Swelling Pressure and Retaining Wall Design in Expansive Soils

A thesis submitted in fulfilment of the requirements for the degree of

Master of Engineering

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Bachelor of Engineering (Civil and Infrastructure) at RMIT University

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College of Science Engineering and Health

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Declaration

I certify that except where due acknowledgement has been made, the work is that of the author alone; the work has not been submitted previously, in whole or in part, to qualify for any other academic award; the content of the thesis is the result of work which has been carried out since the official commencement date of the approved research program; any editorial work, paid or unpaid, carried out by a third party is acknowledged; and, ethics procedures and guidelines have been followed.

Ya Tan

December 2016
Abstract

A retaining wall constructed on expansive soil can be subjected to lateral swelling pressures due to soil swelling, which may cause a significant horizontal deformation and bending of the retaining wall. In Australia, it is generally assumed that the backfill behind a retaining wall is non-expansive material. Therefore lateral swelling pressures induced by expansive soils are ignored in routine design. However in cases where expansive soils are present behind a retaining wall, swelling pressure should be evaluated based on soil properties and/or laboratory tests so that the wall can be designed properly to withstand these swelling pressures, which can be significant.

In this study, a retaining wall model, 1 m × 1.415 m × 1 m, was set up for the laboratory experiment. Expansive soils were filled into the box in 5 layers to achieve the required soil density. Six load cells were installed on the back of the retaining walls to measure the lateral pressures developed along the depth of the wall. A laser level was placed on a top concrete block to measure any vertical movement of soil. Two LVDT devices placed at the rear of the retaining wall were used to monitor the lateral movement of the retaining wall. Soil suction changes were monitored using psychrometer and the filter paper method. Two laboratory experiments were conducted to evaluate how swelling pressures were developed behind a retaining wall backfilled with expansive soils with different densities. Each test was run for 80 days.

A series of oedometer tests were also carried out to measure swelling pressures developed within soil samples with different initial suctions and densities. The results of oedometer swelling tests indicate that swelling pressures increase with the initial suction and dry densities. In addition to retaining wall model tests and oedometer tests, a series of other laboratory tests were performed which include shear box test, shrink-swell test, soil-water characteristic curve, liquid limit, plastic limit, linear shrinkage, soil suction measurement, and X-ray diffraction (XRD) test. Based on the laboratory tests, a new method/equation is developed for the prediction of the lateral swelling pressure of unsaturated expansive soils behind of retaining walls.
Acknowledgement

Foremost, I am truly indebted to my supervisor Dr. Jie Li, Thanks for his valuable supervision, critical suggestion, and immense help during my studies, especially in those periods when I doubted successful completion of my degree. I would also like to express my deep gratitude to my associate supervisor Dr. Annan Zhou, thanks for his guidance, continuous helps and patience. Without their persistent encouragement, this dissertation would not have been possible.

I gratefully acknowledge the assistance of the technical staff at RMIT University, Haman Tokhi, Kevin Le, Bao Nguyen and Kee Kong Wong and my fellow postgraduate students, Lei Guo, and Mr. Jian Zou. I would not be able to successfully conduct any laboratory test without their helps; their useful discussion and supports are invaluable.

Last but not least, I would thank to my family, thanks for my parents’ financial support during my studies in Australia, thanks to my wife Jiaying for her accompany and encourages me whole my study time at RMIT University. I am extremely grateful to the unconditional love, and support from my parents, my wife, my auntie Wei Yan, and my grandma. To my mother who had sacrificed her happiness to help me overcome my tough time, your love was an invaluable treasure in my life.

