soil specimen will consolidate. If

the external total stress and net pressures are kept constant, introduction of brine (*ie.* high electrolyte concentration permeant) might lead to larger effective porosity with consequent increases in hydraulic conductivity. Barbour and Yang (1993) stated that ( $R_F$  - $A_F$ ) plays a dominant role in controlling the volume change and the shear strength behaviour of clay soils. Macro (*ie.* cracking) and/or micro (*ie.* fabric changes) structural changes can occur as a result of the changes in the net inter-particle unit force (Yang and Barbour, 1992).

## 7.5 EXPERIMENTAL INVESTIGATION

An experimental investigation was carried out to examine the effects of NaCl on the hydraulic conductivity of compacted clay. This investigation was also prompted when in Echuca, a local Water Authority encountered problems of retaining waste water having low levels of salt concentration in their waste water lagoons. To address this issue, soil samples collected from Echuca were used in the testing program. The investigation was carried out at different concentrations of electrolyte. Tests for the physical and the mineralogical analyses were carried out and the results are presented in Chapter 3. Flexible Wall Permeameters (FWPs), Consolidation Cell Permeameter and Compaction Mould Permeameter were used for investigation and the results are discussed in separate headings for each permeameter. As discussed in Chapter 4, the same procedure was followed in the preparation of soil samples. The compaction moisture content was used at 2.8% wet of the standard optimum moisture content of 24.2%, as set out in Chapter 4. The hydraulic conductivity tests were continued at least up to 2 pore volumes or the time required to attain a stable hydraulic conductivity to fulfil the requirement for the termination criteria as discussed in Chapter 4. A few effluent samples were analysed for Na<sup>+</sup> and Cl<sup>-</sup> concentrations at different times of testing and were compared with the original concentrations of the influent. A typical result of the analysis is presented in Figure 7.16. Tests for electrical conductivity (EC) and turbidity values for effluent were also carried out. EC values of effluent were found to increase with time up to a certain level indicating the presence of higher concentrations of salinity. A high turbidity value

at the start of the testing with a gradual decrease with time was observed and the results are presented in Figure 7.17.

#### 7.5.1 Flexible Wall Permeameters

#### Test program

A test program was carried out considering a wide range of NaCl concentrations from 0.05 M to 2 M NaCl (1 M NaCl = 58.5 gm/l) for using FWP for this investigation. The intention was to examine at what threshold concentration level the clay hydraulic conductivity or structural changes start occurring and what could be the likely effects at high concentrations of NaCl. These tests are numbered from ES1 through ES6 and the test program is presented in Table 7.2. First letter of the test number is for the soil type (E for Echuca Soil), second letter is for the permeant (S for salt) and followed by the test number. All the tests were continued for sufficient amount of time (*ie.* at least for two pore volumes) to allow interaction to take place between the electrolyte solution and the clay.

As can be seen from the test conditions in Table 7.2, the effective stresses and hydraulic gradients from test ES1 through ES6 were 100 kPa and 200 respectively. For the test ES7 the corresponding values were 30 kPa and 30. As discussed earlier, the hydraulic conductivity tests were carried out in two different stages, *ie.* initially with water followed by the electrolyte solution. The symbols  $k_w$  and  $k_{salt}$  are used to express hydraulic conductivity with respect to water and salt respectively. The results of the investigations are presented and discussed in the following sections.

Salt	0.05 M	0.1 M	0.2 M	0.4 M	1 M	2M	2M	2M
	NaCl	NaCl	NaCl	NaCl	NaCl	NaCl	NaCl	NaC1
Test number	ES1	ES2	ES3	ES4	ES5	ES6	ES7	ES8
Effective	100	100	100	100	100	100	30	30
stress (kPa)								1
Specimen	50	50	50	50	50	50	20	50
thick. (mm)								
Hyd.	200	200	200	200	200	200	30	40
Gradient, i								
k <sub>w</sub>	13.30	11.30	6.70	11.30	5.43	8.62	4.07	192.00
x 10 <sup>-11</sup> (m/s)								
k <sub>min</sub> x	_	5.47	7.70	4.84	3.80	8.99	-	76.80
10 <sup>-11</sup> (m/s)								
k <sub>salt</sub> max	-	5.88	9.30	5.08	4.18	9.27	22.2	167.00
x 10 <sup>-11</sup> (m/s)	f							
k <sub>salt</sub> final	4.90	4.82	7.30	3.43	3.26	8.46	2.55	163.00
x 10 <sup>-11</sup> (m/s)								
pv	3.10	2.80	3.75	2.90	2.20	3.20	0.54	4.50

Table 7.2: Hydraulic Conductivity Test Results with Flexible Wall Permeameter

## • Results and discussions

Results of the investigation are presented in Figures 7.3 through 7.9 and are tabulated in Table 7.2. Hydraulic conductivity values presented in different figures and in Table 7.2 were calculated on the basis of the inflow volumes. This approach was adopted because the outflow volumes are different from the inflow volumes due to the changes in the sample volume and the likely structural changes that might have occurred within the specimen. The reasons for such changes were discussed in Chapter 4. Volume changes (*ie.* the cell volume and the difference between the outflow and inflow volumes) for all the tests were monitored from the beginning of the testing. Volume changes in Figure 7.8(c) are shown from the beginning of the test. It could be seen that both volume

changes are initially high but reduce with time. As discussed in Chapter 4, the initial increases in the cell volume could be due to the consolidation of the specimen, which was completed within 0.2 pore volumes of flow. From 0.2 to 1.3 pore volumes (*ie.* up to the introduction of electrolyte solution) the cell volume remained relatively constant. During this period, differences in the outflow and inflow volumes could be due to the saturation effect only. After the saturation with water is completed, there should not be any differences between the inflow and outflow volumes. It could be seen that the differences of flow, indicating that there is no difference between inflow and outflow rates. Under these circumstances, it seems reasonable to ignore the initial volume changes that take place thereafter. Therefore, all other volume changes presented in this chapter start from the introduction of saline water.

As discussed in Chapter 2, flocculation and dispersion are the two mechanisms of clayelectrolyte interaction that change hydraulic conductivity. Clay particles flocculate at high electrolyte concentration leading to structural changes increasing hydraulic conductivity. With the decrease in electrolyte concentration in the pore fluid, clay particles disperse leading to the decrease in hydraulic conductivity by reducing the effective pore space. Results presented in Figures 7.3 to 7.6 are for test numbers ES1 to ES4, where the permeation was made with low levels of electrolyte concentrations (0.05M to 0.4M NaCl). Test results with 0.05M NaCl did not show any increase in hydraulic conductivity. At this concentration volume changes were very insignificant. Hence it could be argued that the salt solution at this concentration does not have any significant influence on hydraulic conductivity. At higher concentrations very little increase in hydraulic conductivity at some stages of testing was observed. Figures 7.7 to 7.9 are the results for test numbers ES5 to ES7, where the tests were carried out at 1M and 2M NaCl concentrations. From Figures 7.7(a) to 7.9(a) it can be seen that only a minimal increase in hydraulic conductivity was observed after electrolyte solution was introduced. However, as can be seen from Figures 7.7(b) to 7.9(b), the inflow and the outflow volumes started to show significant changes after electrolyte solution was introduced into the specimen. It is evident that, initially, the inflow increased more than the outflow, and, at the same time the cell volume increased indicating a total sample volume decrease or a sample consolidation. But, after the passage of about 0.7 pore volume of NaCl solution, both the volumes reached a steady state. According to the theoretical concept described previously, the osmotic pressure created by the entrance of electrolyte solution should have increased the effective stresses and, consequently, the sample should have consolidated displaying a cell volume increase. The increased net inflow appears to have resulted from the clay structural changes which created a net expansion of the sample pore spaces despite the external consolidation (or shrinkage) of the sample. Eventually, both the processes came to equilibrium arresting external and internal volume changes (as  $(R_F - A_F) \rightarrow 0$  and  $\Delta\sigma' \rightarrow 0$ ).

After the stable hydraulic conductivity with 2M NaCl was reached in test ES6 (Figure 7.8), the tap water was reintroduced and the test was continued further. Although the expectation was that the reintroduction of tap water would bring back the hydraulic conductivity to its original position, a further slight increase in hydraulic conductivity was observed. This indicates that the structural changes that occurred due to the permeation of electrolyte solution are irreversible at least within the duration of the testing carried out subsequently. This is also in agreement with the findings of Cadwallader (1985). After the reintroduction of tap water the slight increase in hydraulic conductivity may also be due to the variation in viscosity of the two permeants. As can be seen from Table 3.8, the absolute viscosity of 1M NaCl is 8% higher than that of tap water, the hydraulic conductivity should be 8% higher for water than 1M NaCl. After reintroduction of water, the test was continued for 1.3 pore volumes of flow. As discussed in Chapter 4, at least 1 pore volume of flow is required to wash the permeant out from the specimen. From Figure 7.17 it could be seen that about 2 pore volumes of flow is required to achieve the same influent-effluent concentrations. Therefore, to allow for the dispersion and a consequent decrease in hydraulic conductivity to occur, the test should have continued for a longer period of time.

As can be seen from Table 7.2, the volume change in case of 1M NaCl was about 0.7%. This volume change should be dependent on the type of soil and the concentration of the permeant. Barbour and Yang (1993) found a sample volume change of 2 - 3.5% for



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Figure 7.3: Effect of 0.05M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP



Figure 7.4: Effect of 0.1M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP



Figure 7.5: Effect of 0.2M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP



Figure 7.6: Effect of 0.4M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP







Figure 7.8: Effect of 2M NaCl on Hydraulic Conductivity of Echuca Soil using FWP



**Figure 7.9:** Effect of 2M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP at a Hydraulic Gradient of 30.

Regina clay and 1% for Indian Head Till after permeation with brine. They also pointed out that the reason for decrease in volume due to brine permeation results from the apparent increase in the preconsolidation stress and a decrease in the compressibility of the specimen until the new preconsolidation stress has been reached. The reasons for not observing a large volume change as was pointed out by Barbour and Yang (1993) could be the soil mineralogy and the low concentration of NaCl used.

If the volume change in the test specimen did not occur, it is possible that significant changes in hydraulic conductivity might have been observed. But, due to the confining stress in the flexible wall permeameter, structural changes in the specimen due to the physico-chemical processes were masked by the volume changes and large increases in the hydraulic conductivity were not observed. If the volume changes were not allowed to occur, the resulting increase in hydraulic conductivity could be calculated from the e–log k relationship as discussed in the Figure 6.5. However, Figure 6.5 was developed from the changes in the void ratio and the hydraulic conductivity due to the effective stress consolidation and the effect of chemically induced consolidation was not considered. Effects of chemically induced changes within the soil structure could give different values of hydraulic conductivity than the effective stress changes and the result between the two could not be compared. Further research is required to resolve this issue.

In order to reduce testing time, one of the alternatives in testing is to introduce permeant without initial permeation with water. The effect of this approach is known as the first exposure and was discussed in Chapter 2. A test to assess the first exposure effect of 2M NaCl was carried out in the laboratory and the results are presented in Figure 7.11. As can be seen from this figure about 20% increase in hydraulic conductivity was observed in the test featuring the first exposure effect in comparison to the result obtained in the usual way using an identical specimen with the same concentration. As was highlighted in Chapter 2, a direct comparison of two seemingly identical specimens may not be very reasonable because of the natural variability that can arise from the specimen preparation process.

In tests ES6 and ES7 the effective stress, hydraulic gradient and specimen thickness were

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chosen as test variables. The results of the two tests show a significant difference in the increase in hydraulic conductivity. Since the experimental and theoretical results in Chapter 6 indicated that no significant changes in hydraulic conductivity occurs up to a hydraulic gradient of 300, it can be argued that the remaining two variables could be responsible for changes observed. But it is not clear which one is the more influential variable. This issue can be examined by comparing test results of ES7 and ES8. The variable in Tests ES7 and ES8 is specimen thickness neglecting the little variation in hydraulic gradient. From Figure 7.10 (ie. Test ES8), it could be seen that the ratio between minimum to the maximum hydraulic conductivity after the introduction of electrolyte solution is 2, where the specimen thickness was 50 mm. For 20 mm thick specimen in Test ES7, this ratio was 5. Therefore, thickness of the specimen might have been the contributing factor for such differences in hydraulic conductivity. As is discussed in Chapter 9, the desiccation cracking is more pronounced in the case of the thinner specimen than the thicker one. There are possibilities for through cracking to occur in the case of thinner specimens. On the basis of the limited tests undertaken, it seems that larger values of hydraulic conductivity increases may be observed when thinner specimens are used.

## 7.5.2 Consolidation Cell Permeameter

The test technique detailed in Chapter 4 was followed to carry out the testing with the consolidation cell permeameter using two GDS pressure volume controllers. Two tests were carried out in the consolidation cell permeameter using 1M and 2M NaCl concentrations. Test results are presented in Figures 7.12 and 7.13 and are tabulated in Table 7.3. From Table 7.3, it can be seen that the hydraulic conductivity increases by 9% after the introduction of 1M NaCl. However, for 2M NaCl the increase was more than 54 times indicating that a direct flow channel might have been established. Subsequently, it was difficult to monitor the pressure difference, and the test was terminated. The reasons for such increase could be due to the increases in the NaCl concentration and reduction in the effective stress from 100 kPa to 65 kPa. The increase in the NaCl concentrations



Figure 7.10: Effect of 2M NaCl on Hydraulic Conductivity of Echuca Soil at Low Effective Stress (30 kPa) and Hydraulic Gradient (40) Using FWP.



Figure 7.11: Effect of First Exposure of 2M NaCl on Hydraulic Conductivity of Echuca Soil Using FWP


(Inflow)



Figure 7.12: Effect of 1M NaCl on Hydraulic Conductivity of Echuca Soil Using Consolidation Cell Permeameter



Figure 7.13: Effect of 2M NaCl on Hydraulic Conductivity of Echuca Soil Using Consolidation Cell Permeameter.

could result in a more flocculated structure of the soil. Compared to 100 kPa, soil specimen at 65 kPa effective stress has more flexibility to swell. This could cause a net increase in the effective pore space with a consequent increase in hydraulic conductivity. Hence, the effect of effective stress on the hydraulic conductivity should be carefully considered while planning the testing schedule.