To the memory of my beloved mother, forever loved, missed and remembered.
Notation

$\sigma'$ effective stress

$\sigma_v'$ vertical stress

$\mu_a$ pore air pressure

$\mu_w$ pore water pressure

$\bar{\sigma}$ net stress

$s$ suction

$m_a$ the mass of container

$m_w$ the mass of container and wet soil

$m_c$ the mass of container and dry soil

$\theta_s$ saturated water content

$S_p$ predicted values of swelling potential

$L_s$ the longitudinal shrinkage of the specimen

$I_{sss}$ shrink swell index

$\varepsilon_{sw}$ swelling percentage

$\varepsilon_{sh}$ shrinkage percentage

$\gamma_d$ dry density

$K_0$ lateral earth pressure coefficient at rest

$K_a$ active lateral earth pressure coefficient

$K_p$ pasive lateral earth pressure coefficient

$P_a$ Rankine or Coulomb active force (kN/m)

$\alpha$ the angle of inclination of ground surface above the horizontal
\( \beta \) back face inclination of the structure

\( \delta \) Coulomb theory friction angle

\( \varphi \) Rankine theory soil friction angle

\( \tau_f \) shear stress along failure plane

\( c \) cohesion (kPa)

C clay content

\( D \) minimum horizontal distance

\( \theta \) volumetric water content

\( \psi \) total suction

\( \psi_o \) osmotic suction

\( S_r \) degree of saturation

\( PI \) plastic index

\( \varepsilon_v \) volumetric strain

\( \varepsilon_v^e \) volumetric strain in elastic region

\( t_f \) the time taken to reach failure, in minutes

\( t_{50} \) the time taken to reach 50% consolidation, in minutes

\( \lambda \) logarithmic hardening constant

\( e \) void ratio

\( e_0 \) initial void ratio

\( e_f \) final void ratio

\( w_i \) initial water content

\( h \) soil sample ring height
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Chapter 1
Introduction

1.1 General

Expansive soil (also referred to as swelling soil or reactive soil) has long been recognized as important problem soil in geotechnical engineering. Expansive soil is a clay soil that has a tendency to expand as its water content increases, and shrink when it dries out. This volume changes can cause damage to lightweight structures such as residential building, retaining walls, road and pavement (Li and Cameron, 2002; Delaney et al. 2005). Damage to lightly loaded structures founded on expansive soils has been widely reported throughout the world (Li et al. 2014). The problem is particularly significant in Australia as approximately 20% of the surface area in Australia is covered by expansive soils (Li and Zhou, 2013; Cameron, 2015). A map of the distribution of expansive soils around world and in Australia is provided in Figures 1.1 and 1.2 respectively.

Figure 1.1: Location of expansive soils around the world (Chen 1975)
A retaining walls constructed on expansive soil may be subjected to lateral swelling pressures and friction forces due to the swelling of the soils, which may cause a large horizontal deformation and bending of the retaining wall. In Australia, it is generally assumed that the backfill behind a retaining wall is non-expansive material. Therefore lateral swelling pressures induced by expansive soils are ignored in conventional design. However, field evidences indicated that horizontal swelling pressures and strains can affect the performance of structures. For example, Chen (1988) found that in the highly expansive soil area of South Denver, USA, a number of residential houses had bowing basement walls with horizontal deflection as much as six inches (15.2 cm) due to the lateral swelling pressures. Therefore both theoretical and experimental research is needed to improve the current design methods.

Figure 1.2: Location of expansive soil in Australia (Cameron, 2015)
In cases where expansive soils are present behind a retaining wall, swelling pressures should be evaluated based on soil properties and laboratory tests so that the wall can be designed properly to withstand these swelling pressures, which can be significant.

1.2 Research Objectives
The main objective of this research study is to develop a model/method which can be used by the local geotechnical engineers to predict swelling pressure acting on the retaining wall.

The specific objective of this research study includes:

- To investigate the properties of expansive soil, by conducting relevant laboratory experiments including Atterberg limit test, shear box test, shrink-swell test, soil-water characteristic curve test, and X-ray diffraction.
- To measure the swelling pressures developed within soil samples with different initial moisture contents and densities.
- To evaluate the relationship between the index properties and swelling pressure of expansive soils.
- To simulate a retaining wall against lateral swelling pressure caused by expansive soil in the laboratory.