Salt	1 M NaCl	2M NaCl
Test number	ES9	ES10
Effective stress (kPa)	100	65
Hydraulic gradient, i	200	100
k <sub>w</sub> x 10 <sup>-11</sup> (m/s)	6.67	13.6
k <sub>salt</sub> max x 10 <sup>-11</sup> (m/s)	7.26	
k <sub>salt</sub> final x 10 <sup>11</sup> (m/s)	6.54	747.00
% Increase in hydraulic	9	5400
conductivity		
pv	1.83	2.18

 Table 7.3: Hydraulic Conductivity Test Results with Consolidation Cell Permeameter

#### 7.5.3 Compaction Mould Permeameter

A compaction mould permeameter was also used to study the effect of electrolyte concentration on hydraulic conductivity of compacted clay. Two testing techniques were used with the compaction mould permeameter as: (a) zero effective stress, and (b) samples with the confinement against swelling. As discussed in Chapter 4, the compaction mould permeameter was converted into a double ring permeameter to monitor the side wall leakage. The outer and inner rings of the permeameter were of the same area. The difference between the flow through the inner and outer rings of the specimen would indicate the side wall leakage. The outflow through the inner ring of the specimen would indicate the side wall leakage. The outflow through the inner ring of the specimen. All the tests were carried out at 30 kPa to 40 kPa inflow water pressure at the

top and zero outflow water pressure at the bottom. To reduce the testing time, the specimen thickness was reduced to 20 mm.

With zero effective stress and no confinement at the top, the soil specimen was allowed to swell or shrink freely. In such a situation, it was found that the side wall leakage is the one which dominates the flow. Minimal flow was detected through the inner half of the specimen and, in such a case, calculation of hydraulic conductivity was impossible. The results of the test are not reported in the report. Attempt was made to improve the leakage by putting a generous amount of grease along the sides of the permeameter. The test results in the form of hydraulic conductivity versus pore volume, and pore volume versus inner and outer flow and cumulative inner and outer flow that are presented in Figures 7.14(a) and 7.14(b) respectively. Outflow volumes through the inner and outer rings were collected separately and the results are reported in Figure 7.14(b). The test number is ES11 and the results of the investigation are tabulated in Table 7.4.

It could be seen from the Figure 7.14(a) that the hydraulic conductivity with water initially started at a higher value, decreased quickly with time and came to a stable hydraulic conductivity value. After the introduction of 4M NaCl solution, 15 times increase in hydraulic conductivity was observed. As is evident from Figure 7.14(b), there were initial differences between the inner and outer flow volumes as soon as water was introduced. After the permeation was made with the saline water, side wall leakage increased at a rapid rate and keeps on continuing with time. This could be attributed to the shrinkage of the specimen that moved the specimen away from the walls.

As discussed before, tests where the specimens were confined against swelling at the top were also carried out with the compaction mould. This was undertaken by compacting the specimen in the mould and placing it just touching the top cap of the mould. Test results of the investigation are tabulated in Table 7.4 as Test ES12, and are presented in Figure 7.15. It can be seen from the test results that the increase in hydraulic conductivity after the introduction of 2M NaCl solution is very negligible. Although a significant reduction in side wall leakage was observed in this technique, still noticeable side wall leakage was there. The problems of the side wall leakage, as





Figure 7.14: Effect of 4M NaCl on Hydraulic Conductivity of Echuca Soil Using Compaction Mould Permeameter



**Figure 7.15:** Effect of 2M NaCl on Hydraulic Conductivity of Echuca Soil Using Compaction Mould Permeameter with Swelling Restriction

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was discussed previously in this section, could be avoided by the technique proposed in Chapter 9.

Salt	4M NaCl	2M NaCl	
Test number	ES11	ES12	
Hydraulic gradient, i	150	150	
$k_w \ge 10^{-11} (m/s)$	8.95	2.35	
$k_{salt} \max x 10^{-11} (m/s)$	206.00	4.10	
$k_{salt}$ final x 10 <sup>-11</sup> (m/s)	153.00	2.50	
pv	8.5	1.9	

Table 7.4: Hydraulic Conductivity Test Results with Compaction Mould Permeameter

# 7.6 TESTING OF EFFLUENT

# 7.6.1 Testing of Effluent Concentration

Effluent concentrations of Na<sup>+</sup> and Cl<sup>-</sup> were measured at different times when NaCl was used as a permeant. Figure 7.16 presents the measured concentrations corresponding to the pore volumes of flow for test ES2. It could be seen from the figure that the relative cumulative effluent concentrations of Na<sup>+</sup> and Cl<sup>-</sup> are 72% and 75% respectively at 1.55 pore volumes. If the line is extended further, as is shown by the dotted lines in Figure 7.16, it could be seen that about 2 pore volumes correspond to 100% effluent concentrations. The test was continued up to 2.85 pore volumes of flow but the final effluent concentration measurement were not made and the test was terminated by considering the stable hydraulic conductivity. In cases where serious disruptions did not occur in the testing program, the tests were continued up to at least 2.0 pore volumes of flow or up to the level where stable hydraulic conductivity is obtained.



Figure 7.16: Effluent Concentration vs Pore Volume Relationship for Echuca Soil

# 7.6.2 Testing of Effluent Turbidity

Turbidity tests of the effluent samples were carried out at different times of the testing and one set of results is reported in Figure 7.17. Turbidity of tap water used for testing was 1 and the saline water (0.1M NaCl) turbidity was 2. At 0.5 pore volumes of flow, very high turbidity value of 32 of the effluent was observed. From the high turbidity values of the effluent, the presence of colloidal particles in the effluent is evident. This could probably be due to the particle migration from the soil specimen, as discussed in Chapter 6. Effluent turbidity dropped abruptly to almost a constant value and at the same time hydraulic conductivity also decreased indicating that the plugging of particles may have occurred.

# 7.7 CONCLUDING REMARKS

The effect of electrolyte concentration on hydraulic conductivity of compacted clay was tested in the laboratory by using FWP, consolidation cell permeameter and compaction mould permeameter. The testing was conducted over a wide range of NaCl concentrations, ranging from 0.05M to 4M. Changes in hydraulic conductivity or volume changes, which could be another form of compatibility assessment, were not observed when the permeation was made with 0.05M NaCl in flexible wall permeameter. In all other cases increase in the hydraulic conductivity and volume changes were observed. This increase was found to be higher as the NaCl concentration increased.

It was found from the test results that the permeants with higher electrolyte concentrations could increase the clay hydraulic conductivity, although the increase was not that significant in the case of FWP. In FWP, increases in the hydraulic conductivity may be reduced by osmotic consolidation resulting from elevated electrolyte concentrations. A volume change of about 0.7 % was observed during the permeation of 2M NaCl solution.



Figure 7.17: Effluent Turbidity vs Pore Volume Relationship for Echuca Soil

Therefore, it is essential in compatibility testing using FWPs to measure the changes in cell, inflow and outflow volumes in order to assess the soil structural changes.

In case of consolidation cell permeameter, an increase in hydraulic conductivity of about 54 times was observed at 2M NaCl concentration and at an effective stress close to preconsolidation pressure. This increase in hydraulic conductivity could be due to the structural changes and/or due to the side wall leakage. Both the mechanisms could be higher at low effective stresses. At 1M NaCl, very little increase in hydraulic conductivity was observed at higher effective stress.

Two types of tests were carried out with compaction mould permeameter: one with zero effective stress at the top of the soil specimen; and the other with confinement at the top of the specimen. With the zero effective stress at the top, the soil specimen had the flexibility to swell and shrink, consequently large increases in side wall leakage were observed. The inflow was mainly dominated by the side wall leakage and hydraulic conductivity measurements became meaningless. With the application of a side sealant, the side wall leakage was reduced when the effluent was water at the beginning of the test. But with the introduction of saline water this side wall leakage started increasing again. When a confinement was placed on the top of the specimen against swelling, side wall leakage reduced significantly and a relatively small increase in hydraulic conductivity was observed.

The tests carried out in this chapter highlight the problems associated with different types of permeameters used. Flexible wall permeameter suffers specimen volume change while the compaction mould and consolidation cell permeameter suffers from side wall leakage. Improvement of the testing technique is required to reduce or eliminate the problems related to different types of permeameters to monitor the changes in hydraulic conductivity. A new testing technique is proposed in Chapter 9, highlighting this issue more specifically.

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# 8 EFFECT OF WASTE LEACHATE AND ORGANIC LIQUIDS ON HYDRAULIC CONDUCTIVITY OF COMPACTED CLAY

# 8.1 INTRODUCTION

Clay liners in their natural or compacted state are used as a barrier to the pollution migration from the waste containment facilities for the protection of geo-environment. Within these containment facilities leachates are generated from the decomposition of waste in the presence of water. As was discussed in Table 2.2, these leachates may contain a variety of hazardous and non-hazardous organic and inorganic chemicals. Leaching of these chemicals may lead to the pollution and potential degradation of the natural environment, particularly the ground water. Although in the USA and Canada composite liners, clay in conjunction with synthetics, are mandatory, Australia is relying solely on the use compacted clay liners. Therefore, hydraulic conductivity of the clay liner material is the desirable property for the waste containment facilities. General design criteria for the earthen barriers require an in-situ hydraulic conductivity of less than  $10^{-9}$  m/s and the thickness be selected on the basis of water movement and an arbitrary design life.

Overseas research indicated that the hydraulic conductivity of the compacted clay can increase up to million fold when permeated with some chemicals specially with liquid hydrocarbons (Fernandez, 1989). Concentrated organic chemicals may cause a change in the ionic bonding conditions in the clay minerals, modifying the diffuse double layer around clay particles and draws the particles together. Shrinkage of this nature could increase the hydraulic conductivity by several orders of magnitude. Although in general, organic compounds increase hydraulic conductivity of a clay soil, certain compounds and reaction types may decrease the hydraulic conductivity (Mitchell and Madsen, 1987). Leachates, on the other hand, are supposed to have little effect on the clay hydraulic conductivity, and may well help to prevent clay cracking through desiccation due to the presence of dispersed colloidal particles. A detailed laboratory investigation was carried out on local soils used for liner construction with different chemicals. Traditional testing equipment such as compaction mould permeameter, consolidation cell permeameter and flexible wall permeameter (FWP) was used in the testing. Leachate, methanol and modified leachate at different concentrations were used and the results are discussed in the following sections.

# 8.2 MECHANISMS OF CLAY MINERALS INTERACTION WITH ORGANIC CHEMICALS AND WASTE LEACHATE

Interaction of organic chemicals and waste leachate with the clay minerals is influenced by several factors. Mitchell and Madsen (1987) stated that the influence of organic chemicals is primarily by their water solubility, dielectric constant, polarity, and the concentration of chemicals. In addition to these factors pH, double layer, flocculation of clay particles, shrinkage and swelling of the soil matrix and acid-base dissolution of soil are also important. The gradual replacement of pore fluid molecules by the molecules of the permeating fluid may result in the modification of net intermolecular forces changing the pore volume of the specimen which could result in the changes in hydraulic conductivity of the soil. The confining pressure, the locked-in stresses developed during compaction and the stiffness and strength of soil fabric will attempt to restrain any changes in the soil structure. But if the new intermolecular forces developed by chemicals exceed the existing intermolecular forces, then a change in the soil structure can be expected. The mechanism of effective stress increase due to electrolyte concentration was discussed in Chapter 7. A similar mechanism of effective stress increase is also expected for organic chemicals and leachetes. A detailed description of some of these factors was discussed in previous chapters and the remaining factors will be discussed in this chapter. 0

#### 8.2.1 Volume Change

Volume changes due to clay-electrolyte interactions were already discussed in detail in Chapter 7. Volume changes in the specimen could be considered to occur due to various reasons. These could be the substitution of one cation by the other, changes in double layer thickness and c-axis contraction or expansion. Depending on the chemicals and leachate composition of the permeant, the tendency exists for Ca<sup>++</sup> to replace  $Na^{\dagger}$  on the montmorillonite structure causing shrinkage and development of cracks. However, if the substitution takes place with  $Ca^{++}$  to  $Mg^{++}$  then less shrinkage is likely. The double layer of cations and water, which surround the clay particles, are important for clay/permeant compatibility. Double layer contraction at constant void ratio can increase the hydraulic conductivity of the soil because the increase in free pore space resulted in the development of the interconnected macro pores within the soil (Fernandez and Quigley, 1985). C-axis contraction of dispersed smectites caused by cation exchange and or increases in salt concentration is another factor similar to double layer contraction that may reduce soil plasticity and cause significant shrinkage of the clay liner. The fixation of  $K^{\dagger}$  by high charge smectites and vermiculites and consequent c-axis collapse of crystal lattice to illite which decreases both mineral volume and the double layer thickness to decrease cation exchange capacity are also discussed in Chapter 7. In the extreme, all above phenomena may cause volumetric shrinkage, which could lead to cracking and increased hydraulic conductivity as it undergoes chemical and mineralogical change.

# 8.2.2 Concentration of Chemicals

The concentration of chemicals plays an important role in the hydraulic conductivity of compacted clays. As can be seen from the Figure 8.1(a), the compacted clays permeated with the leachate containing up to 70% dioxane produced significant decreases in hydraulic conductivity due to the viscosity effect. Leachates containing

85 to 100% dioxane experienced two to three orders of magnitude increase in hydraulic conductivity and the flow was controlled by the decrease in double layer thickness. This was so for the zero effective stress, which is the case for compaction mould permeameter. This increasing trend was found to be reduced by the increase of effective stress and as could be seen from Figure 8.1(d), the increase is almost insignificant at very high confining effective stress. The similar observations were also made by other investigators (*eg.* Fernandez and Quigley, 1988; Fernandez and Quigley, 1991).

Fernandez and Quigley (1988) reported substantial increases in viscosity as the organics increased to a concentration of 50-60%. They also found a large decrease in dielectric constant as the percentage of soluble organics increased from 0 to 100%. Effect of the dielectric constant and the viscosity on double layer and on the hydraulic conductivity was discussed previously. Although concentrated organic chemicals are unlikely to be present in the waste leachate, still there could be significant decreases in dielectric constant of the leachate compared to that of water. The large increases in hydraulic conductivity at concentrations above approximately 70% were explained in terms of the diffused double layer contraction as the low dielectric constant organics entered the micelle structure causing an increase in void space at constant void ratio (Fernandez and Quigley, 1991).

#### 8.2.3 Polarity

Mesri and Olson (1971) found a large increase in hydraulic conductivity when organic fluids other than water were used for permeation in testing. They suggested that the difference in hydraulic conductivity could be due to the differences in polarity between water and organic permeants. The reason could be that the water molecules, with a large dipole moment, are strongly bonded with hydrogen bonds to the surface of the silicate minerals in a soil. In clay rich soils, which have small pores and often contain high surface area minerals, the greater part of the pore fluid exists in an adsorbed film held in place by intermolecular attractions of various kinds, as



Figure 8.1: Hydraulic Conductivity of Water Compacted Clay Permeated with Municipal Solid Waste Leachate Ethanol Mixture at Effective Vertical Stresses : (a) 0 kPa, (b) 40 kPa, (c) 80 kPa, and (d) 160 kPa (source Fernandez and Quigley, 1991).

discussed later. The existence of an adsorbed layer of water on the surfaces of the soil particles effectively reduces the size of the pore channels, decreasing the ability of the fluid to move through the soil. Many organic liquids having smaller polarities than water are less strongly bonded to the soil minerals and fluid flows more easily through the pore network.

#### 8.2.4 Stabilising and Destabilising Forces

Interacting forces between clay particles were already discussed in Chapter 2. These forces in a clay matrix are responsible for a stable structure in clay. Stabilising forces include high cementation, overconsolidation, high electric forces, stable atomic structure, less surface area, high effective stresses, and suction etc. Destabilising forces come from double layer collapse, and desiccation etc. The physical and physico-chemical interaction of organic chemicals present in waste leachates may significantly influence these forces, resulting an unstable soil structure. This could also lead to substantial changes in hydraulic behaviour of the clay soils.

#### 8.2.5 Change of Fabric

Fabric of the compacted clay was discussed in Chapter 2. The organic fluids interacting with the soil mineral causes the soil to change its fabric in a manner that provides an easier passage for the pore fluid (Budhu et al, 1991). Gilligan (1983) and Gilligan and Clemance (1984) suggested that the addition of organic liquids to clay rich soils cause the formation of tactoids in these soils. Tactoids were described as the spindle shaped agglomerations of clay minerals that are dispersed in the soil matrix. The formation of tactoids in a soil created a more granular fabric with enlarged pore spaces and an easier flow passage. They found that illite has the smallest polarising effect on organic molecules with kaolinite and montmorillonite having a nearly identical effect. The polarising ability controls the distance between

the particles in a face to face fabric, which is essential for tactoid formation. Thus one would conclude that the response of different clay minerals to different organic fluids would be dramatically different.

Foreman and Daniel (1986) found that there was a significant effect of effective stress on the fabric of the soil when methanol was used as a permeant. At low effective stress they found that the fabric of kaolinite soil could apparently change producing macropores and large increases in hydraulic conductivity whereas high effective stresses seem to prevent the development of macropores and deleterious alteration in the soil fabric.

# 8.2.6 Flocculation and Dispersion Effect

As discussed in Chapter 7, clay particles flocculate at low pH environment. Leachates, acids and bases may have a wide range of pH values, which could interact with clay particles leading to flocculation or dispersion of the soil matrix. Gibson (1985) reported that the hydraulic conductivity of mixtures of silty sand with 7.5, 10 and 15 % bentonite increased by 7, 13 and 41 times respectively after permeation with acid liquor at a very low pH value of 2.2. D'Appolonia (1980) noted that the sand bentonite mixtures with low bentonite contents (<3%), a low permeant pH may lead to smaller clay particles due to the flocculation effect, which are then flushed or piped from the soil resulting in larger pore channels and an increase in hydraulic conductivity. Lentz *et al.* (1985) found 2.5 to 13 times decrease in hydraulic and blockage of the flow paths, and ion exchange resulting from the increase of double layer thickness etc.

Storey and Peirce (1989) concluded from settling tests that almost complete flocculation of the clay particles occurs at 80 and 100 % methanol concentration. They attributed this effect to the reduction in the repulsive forces between particles giving rise to a decrease in the double layer thickness. The conclusions reached were

that the decrease in the double layer around the soil particle caused by the lower dielectric constant results in flocculation leading to the possible formation of macropores. This effect, in fact, gives rise to shrinkage away from the side walls in rigid wall permeameters and was responsible for large increases in hydraulic conductivity observed.

#### 8.2.7 Adsorption and Intercalation

Some of the organic molecules are adsorbed in to the clay layer replacing the water molecules in the double layer. The bond strength between the double layer cations varies widely (Lagaly, 1984). The lower dielectric constant of the most organic compounds compared to water may limit swelling or induce a tendency towards flocculation if the particles are not restricted from movement. As stated earlier, these changes in the clay structure might change the hydraulic conductivity of the liner material.

Intercalation is the process where the molecules of the permeants enter the interlayer spaces and split apart the unit layers. The alkali salts of short chain fatty acids, in particular, acetic and propionic acid are of interest (Mitchell and Madsen, 1987). For example, ammonium acetates would change the basal spacing of kaolinite from about 7 to 14 Å. The resulting swelling of kaolinite particles should cause a decrease in the hydraulic conductivity.

# 8.2.8 Dissolution

There is a general agreement by the investigators (*eg.* D'Appolonia 1980; Gordon & Foster 1981; Nasiastka *et al.* 1981; Relyea & Martin 1982; Gibson 1985; Lentz *et al.* 1985; and Peterson & Gee 1985) that strong acids and bases can disintegrate clay minerals into a solution changing the properties of the soil. The solubility of a clay

mineral depends on the nature and concentration of the acid or base and the temperature and the duration of the exposure of these chemicals to clay minerals (Grim, 1968). If dissolution occurs in a soil due to strong acids or bases and flushing or piping washes out the disintegrated particles, a large increase in hydraulic conductivity could be observed. On the other hand, if the disintegrated particles precipitate in the soil matrix, a reduction in hydraulic conductivity may be observed. Dissolution can pose a long-term problem for clay liners. It is, therefore, essential to carry out long term investigation of the liner material in its expected state.

D'Appolonia (1980) stated that strong acids at low pH dissolve the clay minerals and with subsequent increases of solution pH increase possibility for the reprecipitation of the clay particles. This process continues and when the pH of the pore liquid approaches the pH of the influent after permeation of a large volume of acids. This eventually will result in dissolution and flushing of the reprecipitated solids, larger pore channels and an increase in hydraulic conductivity (Bowders et al. 1991). Precipitation effect was confirmed by Gordon and Forrest (1981) after the permeation was made with H<sub>2</sub>SO<sub>4</sub>. They confirmed that the precipitation of CaSO<sub>4</sub> was indicated by: (1) a slight weight gain in soil; (2) decrease in effluent electrical conductance (EC) and; (3) an increase in effluent pH. They also found an increase in volumetric outflow relative to volumetric inflow in the hydraulic conductivity tests to the production of water. Anderson (1982) and Anderson et al. (1985) concluded from the test results that with acetic acid the particles are restrained from piping due to a sieving effect (*ie.* similar to a filter paper), thus hydraulic conductivity may decrease. This interaction appears to be significant particularly in the case of bentonite filter cakes and soil bentonite backfill materials when exposed to extremely acidic or basic permeants pH<2 or pH>11. D'Appolonia (1980) noted that extremely basic solutions usually produce a greater increase in hydraulic conductivity relative to extremely acid solutions since amorphous silica is quite soluble in highly alkaline solutions.

Bowder and Daniel (1987) observed a decrease in hydraulic conductivity after tests were carried out in RWP through kaolinite soil with acetic acid as the permeant. These decreases could have been attributed to the dissolution of some of the soil constituents at the influent end of the specimen followed by the precipitation of these constituents further down stream. As was discussed in Chapter 4, the volume changes might have been partially offset by the confining pressure, which would tend to compress the soil and close any voids created by dissolutioning. However, due to the flocculated soil structure, it is unlikely that such voids would be completely compensated for by compression giving rise to a noticeable increase in hydraulic conductivity.

# 8.3 **RESULTS OF HYDRAULIC CONDUCTIVITY TESTS**

#### 8.3.1 Hydraulic Conductivity with Waste Leachate

A series of hydraulic conductivity tests were carried out with leachate using FWPs, compaction mould permeameter and consolidation cell permeameter. The testing schedule is presented in Table 8.1. As stated before, leachate samples collected from PWM Lyndhurst site were used for the testing. The collection of leachate sample, centrifugation and the preservation techniques were already discussed in Chapter 3. Initially filtered leachate was used as a permeant for hydraulic conductivity testing. Filtering with filter papers did not clear all the colloidal particles and some of the particles were visible, even to the naked eye, in the leachate sample, even though a very fine filter paper (Whatman 55) was used. Another problem with the filtering technique was the requirement of an enormous amount of time. Although it seems that the visible colloidal particles were removed from the leachate sample after centrifugation, still centrifuged leachate were reasonably thick and a little dark in colour. The presence of dispersed colloidal particles were also confirmed by the presence of very high turbidity value of the leachate, more than 450 (NTU) compared to 1 (NTU) for the tap water. The specimen thickness used for FWPs, consolidation cell permeameter and the compaction mould permeameter are 50, 30 and 20 mm respectively.

Tests WL1 and WL2 were carried out with filtered leachate and the other three tests EL1, EL2 and WL3 were carried out with centrifuged leachate. These test results are presented in Figures 8.2 to 8.6 and are summarised in Table 8.1. Although, as could be seen from the Table 8.1 that in only one case (EL2) a very insignificant increase in hydraulic conductivity was observed, the general trend is the decrease in hydraulic conductivity for both FWP and consolidation cell permeameter after the introduction of leachate. As can be seen from the test results in Figures 8.2 to 8.5, a significant decrease in hydraulic conductivity was observed for both FWP and consolidation cell permeameter after the introduction of waste leachate. This decrease in hydraulic conductivity would probably be due to the presence of dispersed colloidal particles, as discussed previously, which already existed in the leachate sample. Slow seepage of this leachate into the clay may cause clogging of the pore spaces, resulting in a decrease in the clay hydraulic conductivity. Again, in cases of FWPs and consolidation cell permeameter, leachate samples had to pass through porous stone and filter paper before entering into the clay specimen. Colloidal particles might clog the pores of porous stones and filter papers resulting a drop of pressure head between the inflow and outflow, which could affect hydraulic gradient as well as the hydraulic conductivity. Another reason could be due to the increase in effective stress due to chemicals that cause a decrease in hydraulic conductivity due to the consolidation (shrinkage) effect. This aspect was already discussed in Chapter 7. These results are consistent with the observation made by other authors (Cartwright et al., 1977; Daniel and Liljestrand, 1984; Brunelle et al., 1987; Fernandez, 1989). Sai and Anderson (1991) tested the compacted clay liners with waste leachate for more than two years and no substantial increase in hydraulic conductivity was observed.

Experimental investigation carried out with the compaction mould permeameter is presented in Test WL3. The results are also presented in Figure 8.6. Hydraulic conductivity values were calculated using the flow through the inner half of the specimen. As can be seen from Figure 8.6, there is more than 2 times increase in hydraulic conductivity of compacted clay when the leachate was used as a permeant. Despite the fact that low effective stress may have been present in the test, this result

suggest that it is still possible for structural changes to occur with leachate. As discussed previously this conflicting result may be a matter of concern. Although the increase was not as significant as in the other cases (*ie.* few orders of magnitude increase), still there are possibilities for the structural changes to occur. This issue is further examined in the new technique proposed in Chapter 9.

Some leachates may comprise either the concentrated acids or bases. As discussed in the literature review in previous sections, strong acids and bases may significantly influence the hydraulic conductivity. There is some evidence to suggest that these have little effect on the liner, but on the other hand it is also argued that acids and bases could cause dissolution of particles which are then able to migrate. If the migration causes clogging then, as noted above, there will be a decrease in hydraulic conductivity. But if the results are in dispersion, or at the limit piping, then the hydraulic conductivity will increase. This was investigated in the testing program by modifying the leachate at various concentrations of NaCl and the results are presented in the Section 8.3.3.

As discussed in Chapters 4 and 7, the volume change in these tests could also be considered to express the possible changes that might occur as a result of the introduction of waste leachate. As can be seen from Figures 8.2 to 8.4, volume changes in different tests were not consistent but the overall picture is that the sample volume shrinks after the application of leachate sample. Initially the shrinkage is at a rapid rate showing the increases in the cell volume and eventually reducing to a constant cell volume. In all the cases, there is a decrease in the (outflow-inflow) volumes after the introduction of leachate. As the inflow is higher than the outflow, the void spaces within the specimen should have increased instead of decreasing by the effective stress. This again signifies the changes in the soil structure within the specimen. This is similar to the discussion made in Chapter 7 about the volume changes. As is shown in the Figures 8.5(a) and 8.5(b), no significant differences in the inflow and outflow hydraulic conductivities were observed for consolidation cell permeameter.



**Figure 8.2:** Effect of Waste Leachate on Hydraulic Conductivity of Echuca Soil Using FWP



**Figure 8.3:** Effect of Waste Leachate on Hydraulic Conductivity of Werribee Soil Using FWP

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Pore Volume

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Figure 8.4: Effect of Waste Leachate on Hydraulic Conductivity of Werribee Soil Using FWP



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(Outflow)

Figure 8.5: Effect of Waste Leachate on Hydraulic Conductivity of Echuca Soil Using Consolidation Cell Permeameter



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Figure 8.6: Effect of Waste Leachate on Hydraulic Conductivity of Werribee Soil Using Compaction Mould Permeameter

Permeameter	FWP	FWP	FWP	Consolidat-	Compaction
				ion cell	mould
Soil	Werribee	Werribee	Echuca	Echuca	Werribee
Test number	WL1	WL2	EL1	EL2	WL3
Effective	100	100	100	100	-
stress (kPa)					
Hyd. Gradient	200	200	200	100	200
$k_{\rm w} \ge 10^{-11}$	4.38	6.44	12.90	5.12	4.26
(m/s)					
k <sub>ch</sub> (min) x	-		-	2.40	-
10 <sup>-11</sup> (m/s)					
k <sub>ch</sub> (max) x		_	-	2.93	9.80
10 <sup>-11</sup> (m/s)					
k <sub>ch</sub> (final) x	2.62	3.40	6.25	2.68	8.53
10 <sup>-11</sup> (m/s)					
pv	2.00	1.50	3.50	1.95	3.00

**Table 8.1:** Effect of Landfill Leachate on Hydraulic Conductivity of Compacted Clay

# 8.3.2 Hydraulic Conductivity with Methanol

The threshold concentration, *ie.* the concentration at which the structural changes as well as the changes in hydraulic conductivity start occurring, of NaCl was established in the previous chapter. The problems related to the testing with the concentrations higher than threshold concentration of NaCl as permeant were also discussed in the previous chapter. Although, it is highly unlikely that the methanol at high concentration could be present in the leachate, the intention was to examine the likely effect of high concentrations of methanol at 60% and 80%. These tests were carried out using FWPs and compaction mould permeameter. These test results are presented in Figures 8.7 to 8.13 and the results are tabulated in Tables 8.2(a) and 8.2(b). Two types of soils were used and some of the tests were repeated in the case of FWPs to

confirm the hydraulic behaviour of the soil with high concentration of methanol.