1.3 Thesis Arrangement
This thesis is divided into 6 chapters. A brief description of each chapter is given below.

Chapter 1 – Introduction
This chapter provides an introduction and the main objectives of the research with background information on expansive soils and routine design of retaining wall. Thesis arrangement is outlined in this chapter as well.
Chapter 2 – Literature Review

This chapter provides an elaborative review on properties of expansive soil. There are two major methods for estimating the swelling potential of expansive soil, i.e. direct measurements and indirect measurements. The direct measurements include oedometer swelling test and constant volume swell test. The indirect measurement methods such as index property method, PVC method and activity method use the plasticity index and liquid limit to estimate the swelling potential of expansive soils. The traditional retaining wall design methods are introduced in this chapter as well.

Chapter 3 – Laboratory Testing

In this chapter, the laboratory testing is discussed in details, which includes set-up and preparation of experiments, soil used for testing, and the test procedures. The results of tests are presented and discussed as well.

Chapter 4 – Retaining Wall Experiments

In this study a model retaining wall was built at RMIT geotechnical laboratory to simulate the retaining wall against lateral swelling pressure developed within soil. Chapter 4 describes the experimental set-up and procedure. The experimental results are also presented and discussed in Chapter 4.

Chapter 5 – Development of New Model for Prediction of Lateral Swelling Pressure

In this chapter, a new model is proposed to estimate/predict the lateral swelling pressures behind a retaining wall, which is based on shrinkage-swell index, initial moisture content and the dry density.

Chapter 6 – Conclusions and Recommendations

This chapter provides the final conclusions and recommendations for future research.
Chapter 2
Literature Review

2.1 Introduction

Expansive soil has attracted the attention of many researchers and practitioners all over the world, studying the behaviors of expansive soil and its damage to infrastructure such as buildings, road, pavements and earth retaining structures. Many factors have an effect on the behaviors of expansive soil, including moisture content (soil suction), dry density, liquid limit and plasticity index and swelling potential. In this chapter, classification methods of expansive soils and current retaining wall design methods are outlined.

2.2 Swelling Potential Expansive soil and Classification Methods

Expansive soil has been of great concern to design and geotechnical engineers for many years. In fact, the first research ever that attempted to study the behaviors of expansive soil was to prevent damages cause by expansive soil to earth retaining structure of infrastructure which was presented in the First International Conference on Soil Mechanics and Foundation Engineering (ISSMGE) held in June 1936 at Harvard University.

Prediction of swelling potential and swelling pressure has been a problem to civil engineering for a long time. Swelling potential of expansive soil depends on a number of factors such as the type and amount of clay minerals, soil structures (i.e. particle arrangement, bonding, and fissures) and nature of pore fluid, and exchangeable cations (Mansour 2011).

A number of different methods have been proposed for prediction of swelling potential and swelling pressure (Chen 1975). Generally, there are two major methods for estimating the swelling potential of expansive soil:
1) Indirect measurement method in which one or more of the related intrinsic properties of expansive soils are measured and complemented with experience to estimate swelling potential.

2) Direct measurement method which involves actual measurement of volume change of expansive soil.

2.2.1 Indirect Measurement Method

There are several indirect measurement methods which are commonly used to estimate the swelling potential of expansive soils, such as index property method, PVC method and activity method. These methods are discussed in the following sections.

2.2.1.1 Index Property Method based Atterberg Limits Test

Atterberg limit test is the most common test to determine basic soil properties (i.e. liquid limit, plastic limit and plasticity index) in current engineering practice. Plasticity index (PI) and liquid limit (LL) have been used to classify reactivity or expansiveness of expansive soils by the Federal Housing Administration (Meehan and Karp 1994) since early 1970s. According to Table 1, soils can be classified into five classes/groups based on their PI and LL.

**Table 1: Classification of expansive soils (FHA 1973) – from Meehan & Karp (1994)**

<table>
<thead>
<tr>
<th>Classification</th>
<th>Plasticity index</th>
<th>Liquid limit</th>
<th>Soil Group</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-expansive soil</td>
<td>0-6</td>
<td>0-25</td>
<td>A</td>
</tr>
<tr>
<td>Marginal</td>
<td>6-10</td>
<td>25-30</td>
<td>B</td>
</tr>
<tr>
<td>Moderately expansive</td>
<td>10-25</td>
<td>30-50</td>
<td>C</td>
</tr>
<tr>
<td>Highly expansive</td>
<td>&gt;25</td>
<td>&gt;50</td>
<td>D</td>
</tr>
<tr>
<td>Expansive claystone</td>
<td>&gt;50</td>
<td>&gt;70</td>
<td>E</td>
</tr>
</tbody>
</table>
Chen (1975) also proposed to adopt PI and LL as indicators to describe the swelling potential of expansive soils (see Table 2). Based on Table 2, the soil is essentially non-expansive when the liquid limit is less than 20 percent and the plasticity index is less than 6 percent.

<table>
<thead>
<tr>
<th>Degree of expansion</th>
<th>Liquid limit</th>
<th>Plasticity index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt;30</td>
<td>0-15</td>
</tr>
<tr>
<td>Medium</td>
<td>30-40</td>
<td>10-35</td>
</tr>
<tr>
<td>High</td>
<td>40-60</td>
<td>20-55</td>
</tr>
<tr>
<td>Very High</td>
<td>&gt;60</td>
<td>&gt;35</td>
</tr>
</tbody>
</table>

Snethen (1984) evaluated a number of published criteria for classifying swelling potential and found that liquid limit and plasticity index are best indicators of swelling potential and soil suction at natural moisture content can be used as decent indicators of potential swell if natural conditions are taken into account as well. A statistical analysis of a large amount of testing data resulted in the classification system as shown in Table 3. The US Department of the Army (1983) and the American Association of State Highway and Transportation Officials (AASHTO, 2008) both adopted the criteria (Table 3) proposed by Snethen (1984).

<table>
<thead>
<tr>
<th>Potential swell classification</th>
<th>Liquid limit (%)</th>
<th>Plasticity index (%)</th>
<th>Natural soil suction (tsf)</th>
<th>Potential swell (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>&gt;60</td>
<td>&gt;35</td>
<td>&gt;4</td>
<td>&gt;1.5</td>
</tr>
<tr>
<td>Marginal</td>
<td>50-60</td>
<td>25-35</td>
<td>1.5-4</td>
<td>0.5-1.5</td>
</tr>
<tr>
<td>Low</td>
<td>&lt;50</td>
<td>&lt;25</td>
<td>&lt;1.5</td>
<td>&lt;0.5</td>
</tr>
</tbody>
</table>
Table 4 was proposed by Kay (1990). He suggested that the liquid limit is a decent indicator of shrink-swell response for natural soil although the test was conducted on reconstituted soil.

<table>
<thead>
<tr>
<th>Site Classification</th>
<th>Liquid limit range</th>
</tr>
</thead>
<tbody>
<tr>
<td>S (slightly expansive)</td>
<td>&lt;20</td>
</tr>
<tr>
<td>M (moderately expansive)</td>
<td>20-40</td>
</tr>
<tr>
<td>H (highly expansive)</td>
<td>40-70</td>
</tr>
<tr>
<td>E (extremely expansive)</td>
<td>&gt;70</td>
</tr>
</tbody>
</table>

Based on the above tables, Mansour (2011) proposed to use the following table for classification of expansive soils