Investigators have put forward a range of arguments regarding the effect of methanol on the hydraulic conductivity of compacted clays. Uppot and Stephenson, (1989) found a decrease in hydraulic conductivity during the first pore volume of methanol permeation when the test was performed in flexible wall permeameter. They argued that the reason could be due to the energy spent in replacing water by methanol in the adsorbed layer. Mitchell and Madsen, (1987) concluded that concentrated organic chemicals may cause a change in the ionic bonding conditions in the clay minerals, modifying the diffuse double layer around each clay particle and causing the particle to draw closer together. They also argued that the shrinkage of this nature could increase the hydraulic conductivity by several orders of magnitude. This was probably the case for compaction mould permeameter where side wall leakage was the contributing factor in such a situation. But for flexible wall permeameter this volume shrinkage will be masked by the effective stress used in the testing. Mitchell (1976) stated that ion exchange reactions with inorganic and organic cations in a new permeant as well as adsorption of organic cations and neutral organic polar molecules in place of previously adsorbed water molecules, will change the thickness and composition of the adsorbed layer. Influx of non-ionic water might leach the existing cations, decrease the ionic strength of the pore fluid and swell the double layer. For the clay minerals with an expanding lattice, such as montmorillonite, these reactions will also take place in the interlayer spaces and cause the individual particles to swell and shrink.

Test results with compaction mould permeameter are presented in Figures 8.7 to 8.8. As can be seen from Table 8.2(a) for Test EM1, more than two orders of magnitude increase in the hydraulic conductivity, compared to the base value established with water as a permeant, was observed when 60% methanol was used as a permeant in compaction mould permeameter. The reason for such increase in the hydraulic conductivity could be due to the decrease in double layer thickness after the permeation of water/clay mixture with methanol. Concentrated methanol has a dielectric constant of 32 compared to 80 for water. Even at 60% and 80%

concentrations, methanol has a much lower dielectric constant compared to that of water. As discussed in Chapter 2, this decrease in the dielectric constant should have the potential to decrease the double layer thickness, resulting the soil particles to flocculating or forming aggregates. In the absence of applied effective stress in CMP, structural changes or cracks can form after methanol permeation. This may lead to a substantial increase in the hydraulic conductivity. This could also increase side wall leakage due to large open pores between the fixed wall and the soil specimen, resulting in a large increase in hydraulic conductivity. This issue will be looked at with the proposed new technique of testing, by applying the no lateral strain boundary condition, as discussed in Chapter 9. Visual observation of the specimen after the hydraulic conductivity testing also revealed the formation of hair cracks and side wall leakage in compaction mould. The cracking was distinct due to the change of colour along the cracks. This is shown in Figure 9.7. Anderson *et al.* (1985) also found large pores and visible cracks on the soil surface after the test was carried out with concentrated methanol as a permeant.

Tests with flexible wall permeameter were carried out with 60% and 80% methanol and the results are presented in Figures 8.9 to 8.13 and are tabulated in Table 8.2(b). In almost all cases, increase in hydraulic conductivity was observed from their lowest minimum values, although the increase was very small (<2 times). But the final hydraulic conductivity values are substantially reduced. This could again probably be due to the effect of effective stress as discussed in the previous chapter. Although the structural changes may have occurred, due to low dielectric constant of methanol, resulting shrinkage effects seemed to be masked with a consequent decrease in the hydraulic conductivity. It seems that the two effects, namely shrinkage and fabric (structural) changes, are taking place concurrently. This may be the reason for transient variation in the hydraulic conductivity values presented in the figures. Visual observation also did not show any discernible cracks or changes in the specimen after the tests were completed.

As could be seen from Tables 8.2(a) and 8.2(b), the test results between the two parmeameters vary significantly. The reasons for that have already been discussed

previously. This is also in agreement with the other investigators. Foreman and Daniel (1986) conducted hydraulic conductivity tests on kaolinite soil using both compaction mould permeameter and flexible wall permeameter and they found that the compaction mould permeameters yielded hydraulic conductivities that are 10 times higher than those obtained from flexible wall permeameters. Bowder and Daniel (1987) observed more than 20 times difference in hydraulic conductivity when concentrated methanol was used as a permeant in these two types of permeameters. Tests WM3 and WM4 were undertaken under identical conditions and the results were found repeatability.

If the volume change is taken into consideration, it could be seen that the rate of volume increase is rapid initially and eventually reduced to a constant cell volume. From the (outflow-inflow) results it can be seen that the inflow is higher than the outflow, and initially the testing cell volume is also increased indicating a sample volume decrease or consolidation. The reasons are already discussed in Chapter 7.

Soil	Echuca	Werribee
Test number	EMI	WM1
Effective stress	0	0
Hyd. Gradient	200	200
Concentration of	60	60
methanol (%)		
$k_{\rm w} \ge 10^{-11}  ({\rm m/s})$	3.96	3.17
k <sub>ch</sub> max x 10 <sup>-11</sup> (m/s)	>511.00	241.00
k <sub>ch</sub> final x 10 <sup>-11</sup> (m/s)	36.40	241.00
pv	13.00	3.4

 Table 8.2(a): Effect of Methanol on Hydraulic Conductivity of Compacted Clay

 Using Compaction Mould Permeameter



**Figure 8.7:** Effect of 60% Methanol on Hydraulic Conductivity of Echuca Soil Using Compaction Mould Permeameter



Figure 8.8: Effect of 60% Methanol on Hydraulic Conductivity of Werribee Soil Using Compaction Mould Permeameter



Figure 8.9: Effect of 60% Methanol on Hydraulic Conductivity of Echuca Soil Using FWP



Pore Volume

Figure 8.10: Effect of 60% Methanol on Hydraulic Conductivity of Werribee Soil Using FWP



Pore Volume

Figure 8.11: Effect of 80% Methanol on Hydraulic Conductivity of Echuca Soil Using FWP


Pore Volume





Figure 8.13: Effect of 80% Methanol on Hydraulic Conductivity of Werribee Soil Using FWP

Soil	Echuca	Werribee	Echuca	Werribee	Werribee
Test number	EM2	WM2	EM3	WM3	WM4
Effective stress	100	100	100	100	100
Hyd. Gradient	200	200	200	200	200
Concentration	60	60	80	80	80
of methanol (%)					
k <sub>w</sub> x 10 <sup>-11</sup> (m/s)	6.69	4.01	2.62	6.67	5.86
k <sub>ch</sub> (min) x	2.99	-	1.36	2.91	3.24
10 <sup>-11</sup> (m/s)					
k <sub>ch</sub> (max)	5.05	-	1.56	3.03	3.68
x10 <sup>-12</sup> (m/s)					
k <sub>ch</sub> (final) x	2.13	2.03	1.53	2.90	3.45
10 <sup>-11</sup> (m/s)					
pv	4.00	3.3	2.00	1.75	2.00

 Table 8.2(b): Effect of Methanol on Hydraulic Conductivity of Compacted Clay

 Using Flexible Wall Permeameter

# 8.3.3 Hydraulic Conductivity with Modified Leachate

Hydraulic conductivity tests were carried out with modified leachate using FWPs and consolidation cell permeameters. Leachate modified with different concentrations of NaCl was used for such testing. Test results presented in Section 8.3.1 for landfill leachate showed no increase in hydraulic conductivity after leachate permeation except in the compaction mould permeameter. The idea of modifying leachate was to see how the varying concentration of different chemicals in leachate affects hydraulic conductivity. In the real field situation, leachate concentration can vary significantly. Hence, these tests were designed to examine the likely influence on hydraulic conductivity due to variation in leachate salinity and other chemical concentrations.

Tests carried out for Echuca and Werribee soils with consolidation cell permeameter

using the modified waste leachate (leachate modified with 2M NaCl) as permeants. are presented in Figures 8.14 and 8.15 respectively. As can be seen from the results presented in Table 8.3, the hydraulic conductivity of the test EML2 increased more than 5 times, whereas a very minimal increase in hydraulic conductivity was observed in the case of WML2. Although two types of soil were used in these two tests, the only difference between the two tests is the effective stress. Tests EML2 and WML2 were carried out at 64 kPa and 100 kPa effective stress. The difference between 5 times and a few percentage increases could have been contributed by these effective stress differences. At low effective stress the soil specimen has more flexibility for the structural changes to occur that could substantially increase hydraulic Side wall leakage, which is the major problem in rigid wall conductivity. permeameter, is also important at low effective stress. Results presented in Figures 8.14 to 8.15 were carried out with FWPs where leachate modified with 1M NaCl was used as permeant. Results of these investigations are also tabulated in Table 8.3. Both of these two tests, EML1 and WML1, present a significant reduction in hydraulic conductivity after the introduction of the modified waste leachate. As could be seen from Section 8.3.1, a constant reduction in hydraulic conductivity was observed when leachate was used as a permeant. Permeation of 1M NaCl caused a 30% increase in hydraulic conductivity (as in Chapter 7), but the combined effect is the overall reduction in hydraulic conductivity.

As can be seen from Figures 8.14 and 8.15, the cell volume increases initially followed by a constant cell volume at the end. This is also similar to the results observed in Chapter 7. (Outflow- inflow) volumes also show a significant difference, which is also an indication of the structural changes that could have happened within the specimen.



**Figure 8.14:** Effect of Modified Leachate (Leachate + 1M NaCl) on Hydraulic Conductivity of Echuca Soil Using FWP



**Figure 8.15:** Effect of Modified Leachate (Leachate + 1M NaCl) on Hydraulic Conductivity of Werribee Soil Using FWP



(Inflow)



(Outflow)

**Figure 8.16:** Effect of Modified Leachate (Leachate + 2M NaCl) on Hydraulic Conductivity of Echuca Soil Using Consolidation Cell Permeameter





Figure 8.17: Effect of Modified Leachate (Leachate + 2M NaCl) on Hydraulic Conductivity of Werribee Soil Using Consolidation Cell Permeameter

Permeameter	FWP	FWP	Consolidat-	Consolidat-	
			ion cell	ion cell	
Soil	Echuca	Werribee	Echuca	Werribee	
Test number	EML1	WML1	EML2	WML2	
Effective stress	100	100	64	100	
Hyd gradient	200	200	60	133	
Influent	Leachate + 1M	Leachate + 1M	Leachate +	Leachate +	
	NaCl	NaCl	2M NaCl	2M NaCl	
k <sub>w</sub> x 10 <sup>-11</sup> (m/s)	7.35	3.93	6.12	9.48	
k <sub>ch</sub> (min) x	4.57	2.72		9.14	
10 <sup>-11</sup> (m/s)					
k <sub>ch</sub> max x	4.98	2.83	33.30	9.84	
10 <sup>-11</sup> (m/s)					
k <sub>ch</sub> final x	4.60	2.47	18.50	9.37	
10 <sup>-11</sup> (m/s)					
pv	2.20	2.00	2.86	1.40	

 Table 8.3: Effect of Modified Leachate on Hydraulic Conductivity of Compacted

#### Clay

# 8.4 CONCLUDING REMARKS

Hydraulic conductivity tests were carried out with landfill leachate, methanol and modified leachate as permeants on the local soils used as liner materials. These testing were undertaken using consolidation cell permeameter, compaction mould permeameter and flexible wall permeameter and the results were presented and discussed in different sections.

Leachate collected from PWM Lyndhurst site was filtered and centrifuged before using as a permeant for testing. From both FWPs and consolidation cell permeameters, a significant reduction in hydraulic conductivity was observed when leachate was used as a permeant. The possible blockage of the flow path due to the presence of colloidal particles in the leachate and the effect of effective stress could be the two possible reasons for such decrease. Volume changes in terms of cell volume and the difference in (outflow-inflow) volumes revealed the possible shrinkage and structural changes with in the specimen. The test carried out with compaction mould permeameter showed more than 2 times increase in hydraulic conductivity.

Methanol concentrations of 60% and 80% were used as permeants for compaction mould permeameter and FWPs. In case of compaction mould permeameter more than two orders of magnitude increase in hydraulic conductivity was observed when methanol at 60% concentration was used as a permeant. Visible cracks in the specimen and side wall gaps were observed after the test was finished. For FWPs a gradual decrease in hydraulic conductivity was observed with the passage of time after methanol was introduced as a permeant. In the case of methanol, similar volume changes and (outflow-inflow) volume change was observed as was found in the case of leachate.

Leachate modified with 1M NaCl and 2M NaCl solutions were used as permeants. Consolidation cell permeameter and FWPs were used to carry out hydraulic conductivity testing. In the case of FWPs, a substantial decrease in hydraulic conductivity was observed with time. As in the case of the other two permeants similar types of volume changes were observed. In cases of consolidation cell permeameter a very little increase in hydraulic conductivity was observed at a high effective stress (100kPa), but at an effective stress of 64 kPa which is close to preconsolidation pressure, a 5 times increase in hydraulic conductivity was observed.

As discussed in Chapter 7, similar problems were observed for different types of permeameters. In flexible wall permeameter, the soil specimen undergoes a volume change due to the applied and the induced effective stresses while in the compaction mould and consolidation cell permeameter, a side wall leakage can occur giving

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unlikely results. Improvement of the testing technique is required to reduce or eliminate these problems. A new testing technique is proposed in Chapter 9, which will be free from these problems and simulate, more realistically, the field condition.