<table>
<thead>
<tr>
<th>Classification</th>
<th>Plasticity index (%)</th>
<th>Liquid limit (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-expansive</td>
<td>0-6</td>
<td>0-25</td>
</tr>
<tr>
<td>Low</td>
<td>&lt;25</td>
<td>25-50</td>
</tr>
<tr>
<td>Marginal</td>
<td>25-35</td>
<td>50-60</td>
</tr>
<tr>
<td>High</td>
<td>&gt;35</td>
<td>&gt;60</td>
</tr>
</tbody>
</table>

2.2.1.2 Potential Volume Change Method (PVC)

The potential volume change (PVC) method was developed by Labme (1961) which has been extensively used by the Federal Housing Administration as well as many US State Highway Departments. The PVC test consists of placing a remolded soil sample into an oedometer ring, wetting it and allowing the sample to swell against a proving ring. The swell index is essentially a measurement of the swelling pressure exerted by the soil sample and is correlated to qualitative range of potential volume change using a chart given by Lambe (1961).
Table 6 shows the swelling categories based on the PVC method (Lambe 1961). It should be pointed out that the PVC values should only be used for comparison of various swelling soils and not be used as design parameters since the test uses remolded samples.

### Table 6: Values of PVC rating (Lambe 1961)

<table>
<thead>
<tr>
<th>Category</th>
<th>PVC Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very critical</td>
<td>&gt;6</td>
</tr>
<tr>
<td>Critical</td>
<td>4-6</td>
</tr>
<tr>
<td>Marginal</td>
<td>2-4</td>
</tr>
<tr>
<td>Non-critical</td>
<td>&lt;2</td>
</tr>
</tbody>
</table>

2.2.1.3 Activity Method

The activity method was proposed by Seed et al (1962). This method was based on remolded artificially prepared soils. The activity is defined as

\[
Activity = \frac{PI}{C-10}
\]  

(2.1)

Where

\( PI = \text{Plasticity index (\%)}; \)

\( C = \text{the percentage clay size finer than 0.002mm}. \)

2.2.2 Direct Measurement Methods

The direct methods involve actual measurement of the swell parameters of expansive soil. Therefore they are more reliable than the indirect methods. The disadvantage of the direct methods is that they usually require special equipment and time consuming (i.e. not suitable for quick identification of expansive soils).
There are three types of oedometer tests which are used to evaluate the swelling characteristics: (1) constant volume swell test, (2) improved swell oedometer test and (3) swell overburden test. These tests are outlined in ASTM D4546 (2003) accept the shrink-swell test which is introduced in Australian Standard AS 1289.7.1 (2003).

2.2.2.1 Constant Volume Swell Test

In this technique, soil specimen is placed into the oedometer cell and its vertical expansion when inundated with water is prevented by progressively loading the specimen. Once no more expansion is observed when saturation is completed, the final total pressure applied to the specimen represents a direct measure of swelling pressure.

2.2.2.2 Improved Swell Oedometer Test

In the improved oedometer test, the sample is allowed to swell freely in the vertical direction under a seating load of 7 kPa until the primary swell is completed. The sample is then loaded to its initial height. The pressure that brings the sample to its initial height (i.e. zero swell) can be taken as the swelling pressure.

2.2.2.3 Swell Overburden Test

In the swell overburden test, the specimen is loaded to in situ overburden pressure or a predetermined surcharge pressure, inundated with water under this pressure until the primary swell is completed. The specimen is then loaded until it reaches to its original height (i.e. at 0% vertical swell).
2.2.2.4 Shrinkage-Swell Test

In Australia, methods for directly testing the swelling potential of expansive soils are given in AS1289.7.1.1 (2003). The shrink-swell test consists of a core shrinkage test and a swelling test. During the core shrinkage test, a cylindrical core sample is allowed to dry out in air on a smooth surface for a period of approximately 1-2 weeks. Two drawing pins are placed in the centre of the core at either end and the distance between the rounded heads of the pins is regularly measured with a digit vernier calliper to the nearest 0.01 mm. The core shrinkage test is used to obtain the shrinkage strain. In the swelling test, the specimen is placed into a consolidation cell which is inundated with water and allowed to swell under a vertical pressure equal to the overburden pressure or 25 kPa. The swelling displacement is regularly recorded.

After obtaining the shrinkage strain and swelling strain, the shrink-swell index of a soil can be calculated by the following equation (AS1289.7.1.1, 2003):

\[
I_{ss} = \frac{\varepsilon_{sw}}{2} + \varepsilon_{sh} \quad (2.3)
\]

Where

\[I_{ss} = shinkage - swell index;\]

\[\varepsilon_{sw} = swelling strain;\]

\[\varepsilon_{sh} = shinkage strain.\]

2.3 Suction in Expansive Soils

A retaining wall constructed on expansive soil is subjected to the lateral swelling pressure due to soil swelling caused by a change in soil suction. Soil suction is an important parameter in describing the moisture stress condition of unsaturated soils. Soil suction, usually expressed in pF or kPa, is a measure of a soil’s affinity for moisture.
Therefore, “dry” soils which have a higher affinity to absorb moisture have high values of kPa (pF), while “wet” soils which have a low affinity to absorb more moisture have low values of kPa (pF).

The total suction is equal to the sum of osmotic (or solute) suction and matric suction. Osmotic suction is a measure of the potential to build water in the soil due to the osmotic effects of the solutes in the bulk soil solution. This type of suction refers to the potential of the soil and water to bind due to osmotic effects of the solutes in the bulk soil solution. Matric suction refers to the soil’s affinity for water at the same salinity level (AS2870, 2011) and relies upon capillary forces generated by the small radius of the interface between water and air within the voids in the soil.

Soil suction can be measured both directly and indirectly. Direct methods used to measure matrix suction include tensiometer, pressure plate, pressure membrane, and psychrometers technique. Indirect matric suction measurements include filter paper technique, electrical conductivity sensors, time domain reflectometry (TDR) and thermal conductivity sensor. Osmotic suction can be measured indirectly using several methods, such as saturation extract method and pore fluid squeezer technique.
2.4 Retaining Wall Design

2.4.1 Introduction

Retaining walls have been widely used in civil engineering and construction for stabilizing slopes and excavations. Estimating lateral earth pressures has been of geotechnical interest for well over a century because a safe retaining wall design requires accurate prediction of earth pressure imposed on the retaining wall by soils.

There are two methods for calculation of lateral earth pressures which are widely used in routine design. One was developed by Coulomb (1776) and the other by Rankine (1856). These two methods differ in the assumptions made to simplify the problem, but both are very simple in application. Coulomb’s method for the determination of lateral earth pressure was based on the limit equilibrium, which includes wall friction, wall slope, and backfill slope. Rankine’s method was based on the stress states of the backfill materials which do not consider wall friction. The failure plane assumed by both Coulomb and Rankine methods is planner surface.

There are various types of retaining walls. Retaining walls can be broadly classified into three categories: mass gravity, flexible or sheet pile and mechanically stabilized earth walls. The gravity retaining wall is a massive concrete wall relying on its mass to resist the lateral force from the retain soil mass. Flexible or sheet pile wall is long, slender wall relying on passive resistance and anchors or props for its stability. Mechanically stabilized earth is a gravity-type retaining wall in which the soil is reinforced by thin reinforcing elements. (Budhu 2010)
2.4.2 Lateral Earth Pressure for Retaining Walls

The lateral earth pressures on a vertical wall that retains a soil mass dealt with two theories that are Coulomb’s earth pressure theory and Rankine’s earth pressure theory. The pressures on the retaining walls cause movements and its three different lateral earth pressures conditions are shown in Figure 2.1 (Budhu 2010).