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# 9 A NEW APPROACH TO COMPATIBILITY TESTING USING HYDRAULIC CONDUCTIVITY MEASUREMENT

## 9.1 INTRODUCTION

In the previous chapters, it was identified that low hydraulic conductivity is the most desirable property for a compacted clay liner. The problems related to the hydraulic conductivity testing using the traditional equipment are well recognised by the investigators in this field. The problems observed while carrying out the testing for this research work were already discussed in Chapters 7 and 8. In summary, these problems are side wall leakage related to the rigid wall permeameters, and the effective stress consolidation that happens in flexible wall permeameters. Although these problems are well recognised, still a very minimal attention was given to overcome them. Nevertheless, these attempts are also not adequate and further improvement is warranted in this regard. A new testing technique, using the field boundary conditions with lateral restraint was proposed and validation of this technique was also carried out in the laboratory. NaCl, methanol and leachate with varying concentrations were used to carry out hydraulic conductivity testing under the new boundary conditions, and the results are presented in this chapter.

As was found in the previous chapters, the volume changes during testing plays a significant role in the hydraulic behaviour of compacted specimens. These volume changes were attributed to the effective stresses generated by chemicals during testing, which can mask any structural changes that can happen due to chemical permeation. Chapter 7 explained the development of effective stresses in the specimen during NaCl permeation. Similar behaviour was expected with other chemicals. If there are differences in the applied and the generated effective stresses, specimen volume changes could also occur leading to variation in the hydraulic conductivity.

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It is well known that residual stresses are developed after the compaction of granular fills next to non-yielding retaining walls. If similar residual stresses are developed during the compaction of a liner, then those stresses will also influence the subsequent behaviour of clay liners to chemical stresses. There is no data available on the residual stresses of compacted clay under lining conditions. In the current chapter this issue is theoretically examined in some detail. The assessment of the residual compaction stresses using FLAC (Fast Lagrangian Algorithm of Continuua) and semi-analytical model of Filz and Duncan (computer program EPCOMPAC) were carried out. The influence of these residual compaction stresses on cracking was investigated. The evaluation of crack formation due to desiccation, and structural changes and cracking due to chemicals were also investigated.

## 9.2 IDENTIFICATION OF PROBLEMS

As discussed in the previous chapters, a main intention of the research program was to undertake the laboratory investigation of the hydraulic conductivity of some selected Victorian soils used for liner construction. These investigations were carried out in the laboratory using rigid and the flexible wall permeameters. The main problem concerning the rigid wall permeameter is the side wall leakage and it was found that this problem is severe in cases where the permeating fluid is other than water. If specimen shrinkage does occur due to the physico-chemical interaction, there is no adhesive force to hold the soil specimen with the wall and consequently, a large increase in hydraulic conductivity could result. Although the use of the double and triple ring technique could help in detecting side wall leakage, nevertheless, the problem is not addressed.

The main problem concerning the flexible wall permeameter is associated with the effective stress consolidation. Because of the flexible membranes at the boundary, the volume shrinkage due to chemical stresses is unrestrained. As a consequence, the large increases in the hydraulic conductivity in RWPs were not observed in FWPs. These two issues highlight the inadequacy of current methods of compatibility testing. These problems basically stem from the direct adaptation of the testing techniques that were

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used primarily for strength testing to hydraulic compatibility testing. In strength testing, the main focus is to simulate uniform stress conditions in a soil specimen to assess the constitutive and failure criteria. In contrast, the hydraulic compatibility testing should focus on simulating the field boundary conditions in order to facilitate the likely structural changes that affect the hydraulic conductivity.

#### 9.2.1 Boundary Conditions for Testing

Flexible wall permeameters allow the lateral consolidation or swelling to occur because of the flexible rubber membrane and the constant cell pressure used around the specimen. The consolidation is supposed to occur due to the effective stress applied to the specimen during testing. As was discussed in Chapter 7, the permeation of NaCl may lead to an increase in net effective stress and a net increase in pore volume may take place. If the developed effective stress is higher/lower than the effective stress applied in testing, specimen consolidation /swelling might occur.

Rigid wall permeameters approximate the field boundary condition of no lateral strain but only under compressive lateral stresses. If the lateral stresses become tensile due to soil shrinkage, then specimen can break away from the side wall leading to side wall leakage. This boundary contact problem leads to the side wall flow and is referred to as the side wall leakage. The results presented in Chapter 7 indicated that the side wall leakage was higher for permeants having a lower dielectric constant than water.

### 9.2.2 Previous Research

Shuai (1996) used the Rigid Wall Triaxial Permeameter developed by Huang (1994) for the measurement of hydraulic conductivity and the test setup is shown in Figure 9.1. In his technique, a stainless steel sampling tube was used as a mould for the triaxial



Figure 9.1: General Assembly of the Modified Triaxial Permeameter Cell (source: Huang, 1994)

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permeameter. Due to this improvement, there could be few advantages over the traditional method. The intention was to minimise the lateral specimen consolidation, which is the common problem for the flexible wall permeameter. The application of effective stress is still possible in this technique through the top and bottom of the specimen. Although this technique may present some advantages over the traditional technique, it still suffers from the side wall leakage problem.

Mitchell (1993) developed a flexible, no lateral strain centrifuge test cell to evaluate clayliner permeant compatibility under prototype effective stress conditions. The test cell is shown in Figure 9.2. In this technique, centrifugal acceleration was used to increase gradient rather than using elevated permeant head pressures. According to the author, both the boundary conditions of rigid and the flexible wall permeameters as well as the actual field effective stress regime can be approximated in the centrifuge model. Centrifugal acceleration acting on a fluid standpipe oriented along the centrifuge radius causes a proportional fluid pressure gradient in that standpipe. By allowing pressures generated in a silicon oil standpipe to act on a latex rubber membrane laterally confining a clay liner sample, and maintaining a constant oil volume in the surrounding cell, a no lateral strain boundary condition was created. Although the author considered the result to be reasonable in hydraulic conductivity on a small scale, still further study in larger equipment was recommended. Furthermore, the method relies on the physical behaviour of oil (incompressibility, surface tension) to indirectly apply the zero lateral strain condition. The testing technique is relatively complicated and is not available to routine compatibility testing.

# 9.3 THEORETICAL ANALYSIS OF COMPACTION STRESSES AND CRACKING

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As discussed in the previous chapters, the changes in clay structure, which may take place after the compaction of the liner, can lead to large increases in the hydraulic conductivity. One such situation is when the liner undergoes desiccation due to moisture loss. This could happen if the liner is left uncovered during construction or the cover material is not sufficiently thick to prevent moisture evaporation. Liners used as landfill caps are more

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Figure 9.2: Schematic Diagram of Flexible Wall No Lateral Strain Cell (source: Mitchell, 1993)

susceptible to evaporative moisture loss because they generally have a relatively small amount of cover above them. Base liners could suffer desiccation if the temperature at the top of the liner rises above that of the bottom owing to high temperatures that could develop within some waste fills (Jessberger, 1995).

Currently, experimental methods are predominantly used to examine the likelihood of the above clay structure changes. Reliable theoretical methods are not readily available mainly because of the complexity of these mechanisms and their dependence on many interacting variables. However, moisture losses during desiccation can be examined relatively accurately using unsaturated flow theory, provided reliable input parameters are available. But relating these moisture losses to the initiation and extent of cracking is not straightforward. Jessberger (1995) has examined some of the influencing variables on the basis of a theoretical investigation of desiccation cracking in clayey soils given by Morris et al. (1992). In the model presented by Morris et al., cracking occurs when the tensile horizontal stresses created by matric suction exceed the compressive stresses due to overburden pressures in excess of the tensile strength of the clayey soils. While this approach appears to be reasonable for cracking of natural or loosely dumped clayey soil deposits (eg. mine tailings), for compacted clayey soils the residual stresses generated during the compaction process should also have some inhibitive effect on the crack formation. Previous investigations have shown that significant residual horizontal stresses can develop when soils are compacted adjacent to rigid retaining walls (eg. Clayton and Symons, 1992; Filz and Duncan, 1992). Although direct experimental evidence is not readily available, it can be expected that substantial residual stresses would also develop during clayey liner compaction and these need to be accounted for in theoretical stress analyses.

In this study, simplified theoretical models for structural changes due to desiccation and chemical effects are developed. Residual compaction stresses are modelled using the semi-analytical method developed by Duncan and Seed (1986) and subsequently modified by Filz and Duncan (1992). The model presented by Morris et al (1992) is extended to account for residual compaction stresses and is then used to examine desiccation cracking. An attempt was also made to use the same model to predict the

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likelihood of structural changes in clay due to chemicals using the double layer theories of clay chemistry.

### 9.3.1 Modelling of Residual Compaction Stresses

#### • Modelling with continuum mechanics

An attempt was made to model the compaction process using continuum mechanics. FLAC (Fast Lagrangian Algorithm of Continuua, Itasca, 1993), an explicit finite difference code, was used for the modelling. A one dimensional soil column with lateral restraints (*ie.* uniaxial *or*  $K_0$  conditions) was subjected to monotonically increasing strains applied vertically downwards at the top surface, representing the application of external compaction loading. These boundary conditions are generally considered applicable to soil compaction in the laboratory and are assumed to approximate the field condition, too. Subsequently, the direction of vertical strains was reversed and the soil column was relieved from external compaction loading. The stress paths followed by vertical and horizontal stresses were monitored during this simulation.

Firstly, a conventional elasto-plastic model with Mohr-Coulomb shear failure criterion (*ie.*  $c - \phi$  soil) was used. The initial principal stress system included gravity vertical stresses ( $\sigma_v$ ) and horizontal stresses ( $\sigma_h$ ) given by  $K_0\sigma_v$  where  $K_0$  is given by the well known expression of  $(1 - \sin \phi)$ . The model showed that initially, the horizontal and vertical stresses increase elastically so that their incremental ratio ( $\Delta \sigma_h / \Delta \sigma_x$ ) is given by:  $K_e = v/(1 - v)$  where v is Poisson's ratio of the soil. If the coefficient of active earth pressure  $K_a$  (= $(1 - \sin \phi)/(1 + \sin \phi)$ ) is less than  $K_e$ , then shear failure would occur and the incremental ratio of horizontal to vertical stresses would be maintained at  $K_a$ . If  $K_a$  is greater than  $K_e$ , then the soil will behave elastically indefinitely. The unloading of the model would take place with the stress gradient given by  $K_e$ . The residual stresses resulting from these stress paths do not conform to experimental evidence of  $K_o$  loading, and this clearly indicates the inadequacy of the Mohr-Coulomb elasto-plastic model for

simulation of the compaction behaviour. In particular, firstly, it cannot maintain the loading along the  $K_0$  line and, secondly, active shear failure develops even without any lateral straining.

Subsequently, a double-yield (cap) model allowing for plastic volumetric strains under isotropic stresses (or irreversible compaction strains) was utilised in the modelling. This also failed to establish  $K_0$  loading and the loading path changed from elastic to volumetric failure and then to shear failure. Based on this simplistic study, which ignores the pore pressure development in unsaturated soils, it is apparent that these continuum models are not capable of modelling the compaction process effectively. After all, the original Jaky expression for  $K_0$  was developed using particulate mechanics and, therefore, it is not surprising that continuum models were unable to capture that behaviour. It would be interesting to test newly emerged particle flow codes (Cundall, 1996) to examine uniaxial loading and unloading behaviour without lateral straining.

#### • Modelling with semi -analytical model of Filz and Duncan (1992).

Based on the experimental evidence of  $K_0$  loading, Duncan and Seed (1986) developed a model to compute residual horizontal stresses adjacent to a rigid wall subsequent to compaction of granular soils in multiple lifts. This model assumes that a transient vertical stress due to compaction loading (*eg.* roller weight) is applied on the soil and removed as the roller moves away from the wall. The model simulates this by  $K_0$  (= 1- sin  $\phi$ ) loading of the transient vertical stress and subsequent unloading based on a non-linear path for removal of the transient stress. This results in a residual horizontal stress. Placement and compaction of subsequent lifts of soils are also modelled using hysteretic loading paths. This model deals with cohesionless soils and no pore pressure generation is taken into account. Filz and Duncan (1992) have extended this model to partially saturated cohesive soils where pore pressure generation is considered by an application of Skempton's *A* and *B* parameters. The model has been implemented in a computer program EPCOMPAC, and has been validated against several field and laboratory tests. However, Ishihara and Duncan (1993) pointed out that the model needs further, refinement and validation, particularly in the area of pore pressure generation. Despite that, this model appears to be the only one currently available for modelling compaction stresses in cohesive soils, and hence is used in the current study.

The EPCOMPAC program was used to simulate the compaction of a 1.5m thick clavey liner in 10 equal lifts. It was assumed that the compaction stresses generated would be equivalent to those adjacent to a rigid retaining wall, and that  $K_0$  conditions would apply. This seems reasonable because the stresses were computed at the centre of the layers along the centre line of the rollers. Parametric conditions used in these simulations are given in Table 9.1. A medium size roller and a heavy roller, representing standard and modified compaction efforts respectively, were used in the simulation. The total stresses computed for these two conditions are shown in Table 9.2. It can be seen that substantial residual horizontal stresses can develop within the liner depending on the compaction effort used. Clayton and Symons (1992) have noted that residual horizontal stresses in compacted cohesive backfill adjacent to rigid walls have been measured to be in the range of 0.4  $c_u$  to 0.8 $c_u$  for highly plastic soils where  $c_u$  is the undrained shear strength of the soil. If the undrained shear strength of compacted clay with moisture content a few percentage points wetter than optimum is assumed to be about 100 kPa, the above range will give residual stresses in the range 40 to 80 kPa. On this basis, it appears that the computed residual stresses are in the right order with respect to the experimental results.

### Influence of residual compaction stresses on cracking

In order to appreciate the influence of residual compaction stresses on cracking of clay liners, a simplified numerical study was undertaken using FLAC program. A 1.5m thick compacted clayey liner with parametric conditions and residual stresses, as in the previous section, was used in the study. It was assumed that cracks would develop at 1 m spacing within the clayey liner. Hence, as shown in Figure 9.3, a block of soil 0.5 m long and 1.5 m high was considered with the boundary conditions shown and with the initial stress conditions and parameters given in the previous section.

Parameter	Roller 1	Roller 2
Roller load (kg)	20,000	32,400
Liner thickness (m)	1.5	1.5
Lift thickness (mm)	150	150
K <sub>0</sub>	0.5	0.5
φ' (degrees)	30	30
c'(kPa)	57	57
φ (degrees)	22	22
c (kPa)	65	65
А	0.5	0.5
<sup>*</sup> E (MPa)	35	35
Unit weight, $\gamma$ (kN/m <sup>3</sup> )	20	20

Table 9.1 Parameters Used in Compaction Modelling

\*E is used in FLAC modelling only.

 Table 9.2 Residual Total Horizontal Stresses After Compaction of a Clayey Liner of

 1.5m Thickness

Depth	Roller 1		Roller 2			
below surface	Total Horizontal	Pore pressure	Total Horizontal	Pore pressure		
(mm)	Stress (kPa)	(kPa)	Stress (kPa)	(kPa)		
75	47.3	-15.6	56.9	-14.4		
225	51.1	-14.8	61.3	-13.5		
375	54.1	-14.0	64.9	-12.6		
525	56.7	-13.3	67.9	-11.9		
675	58.9	-12.6	70.6	-11.2		
825	60.8	-12.0	72.8	-10.5		
975	62.6	-11.4	74.8	-9.9		
1125	64.2	-10.9	76.7	-9.3		
1275	65.7	-10.3	78.3	-8.7		
1425	67.2	-9.7	79.9	-8.1		

An ideal elasto-plastic model with Mohr-Coulomb criterion was assumed for the soil constitutive behaviour. Then, the lateral restraint on the right hand vertical face was removed to simulate the formation of a crack. The results indicated that the block deformed towards the right side with the release of residual horizontal stresses. Several computations were conducted with different levels of overburden pressures applied on the liner surface. Figure 9.3 shows the lateral deformation of the right hand top corner of the block plotted against the overburden pressure. It is clear that the block behaves elastically up to about 160 kPa, and then shear failure takes place causing large deformations. Similar behaviour was observed for the situation with residual stresses generated by using the heavy roller, although the deformations in the elastic range were higher with proportional increases to residual stresses.

This study indicates that the residual horizontal stresses generated by the compaction process have the capacity to inhibit the formation of cracks in the clayey liner, and in these examples, they can accommodate displacements up to about 1.5 mm before a crack is formed. As expected, the overburden stresses also have the same effect, and if they are high enough, shear failure can occur in the liner closing the cracks. Furthermore, the study indicates that it is possible to approximate the liner behaviour as purely elastic for low overburden pressures and purely plastic for higher pressures. It follows then that these factors play significant roles in clayey liner cracking.



Figure 9.3 Results of FLAC Deformation Analysis of a Compacted Liner Section

# 9.3.2 Prediction of Crack Formation for Clayey Liners

#### Desiccation cracking

Lau *et al.* (1991) and Morris *et al.* (1992) have shown that, in an elastic soil with matrix suction  $(u_a - u_w)$ , the horizontal strain  $\varepsilon_x$  resulting from internal stresses  $\sigma_x$ ,  $\sigma_y$  and  $\sigma_z$  is given by:

$$\varepsilon_x = \frac{\sigma_x - u_a}{E} - \frac{v}{E} \left( \sigma_y + \sigma_z - 2u_a \right) + \frac{\left( u_a - u_w \right)}{H}$$
(9.1)

where  $u_a$  and  $u_w$  are the air and water pressures in the soil and E is a compression modulus for changes in the net total stress relative to the pore air pressure  $(\sigma - u_a)$ . H is an elastic modulus for changes in matrix suction  $(u_a - u_w)$ , and v is Poisson's ratio. Here, x and y directions are horizontal directions and z is the direction vertically down. Because the stress changes take place in the horizontal plane, cracks are considered to form in the vertical direction, and the x direction is assumed to be perpendicular to the cracks, whereas the y direction is parallel to the cracks.

Since lateral straining is not allowed before cracking, and assuming  $\sigma_y = \sigma_x$ , Equation 9.1 can be re-arranged as:

$$\sigma_{x} - u_{a} = \frac{v}{(1 - v)} (\sigma_{z} - u_{a}) - \frac{E}{H} \frac{(u_{a} - u_{w})}{(1 - v)}$$
(9.2)

For a compacted clayey liner subjected to a residual horizontal compaction stress of  $\sigma_c^*$ , and a vertical overburden pressure of  $p_o$ , Equation 9.2 may be modified as:

$$\sigma_{x} - u_{a} = \sigma_{c}^{*} + \frac{\nu}{(1-\nu)} p_{0} + \frac{\nu}{(1-\nu)} (\sigma_{z} - u_{a}) - \frac{E}{H} \frac{(u_{a} - u_{w})}{(1-\nu)}$$
(9.3)

Morris *et al.* (1992) has shown that it is reasonable to use E / H = (1 - 2v). It is assumed that cracking initiates when the net horizontal stress  $(\sigma_x - u_a)$  falls below the tensile strength of the soil. Substituting these conditions and using  $\sigma_z = \gamma z$ , where  $\gamma$  is the unit weight of liner soil and z is the depth measured from top of the liner surface and  $u_a = 0$ , an expression for depth of cracking  $(z_c)$  can be established as:

$$z_{c} = \frac{(I-2\nu)}{\nu\gamma} S_{0} + \left[ t_{s} - \sigma_{c}^{*} - \frac{\nu}{(I-\nu)} p_{o} \right] \frac{(I-\nu)}{\nu\gamma}$$
(9.4)

where  $S_0$  is the average matric suction within the clayey liner. It should be noted that here a constant suction value was assumed to exist within the liner. However, this will produce conservative results with greater crack depths if a significant suction gradient exists, for example due to a water table in the ground below. As demonstrated by Morris *et al.* (1992), Equation 9.4 can be easily modified for such a situation. The tensile strength of clayey soils can increase with suction. Morris *et al.* (1992) derived a relationship of tensile strength *t* and matrix suction which could be expressed as:

$$t_{s} = -0.5 \left[ c' + (u_{a} - u_{w}) \tan(\phi' - 5) \right] \cot \phi'$$
(9.5)

where c' = effective cohesion in kPa,  $\phi'$  is in degrees. For slurries the value of c' is very low and is ignored by Morris *et al.* (1992). For simplicity, the same assumption is considered applicable for compacted clays. The final form of the equation is given below:

$$t_s = -0.5 (u_a - u_w) \tan(\phi' - 5)$$
(9.6)

Equation 9.4 is applied to a parametric evaluation of cracking of clayey liners, and the results are presented in Table 9.3. Cracking depths were determined for two Poisson's ratio values (0.3 and 0.4), two overburden pressures (0 and 20 kPa) and three values of

residual compaction stresses (0,50,70 kPa) under a range of suction values (20 to 500 kPa).  $S^*$  gives the suction values required to initiate cracking.

Suction,	Depth of Cracking at					Depth of Cracking at						
kPa, (pF)	v = 0.3					$\nu = 0.4$						
	$p_0 = 0$ $p_0 = 20$		$p_0 = 0$			$p_0 = 20$						
	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$	$\sigma_c^* =$
	0	50	70	0	50	70	0	50	70	0	50	70
20 (2.30)	0.8	nc	nc	nc	nc	nc	0.15	nc	nc	nc	nc	nc
40 (2.60)	1.6	nc	nc	0.6	nc	nc	0.3	nc	nc	nc	nc	nc
60 (2.78)	2.4	nc	nc	1.4	nc	nc	0.45	nc	nc	nc	nc	nc
80 (2.90)	3.2	nc	nc	2.2	nc	nc	0.6	nc	nc	nc	nc	nc
100 (3.00)	3.9	nc	nc	2.9	nc	nc	0.75	nc	nc	nc	nc	nc
200 (3.30)	7.9	2.1	nc	6.9	1.1	nc	1.5	nc	nc	0.5	nc	nc
500 (3.70)	19.7	13.9	11.6	18.7	12.9	10.6	3.8	0.01	nc	2.8	nc	nc
$S^*(kPa)$	>0	147	207	>0	173	232	>0	500	700	135	632	830

Table 9.3 Calculations on Depth of Cracking (m)

Notes:

nc - no cracking

 $S^*$  - the suction required to initiate cracking (all the pressures and stresses are in kPa.)

The results indicate that, for compacted clay under zero overburden pressure, cracking would develop even at very low suctions (high saturation) if the residual compaction stresses are ignored (see  $\sigma_c^* = 0$  and  $p_0 = 0$  results). This result is contrary to the generally observed behaviour of clayey soils where cracking is not normally observed at these suctions or saturation levels. The suction necessary to create cracking, however, increases markedly when residual compaction stresses are taken into account. For example, when Poisson's ratio is 0.4, and under zero overburden stress, cracking would not initiate until suction reaches 500 kPa and 700 kPa respectively for residual compaction stresses of 50 and 70 kPa. For Poisson's ratio of 0.3, cracking would initiate at lower suction levels.

cracking. It is also clear that by increasing overburden stress to 20 kPa (approx. 1m of soil), cracking can be minimised. It should be noted that covering with additional soil cover would be of more advantage in minimising the moisture loss and, therefore, the generation of high suction levels.

The preceding analysis is a simplified account of the possible influence of residual compaction stresses on desiccation cracking, but nevertheless highlights their importance. The depths of cracks in Table 9.3 were computed directly from Equation 9.4, and may not be applicable to a clayey liner of limited thickness. Furthermore, if ground water is present near the surface, then that would also control the suction profile and the cracking depth. The cracking depths may also be influenced by stress concentrations at crack tips as well as shear failure of soils (Morris *et al.*, 1992).

#### • Clay structure changes due to chemicals

It is believed that some chemical liquids reduce the clay double layer thickness and this causes flocculation and increases hydraulic conductivity. The diffuse double layer theory based on Gouy-Chapman equation is commonly used to explain these effects. Based on this equation, the physico-chemical properties of leachate, which can lead to these changes, include high electrolyte concentration, low dielectric constant and high cation valence. The effect of these properties is to bring the clay particles closer together and, therefore, it can be argued that this effect is similar to that of shrinkage of soils due to desiccation. On this basis, it may be possible to establish a relationship between the stress system within the compacted clay and the physico-chemical properties of the leachate at the onset of these clay structural changes.

In this present analysis, only the effect of high electrolyte concentration of the leachate is considered in some detail. In this situation, the liner is most likely to be flooded with leachate, and, it is reasonable to assume that the clay is saturated. Therefore, the effective stresses are used to compute the lateral strain, which can be given as:

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$$\varepsilon_{x} = \frac{\sigma'_{x}}{E} (1 - \nu) - \frac{\sigma'^{*}_{c}}{E} (1 - \nu) - \frac{\nu}{E} \sigma'_{z} + \frac{S}{H}$$
(9.7)

In the above equation it is assumed that  $u_{e} = u_{w}$  at saturation in Equation 9.1 (Fredluand and Raharjo, 1994). S is the solute suction generated by electrolytes in the leachate. However, there is some experimental evidence to indicate that modulus of solute suction is much higher than that of matric suction (Miller and Nelson, 1992). For simplicity, the same modulus H is used for matric suction. Here, the effective residual compaction stress  $\sigma_{c}^{\prime*}$  is conveniently approximated by total stress  $\sigma_{c}^{*}$ . This conservative assumption is not unreasonable because the pore pressure generated after compaction is not apparently very significant (as in Table 9.2) unless the equilibrium with external water pressure is already achieved. However, in compatibility tests, where equilibrium is first established with water, effective solute suction will be S, provided no change has been made to the external pore water pressure, when switching from water to the chemical permeant. In the foregoing analysis, initial equilibrium in water pressures are assumed (*ie.* net solute suction = S).

Imposing these conditions and using  $\varepsilon_x = 0$  as well as the  $\sigma'_x = t_s$  condition, Equation 9.7 can be rearranged to give the solute suction S at the onset of clay structural changes as:

$$S = \frac{\nu}{(1-2\nu)} \left( p'_0 + \gamma' z \right) + \frac{(1-\nu)}{(1-2\nu)} \left( \sigma_c^* - t_s \right)$$
(9.8)

where z is the depth measured from the top of the liner and  $\gamma'$  is the submerged unit weight of soil.

Table 9.4 shows a parametric study undertaken using Equation 9.8, in which the suctions required to initiate structural changes at the top (z = 0) of a clayey liner were computed for a range of conditions assuming that  $t_s = -10$  kPa. It was deemed interesting to assess the likely electrolyte concentrations, which would cause these suctions. For this purpose, an equation presented by Mitchell (1976) is used:

where *R* is the gas constant (8.314 J/K/mol); *T* is absolute temperature (K); and *C* is the difference in the cation concentration between leachate and the mid planes of the double layers of clay particles. The results shown in Table 9.4 were computed for NaCl leachates at  $25^{\circ}$ C. They again highlight the importance of compaction stresses in providing resistance to the effects of structural changes. The results also indicate that liners seem to be unaffected by leachate unless NaCl concentrations exceed about 4200 mg/L for the conditions assumed.

(9.9)

$p'_0$	$\sigma_c^* = 0, t_s = -10$ v = 0.4		$\sigma_c^* = 50, t$ $v = 0.4$	$f_{s} = -10$	$\sigma_c^* = 70, t_s = -10$ v = 0.4		
	S	С	S	С	S	С	
0	30	708	180	4245	240	5660	
20	70	1651	220	5189	280	6604	
40	110	2594	260	6132	320	7547	
60	150	3538	300	7075	360	8491	
80	190	4481	340	8019	400	9434	
100	230	5425	380	8962	440	10377	

**Table 9.4** Parametric Results for Chemical Effects

Notes:

All the pressures and stresses are in kPa. Electrolyte concentration is in mg/L.

# 9.4 New Testing Technique

On the basis of the previous discussions concerning the problems of the traditional testing techniques and the theoretical analysis carried out in the previous section, a new testing technique is proposed. In this proposed technique, testing is suggested in the rigid wall triaxial permeameter similar to the technique used by Huang (1994). As

discussed previously, Huang (1994) used a rigid mould in the triaxial permeameter to avoid the volume reduction due to the effective stress. Although this approach could reduce the effective stress consolidation problem, it does not address the problem of side wall leakage. The rigid mould was used in his technique and, therefore, the problem of side wall leakage could still exist. This was taken into consideration in the new technique by gluing the soil specimen with the fixed ring. By gluing the soil specimen with the mould, the specimen boundary was laterally restrained against shrinking of soil. Gluing restricts the soil movement away from the walls of the permeameter. Therefore, if any changes occur within the soil specimen due to chemical stresses, they should occur within the soil.

#### 9.4.1 Selection of Glue

A range of glues (wood glue, plumbing glue and glue for heavy construction) were tested for the compatibility of soil with the fixed ring. Some of the glues tested were found to be unsuitable for the soils probably due to the presence of moisture in the soil. Since the steel mould had a smooth inside surface, bonding was not effective with some of these glues. Super strength Araldite, an epoxy resin manufactured by Shellys Chemical Company Pty Ltd., was found to be compatible with the wet soil and the solid steel mould. Araldite, which is readily available in common hardware stores, is mixed in the proportion of 1:1 from Tube A and Tube B in order to manufacture the glue for testing. Full strength of Araldite is attained after 48 hours. According to the company specification, the strength of Araldite is 10MPa after 24 hours.

Split steel rings that fit into the flexible wall permeameter were manufactured for the testing. The soil was compacted in the split steel rings that were maintained in position by clamps during compaction. Subsequently, the clamps were removed and the rings were taken apart. The glue was applied to the inner surfaces of the rings and was placed around the compacted soil specimen. The rings were also joined together using the glue and clamped again to keep the specimen in position. The specimens were then placed in a moist chamber for 48 hours to attain the full strength of the glue. This was undertaken to

protect the soil from drying and cracking while the full strength of the glue was attained. The glued specimen along with the rigid mould was then wrapped with the rubber membrane and was placed in the flexible wall permeameter for testing. Performance of the glue was tested during drying and soaking with 4M NaCl and 60% methanol. The results of the investigations are explained below.

Two identical soil specimens were compacted and tested for desiccation. After compaction, one of the specimens was glued with the mould and cured for 48 hours and was left open to the atmosphere in the soils laboratory. The other specimen after compaction in the mould was allowed to dry in the same environment. The results of these two investigations are presented in Figures 9.4(a) and 9.4(b). Figure 9.4(a) shows the drying of the soil specimen without any glue. It can be seen that the soil shrank away from the steel mould leaving a gap between soil specimen and the inner solid ring wall. This behaviour can give rise to the side wall leakage as explained earlier. But in the case of the glued specimen as shown in Figure 9.4(b), the soil specimens cracked within the centre, and no side wall gaps were found. Effects of thickness on desiccation were also investigated in a similar way by compacting and gluing the specimen within fixed rings and leaving them in the open atmosphere in the soils laboratory. Specimen thickness of 40 mm and 20 mm were considered for this investigation. From Figure 9.5 (a) and 9.5 (b), it is clear that the cracks penetrated the entire depth. Furthermore, the crack widths in the thinner specimen were found to be greater than those in the thicker specimen. This observation is consistent with similar experimental observations made on desiccation cracking (Lau, 1987). This highlights that the specimen height can have an influence on the hydraulic conductivity testing results.

The test results from Figure 9.6 show the effectiveness of the glue while soaking in 4M NaCl solution. The tests were carried out with compacted soil specimens soaked in saline water (4M NaCl) without any effective stress. It could be seen from Figure 9.6 that tiny hair cracks were formed in soil at the centre of the specimen but the sides were found intact. In test EMN1, the permeation of 60% methanol was continued for a long duration (6.6 pore volumes). Figure 9.7 is a photo taken of the specimens after



Figure 9.4: Effectiveness of Glue on Drying: (a) Without Glue; (b) Glued Specimen



20 mm

40 mm

Figure 9.5: Effect of Specimen Thickness on Cracking of Soil



Figure 9.6: Effectiveness of Glue While Soaking in 4M NaCl Solution



Figure 9.7: Effectiveness of Glue After Testing with 60% Methanol



**Figure 9.8:** Effectiveness of Glue After Soil Specimen was Broken on Completion of Testing with 60% Methanol
completion of the testing in Test EMN1. It can be seen from the test result that the visible hair cracks were found in the centre of the specimen, but no indication of the side wall leakage was observed. Equal pressure was applied to the whole specimen in order to remove the soil from the mould. It could be seen from Figure 9.8 that only soil from the centre of the specimen came out very easily leaving the soil along the sides of the mould. This also indicates the effectiveness of the glue even after the testing.

#### 9.4.2 Test Results for Hydraulic Conductivity

Hydraulic conductivity tests using the new proposed technique were carried out in the laboratory using different concentrations of NaCl, methanol and leachate. The results are presented in Figures 9.9 to 9.17 and are tabulated in Tables 9.5(a) and 9.5(b). These hydraulic conductivity values were computed on the basis of inflow rates. As explained in previous chapters,  $k_w$  and  $k_{ch}$  are used to express hydraulic conductivity with water and chemicals respectively.

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It could be seen from the test results that the increase in hydraulic conductivity for the Test ESN3 is about 200%, compared to the lowest hydraulic conductivity obtained, after 2M NaCl solution was used as the permeant (Figure 9.11). As can be seen for the Test ES6 from the Table 7.2, an increase in the hydraulic conductivity of 7.5% was observed when the test was carried out with a flexible wall permeameter for the same concentration of NaCl. Despite the fact that there are small differences in specimen thickness and applied effective stress, the difference in the hydraulic conductivity measured using two methods could be considered substantial. For the Test ESN4, more than 4 times increase in hydraulic conductivity was observed when 4M NaCl was used as the permeant. The reasons for the increases in hydraulic conductivity due to the permeation of electrolyte concentrations were already discussed in Chapter 7.

In the Test EMN1 where 60% methanol was used as the permeant, the increase in hydraulic conductivity was about 30 times. In the test results presented in Tables 8.2(a)

for the same concentration of methanol in the compaction mould permeameter, the increase in hydraulic conductivity was about two orders of magnitude. In case of FWP, as the result presented in Table 8.2(b) show, a decrease in hydraulic conductivity was observed after the introduction of methanol at the same concentration. This result highlights the relevance of the lateral boundary condition to compatibility testing.

About two orders of magnitude increase in hydraulic conductivity was observed for 80% methanol in Test EMN2. However, this increase could be much higher because the inflow interface chamber was empty when the reading was taken. As can be seen from Table 8.2(b), the hydraulic conductivity decreased substantially when the test was conducted in FWP with the same concentration of methanol. As discussed in Chapter 8, this decrease may be attributed to the consolidation or shrinkage of the soil specimen under chemical stress imposed by methanol. Despite these tests being conducted in exactly identical conditions, it could still be seen that the new technique yielded hydraulic conductivity values between compaction mould permeameter and FWP. This is considered to be the expected outcome from the three testing techniques. In order to examine the influence of the test method, tests were carried out under identical test conditions in the three techniques using methanol at 60% concentration as the permeant. Results of the investigation are presented in Figure 9.18. It could be seen from the figure that the increases in hydraulic conductivity in cases of RWP, new technique and FWP are 76, 14 and - 0.45 times respectively.

As can be seen from the Test WLN1, the hydraulic conductivity decreased after the leachate was introduced. Although it was found from earlier tests (Chapter 8) that the same leachate has the potential to increase the hydraulic conductivity, this was not observed in the current test. Although the effective stress consolidation problem was eliminated using the new technique, the possibility of plugging of the colloidal particles, present in the leachate, is still possible. Plugging could occur in the soil specimen, in porous stones and in filter papers. In case of FWPs, the reasons for the decreases in hydraulic conductivity due to the leachate permeation were already discussed in Chapter 8. These are the clogging of soil pores, porous stones and filter papers because of the

presence of colloidal particles in leachate. The same reasons could be applicable for the new technique.

Volume change behaviour of the soil specimens was also measured during testing. Similar volume change patterns were observed in all the cases as was observed in the previous chapters and only one result is presented in Figure 9.15. It could be seen from the figure that very minimal cell volume change occurred during testing and the reason for this is discussed in the following section.

 Table 9.5(a): Effect of Different Concentrations of NaCl on Hydraulic Conductivity of

 Compacted Clay Using Laterally Restraint Boundary Condition

Soil	Echuca	Echuca	Echuca	Echuca
Permeant	lM NaCl	1M NaCl	2M NaCl	4M NaCl
Test number	ESN1	ESN2	ESN3	ESN4
Effective stress	50	50	50	60
(kPa)				
Specimen	20	30	20	40
thickness (mm)				
Hyd. Gradient	60	66.7	50	50
$k_w \ge 10^{-11} (m/s)$	6.26	5.05	7.03	40.10
$k_{ch} \ge 10^{-11} (m/s)$	7.29	5.12	14.30	178.00
pv	3.50	1.80	6.80	7.0

**Table 9.5(b):** Effect of Leachate and Methanol at Different Concentrations onHydraulic Conductivity of Compacted Clay Using Laterally Restraint Boundary

Soil	Werribee	Werribee	Echuca	Echuca	Werribee
Permeant	60%	60%	60%	80%	Leachate
	Methanol	Methanol	Methanol	Methanol	
Test number	WMN1	WMN2	EMN1	EMN2	WLNI
Effective stress	50	50	50	50	60
(kPa)					
Thickness (mm)	30	20	30	30	40
Hyd. Gradient	60	80	60	33.3	100
$k_w x 10^{-11} (m/s)$	9.15	8.01	37.10	17.80	14.40
$k_{ch} \ge 10^{-11} (m/s)$	13.10	152.00	1190.00	1020.00	4.95
pv	2.75	3.80	6.60	4.40	2.40

Condition

#### 9.4.3 Comments on the New Technique

As discussed before, a rigid mould was used in the traditional triaxial cell permeameter in the new technique. Instead of using the soil specimen wrapped directly by the flexible rubber membrane, a rigid mould filled with soil was used. The adhesion between the rigid mould and the rubber membrane was maintained by using high vacuum silicon grease. At low effective stresses the adhesion between the rubber membrane and the rigid wall was found to be less effective as some leakage was detected. Under the current technique, effective stress of 40 kPa or above was found to be effective.

Since in the new technique, the soil specimens were glued to the mould, it is not clear how effective the transfer of vertical stresses to the soil is. The transfer of vertical stress is facilitated by making the specimen slightly bigger than the top and bottom caps. This way the caps will rest on the compacted soil away from the glued area. Tests carried out in the new technique considered the inside diameter of the rigid mould of 104 mm whereas the top and the bottom caps are 100 mm in diameter. This provides sufficient



**Figure 9.9:** Effect of 1M NaCl on the Hydraulic Conductivity of a 20mm Thick Echuca Soil Specimen Using the New Technique



Figure 9.10: Effect of 1M NaCl on the Hydraulic Conductivity of a 30mm Thick Echuca Soil Specimen Using the New Technique



**Figure 9.11:** Effect of 2M NaCl on the Hydraulic Conductivity of a 20mm Thick Echuca Soil Specimen Using the New Technique



**Figure 9.12:** Effect of 4M NaCl on the Hydraulic Conductivity of a 40mm Thick Echuca Soil Using the New Technique



**Figure 9.13:** Effect of 60% Methanol on the Hydraulic Conductivity of a 30mm Thick Werribee Soil Specimen Using the New Technique



**Figure 9.14:** Effect of 60% Methanol on the Hydraulic Conductivity of a 20mm Thick Werribee Soil Specimen Using the New Technique



**Pore Volumes** 

**Figure 9.15:** Effect of 60% Methanol on the Hydraulic Conductivity of a 30mm Thick Echuca Soil Specimen Using the New Technique



**Figure 9.16:** Effect of 80% Methanol on the Hydraulic Conductivity of a 30mm Thick Echuca Soil Specimen Using the New Technique



**Figure 9.17:** Effect of Leachate on the Hydraulic Conductivity of a 40mm Thick Werribee Soil Specimen Using the New Technique

space for the easy movement of the top cap into the mould, despite the gluing. Loadsettlement measurements of two specimens, one with the glue and the other without the glue, were carried out to investigate this aspect using a traditional consolidation odeometer in the soils laboratory. The results of the investigation are presented in Figure 9.19. It could be seen from the figure that no significant differences in settlement were observed even up to 240 kPa effective stress. Therefore, it can be concluded that the proposed technique allows effective transition of vertical effective stress. This is also evident from Figure 9.15 where only a very little increase in cell volume was observed. As the specimen was laterally restrainted, this volume change could only be possible due to the vertical consolidation of the specimen.

#### 9.5 CONCLUDING REMARKS

Compacted clay liners can display forms of cracking which lead to large increases in hydraulic conductivity. Cracking can occur due to desiccation when the clayey liners undergo moisture loss due to drying. The clay structure can also flocculate, which has similar effects to cracking, when exposed to certain chemical, liquids and leachates. A simplified theoretical analysis for the initiation and depth of cracking under these conditions was presented. It was found that in addition to overburden stresses, the likely residual compaction stresses could play a significant role in inhibiting crack initiation and growth. However, there is no experimental data on this issue especially in relation to compacted clay liners.

It is important to account for residual compaction stresses in the assessment of likely structural changes in clayey liners due to either desiccation cracking or chemical effects. Equations were derived to predict the conditions required for these changes to occur, and some parametric results were presented. Although these equations are considered to represent the likely mechanisms of liner behaviour, they are based on a number of simplifying assumptions, especially in relation to chemical effects. Therefore, while this

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**Figure 9.18:** Pore Volumes vs Hydraulic Conductivity Test Results from Compaction Mould, New Technique and FWP

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Figure 9.19: Load Settlement Analysis of Glued and Unglued Soil Specimen

equation may be useful in examining the relative effects of various influencing variables, further research is needed to examine the validity of these assumptions in more detail.

The problems related to the traditional hydraulic conductivity testing are discussed with their boundary conditions. On the basis of the limitations of the current testing techniques, a new test setup (ie. the rigid ring triaxial cell permeameter) is proposed. Use of the fixed ring could eliminate the lateral effective stress consolidation, as found in the flexible wall permeameter. The technique involves the use of suitable glue for gluing the soil specimen with the fixed rings. With this methodology, the side wall leakage problem, which is the problem inherent to fixed wall permeameter, could be eliminated. Ĭn consideration of the boundary effect, it was found that the new technique simulates the field condition better than the traditional FWPs or RWPs. Validation of the technique was carried out in the laboratory using different concentrations of NaCl, methanol and leachate as the permeant. As expected, the results obtained from the new technique fall between the results from the FWP and the RWP. Hence, it is considered that the proposed test technique is better suited for conducting chemical compatibility testing through the measurement of hydraulic conductivity.

# 10 CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

#### **10.1 SUMMARY AND CONCLUSIONS**

As discussed in Chapter 1, the overall aim of the research project was to undertake a comprehensive investigation into the laboratory measurement of hydraulic conductivity with particular emphasis on selected Victorian soils used for liner construction. This was carried out in the laboratory using different concentrations of methanol and NaCl, and landfill leachate from a local landfill site. Reagent grade methanol and NaCl were used for the preparation of the permeants at different concentrations. Leachate samples were also collected from PWM Lyndhurst site and were analysed for physical and chemical properties. For such investigation soil samples were collected from four different locations of western and north-western Victoria as Sunshine, Melton, Werribee and Echuca. The collected soil samples were analysed for physical, physico-chemical and mineralogical properties and are discussed in the respective chapters.

Within the broad aim of the project, theoretical investigation of the parametric study and a study of the structural changes within the soil specimen due to the permeation of different chemicals was carried out. There could be a variety of problems in the laboratory testing relating to the structural changes within the soil specimen. The problems of the traditional testing methods were identified and it was found that these methods do not adequately reflect the field boundary conditions. In the light of theoretical work undertaken, a new testing technique was proposed to reflect the field situation better. The validation of the technique was also carried out in the laboratory using a variety of chemical permeants. The following conclusions are made on the basis of the theoretical and experimental work undertaken:

• Environmental guidelines commonly specify that the hydraulic conductivity of compacted clay liners should be less than 1x 10<sup>-9</sup> meters/second during the design

life. However, there is experimental evidence that the hydraulic conductivity could change abruptly when the permeation is made with some chemicals specially with liquid hydrocarbons. Increases in hydraulic conductivity are attributed to the changes in the soil structure due to the collapse of the electrical double layer, resulting in the formation of a flocculated clay structure. Therefore, it is important that the soil hydraulic conductivity be assessed rationally to ensure the effective performance of clay liners.

- Results from the physical, physico-chemical and mineralogical analyses of soil revealed the presence of different amounts of clay, silt and sand particles and a number of mineral groups. It is generally concluded that the soils are suitable as liner material for containing municipal wastes. Sunshine soil has less clay and more silt and sand, which gave higher hydraulic conductivity than for the other soils. Melton, Werribee and Echuca soil have less sand and more clay. Melton soil features high clay content compared to Echuca and Werribee.
- The data of some selected chemical properties (electrical conductivity, EC) revealed that all the soils are saline and the soil pH ranges from slightly alkaline (Echuca) to moderately alkaline for other soils. The soil texture is clayey sand (Sunshine) and sandy clay for other soils. Cation exchange capacity (CEC) of Melton is higher than that of the other soils. Higher concentrations of four major cations in Werribee soils compared to Echuca soil also reflect the higher values of CEC in Werribee soil.
- XRD results revealed the presence of both swelling (smectite and Vermiculite) and non-swelling (kaolinite, illite and chlorite) minerals in all soils. Illite and kaolinite are present in significant amounts in both coarse and fine fractions of all soils. In the finer fraction illite is the major component in Sunshine, Melton and Echuca soils. Vermiculite is also present as a major clay mineral in the finer fraction in Echuca soil. Smectite is the dominant clay mineral for Werribee soil, as would be expected from its residual basaltic origin.

- Moisture content relationships for compacted soil were established. Laboratory test
  results and published laboratory data on relationships of dry unit weight and
  hydraulic conductivity with moisture content for compacted clays were analysed.
  Based on linear regression analyses, it was established that the moisture content
  which gives maximum wet unit weight is a closer approximation and better
  correlated to the moisture content giving minimum hydraulic conductivity than the
  traditional optimum moisture content.
- Atterberg limits for collected soils mixed with chemicals and leachates were determined. In agreement with the published data, the test results indicated that the soil loses the plasticity by decreasing the liquid limit and increasing the plastic limit when mixed with acetic acid, methanol, landfill leachate and NaCl solution. This effect is most prominent for soils containing higher clay content and smectite group minerals. Furthermore, the plasticity of the soils continually dropped with the increase in concentration of the chemicals, and this was most pronounced for liquids having a low dielectric constant. Generally, it can be concluded that the effect on Atterberg limits by various chemicals is a function of the mineralogical composition and the clay content of soils, and physico-chemical properties of the moulding chemical.
- From the theoretical as well as the experimental results, it was found that the hydraulic conductivity decreases with increasing gradient. For a particular soil and the test conditions used with water as permeant, a reduction of less than a factor of 1.5 was observed when the gradient was increased up to 300. Hence, for compacted clay specimen, it was concluded that the use of hydraulic gradient of 300 would not have significant influence on the test results.
- Different options of gradient application were theoretically analysed and their suitability for FWP was judged on this basis. It was found that Option 1, where the application of hydraulic gradient is applied by reducing the top pore pressure and increasing the bottom pore pressure by the same amount, is the most suitable for FWP hydraulic conductivity testing. A wide range of parametric values was

considered for the evaluation and it was found that different parameters have a varying influence on the hydraulic conductivity. It was also found that the effect is different from normally consolidated to over-consolidated soils.

- The non-dimensional parameters that influence the effect of hydraulic conductivity by the hydraulic gradient application were identified. A general trend of decreasing hydraulic conductivity ratio with increasing hydraulic gradient was observed. Normally, increases in C<sub>c</sub>/C<sub>k</sub> ratio decreases hydraulic conductivity ratio. Within the normally consolidated region, the hydraulic conductivity is independent of σ'<sub>c</sub>/σ'<sub>v0</sub> ratio and there is a general trend of decrease in hydraulic conductivity with the increase in hydraulic gradient. For over consolidated soils, hydraulic conductivity starts increasing with the increase of hydraulic gradient. For different σ'<sub>v0</sub>/L<sub>0</sub>γ<sub>w</sub> ratios, a decrease in hydraulic conductivity was observed with the increase of hydraulic gradient. This decrease in hydraulic conductivity is found to be higher with the decrease of σ'<sub>v0</sub>/L<sub>0</sub>γ<sub>w</sub> ratio.
- Effect of electrolyte concentration on hydraulic conductivity of compacted clay was carried out for a wide range of NaCl concentrations, ranging from 0.05M to 4M using both RWPs and FWPs. Hydraulic conductivity and volume changes were not observed when the permeation was made with 0.05M NaCl in flexible wall permeameter. In all other cases increases in the hydraulic conductivity and volume changes were observed with higher values as the NaCl concentration increases. This increase is higher in case of RWPs than FWPs. In FWPs increase in the hydraulic conductivity may be reduced due to the osmotic consolidation and high effective stress. In the consolidation cell permeameter (RWP), an increase in hydraulic conductivity of about 54 times was observed at 2M NaCl concentration and at an effective stress close to preconsolidation pressure. These increases in hydraulic conductivity could be due to the structural changes and/or due to the side wall leakage. Both mechanisms could be more pronounced at low effective stresses. A volume change of about 0.7 % was observed during the permeation of 2M NaCl solution in FWP. Therefore, it is essential in compatibility testing using FWPs to

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measure the changes in cell, inflow and outflow volumes in order to assess the soil structural changes along with hydraulic conductivity.

- Hydraulic conductivity tests were carried out with FWPs and RWPs using landfill leachate, methanol and modified leachate as permeants. It was found for both FWPs and consolidation cell permeameters, that a significant reduction in hydraulic conductivity was observed when leachate was used as a permeant. The possible blockage of the flow path due to the presence of colloidal particles in the leachate and the effect of effective stress could be the two possible reasons for such decrease. Volume changes in terms of cell volume and the difference in (outflow-inflow) volumes revealed the possible shrinkage and structural changes within the specimen. Test carried out with compaction mould permeameter (RWP) showed 2 times increase in hydraulic conductivity.
- In the case of compaction mould permeameter (RWP), more than two orders of magnitude increase in hydraulic conductivity were observed when methanol at 60% concentration was used as a permeant. Visible hair cracks in the specimen and side wall gaps were observed after the test was completed. For FWPs a gradual decrease in hydraulic conductivity was observed with the passage of time after methanol was introduced as the permeant. In the case of methanol, similar cell volume changes and (outflow-inflow) volume changes were observed as was found in the case of leachate. In the tests conducted in consolidation cell permeameter a very little increase in hydraulic conductivity was observed at a high effective stress (100 kPa) but at an effective stress of 64 kPa, a 5 times increase in hydraulic conductivity was observed.
- The problems of different types of permeameters used for different chemicals are as follows. In general, the flexible wall permeameter suffers a volume change due to effective stress, while the compaction mould and consolidation cell permeameters suffer a side wall leakage. With the application of side sealant, the side wall leakage could be reduced but the results may not be representative. Improvement of the

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testing technique is required to reduce or eliminate the problems related to different types of permeameters to monitor the changes in hydraulic conductivity.

- Theoretical analyses for structural changes and cracking due to desiccation and chemical effects were carried out. It was found that the clay structure could flocculate, which has similar effects to cracking, when exposed to certain chemicals and leachates. A simplified theoretical analysis for the initiation and depth of cracking under these conditions was presented. It was found that in addition to overburden stresses, the residual compaction stresses play a significant role in inhibiting crack initiation and growth.
- A new testing technique (*ie.* the rigid ring triaxial cell permeameter) was proposed. Use of the fixed ring could eliminate the effective stress consolidation, which is the problem related to the flexible wall permeameter. The technique involves the use of suitable glue for gluing the soil specimen with the fixed rings. This methodology will eliminate side wall leakage problem inherent to fixed wall permeameter. In consideration of the boundary effects, the new technique simulates the field condition better than the traditional FWPs or RWPs. Validation of the technique was carried out in the laboratory using different concentrations of NaCl, methanol and leachate as the permeant. It was also found that the results obtained from the new technique fall in between the results from the FWP and the RWP. This is expected because in RWPs side wall leakage makes the hydraulic conductivity significantly higher and in the cases of FWPs, the hydraulic conductivity is decreased due to the effective stress consolidation or shrinkage.

### **10.2 Recommendations for Further Research**

As discussed earlier, a detailed laboratory investigation into the hydraulic conductivity of compacted clay was carried out using selected Victorian soils. On the basis of the laboratory investigation, it was found that the soils used in the present study are suitable for liner construction. However, further research is necessary in the following areas:

- It was found that the hydraulic conductivity is dependent on the mineralogy and the type of chemicals used as the permeant. Study of the clay mineralogy before and after the permeation could help explain the structural changes that might have occurred due to the permeation of different chemicals, and may subsequently give an indication of the cause of the variation of hydraulic conductivity.
- Cracking of the soil is an important topic as far as the hydraulic conductivity of the liner is concerned. There is still more research to be undertaken on this topic. It was found from the limited test results that the thickness might have an influence on the cracking of the specimen and these need to be studied further. This could give a good guide for the selection of a suitable specimen size for the laboratory hydraulic conductivity testing. This result can be investigated theoretically and can be followed with some validation testing.
- In the present study the leachate sample was collected from the PWM Lyndhurst site. There is evidence of the significant variations in the quality of these leachate depending on the type and age of the landfill, time of the year, climatic condition etc. Study of the range of leachate qualities in Victoria may be useful to find the adverse effect on the liner material with the specific leachate. The long term behaviour and the possible clogging of pores from suspended particles may need further attention.
- As discussed in Chapter 6, one of the problems related to the testing with high hydraulic gradient is the particle migration. Both piping and plugging might occur due to particle migration, which may have significant influence on the hydraulic conductivity of the tested specimen. This issue can be studied at fundamental level by undertaking targeted theoretical and experimental studies.

- As was discussed in Chapters 7 & 8, there is evidence of the effective stress volume changes and the chemically induced volume changes in the traditional FWP testing. Although the effect of effective stress consolidation on hydraulic conductivity can be easily assessed, the effect of chemically induced consolidation needs further theoretical and experimental study in order to predict the likely effect on hydraulic conductivity.
- Finally, further study on the new technique needs to be carried out. This may include laboratory testing for soil as well as testing for other liner materials (*eg.* GCL) to investigate the problems the new technique may have for different types of specimen. It is also necessary to advance the theoretical aspects for the prediction of structural changes under laterally restrained boundary conditions. This will need to incorporate the development of solute suction profiles in the soil and the possible initiation and propagation of cracks. The concepts of double layer theory and fracture mechanics may be used for the purpose.

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ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
JGED	Journal of Geotechnical Engineering Division
JGGEvED	Journal of Geotechnical and Geoenvironmental Engineering Division
JSMFD	Journal of Soil Mechanics and Foundation Division
ICSMFE	International Conference on Soil Mechanics and Foundation Engineering
SMFE	Soil Mechanics and Foundation Engineering
STP	Special Technical Publication

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**APPENDICES** 

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A3.1(a): Location for Brooklyn Municipal Tip, Sunshine



A3.1(b): Location of Borrow Site for Sunshine Soil







A3.3: Location for West Road Landfill, Werribee



A3.4: Location for Terrick Road Waste Water Lagoon, Echuca









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A3.8: Grain Size Analysis for Echuca Soil



A3.9: Moisture Content Density Relationship for Sunshine Soil



A3.10: Moisture Content Density Relationship for Melton Soil





A3.11: Moisture Content Density Relationship for Werribee Soil



A3.12: Moisture Content Density Relationship for Echuca Soil



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A3.13: X-Ray Diffraction Patterns for the Fine Fraction of Sunshine Soil after Potassium Treatment



-ZHUZO-HY OLO X -00











A3.19: X-Ray Diffraction Patterns for the Coarse Fraction of Sunshine Soil after Potassium Treatment









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-ZHMZQTH> OTQ × 4000

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A3.41: X-Ray Diffraction Patterns for the Fine Fraction of Werribee Soil after Magnesium Treatment and Ethylene Glycolation









-ZHUZO\_H> OLO × -000















-ZHWZ0-HY OLO X -000





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A3.52: X-Ray Diffraction Patterns for the Fine Fraction of Echuca Soil after Magnesium Treatment



























Time (days)

A4.1: Leakage Test Results of Cell 1 in Tri-Flex 2 Equipment

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Time (days)

A4.2: Leakage Test Results of Cell 2 in Tri-Flex 2 Equipment



A4.3: Leakage Test Results of Cell 3 in Tri-Flex 2 Equipment

Soil sample	Echuca	Echuca	Echuca	Werribee	Werribee	Werribee	Werribee
Effective	100	100	100	100	100	100	100
stress (kPa)							
Specimen	50	50	50	50	50	50	50
thick. (mm)							
Hyd.	200	200	200	200	200	200	200
Gradient, i							
k <sub>w</sub>	11.3	10.3	10.0	8.26	9.01	9.13	9.20
x 10 <sup>-11</sup> (m/s)							
pv	1.07	0.8	1.0	0.82	1.00	0.85	0.8

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A 5.1: Hydraulic Conductivity Test Results with Flexible Wall Permeameter Using Water as Permeant