![Figure 2.1: Stresses on soil elements in front of and behind retaining wall (Budhu 2010)](image)

If the wall remains the rigid and no movement, then the effective stresses at rest on element A and element B are:

\[
\sigma'_{xz} = \sigma'_1 = \gamma' z \\
\sigma'_x = \sigma'_3 = K_0 \sigma'_1 = K_0 \gamma' z
\]

Where

\( \gamma' \) is the effective unit weight of soil;

\( K_0 \) is the lateral earth pressure at rest.
Based on Figure 2.2, the $K_a$ and $K_p$ is given:

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right)$$  (2.3)

$$K_p = \tan^2\left(45 + \frac{\phi'}{2}\right)$$  (2.4)

Where

$K_a$ is active lateral earth pressure coefficient;

$K_p$ is passive lateral earth pressure coefficient.

Therefore $K_a = 1/K_p$. 

**Figure 2.2: The Mohr-Coulomb failure envelope (Budhu 2010)**
2.4.3 Coulomb Theory

Coulomb’s analysis of lateral earth forces on a retaining structure is based on limit equilibrium. And a soil mass behind a vertical retaining wall will slip along a plane inclined an angle \( \theta \) to the horizontal as Figure 2.3. Then the slip plane was determined by the maximum thrust acts on the plane. The basic tenets of the model consist of:

1. selection of a plausible failure mechanism,
2. determination of the forces acting on the failure surface, and
3. use of equilibrium equations to determine the maximum thrust.

![Figure 2.3: Coulomb failure wedge (Budhu 2010)](image)

\[
\sum F_x = P + T \cos \theta - N \sin \theta = 0 \quad (2.5)
\]

\[
\sum F_z = W - T \sin \theta + N \cos \theta = 0 \quad (2.6)
\]

The weight of sliding mass of soil is

\[
W = \frac{1}{2} \gamma H_0^2 \cot \theta \quad (2.7)
\]

At limit equilibrium,

\[
T = N \tan \phi' \quad (2.8)
\]
Solving for $P$

$$P = \frac{1}{2} \gamma H_0^2 \cot \theta \tan(\theta - \phi')$$  \hspace{1cm} (2.9)

The maximum thrust and the inclination of the slip line was determined by using calculus to differentiate Equation 2.9 with respect to $\theta$.

$$\frac{\partial P}{\partial \theta} = \frac{1}{2} \gamma H_0^2 \left[ \cot \theta \sec^2(\theta - \phi') - \csc^2 \theta \tan(\theta - \phi') \right] = 0$$  \hspace{1cm} (2.10)

Which leads to

$$\theta = \theta_{cr} = 45 + \frac{\phi'}{2}$$  \hspace{1cm} (2.11)

Substituting $\theta$ into Equation 2.9,

$$P = P_a = \frac{1}{2} \gamma H_0^2 \tan^2 \left( 45^\circ - \frac{\phi'}{2} \right) = \frac{1}{2} K_a \gamma H_0^2$$  \hspace{1cm} (2.12)

From the Figure 2.4, the retaining structure has wall friction ($\delta$), the wall face inclination of vertical with an angle $\eta$ and back fill is sloping at an angle $\beta$. Based on Coulomb limit equilibrium approach, Poncelet (1840) obtained expressions for $K_a$ and $K_p$ and the lateral earth pressure coefficients was estimated below:

$$K_{ac} = \frac{\cos^2(\phi' - \eta)}{\cos^2 \eta \cos(\eta + \delta)[1 + \left\{ \frac{\sin(\phi' + \delta) \sin(\phi' - \beta)}{\cos(\eta + \delta) \cos(\eta - \beta)} \right\}^{1/2}]^2}$$  \hspace{1cm} (2.13)

$$K_{pc} = \frac{\cos^2(\phi' + \eta)}{\cos^2 \eta \cos(\eta - \delta)[1 - \left\{ \frac{\sin(\phi' + \delta) \sin(\phi' + \beta)}{\cos(\eta - \delta) \cos(\eta - \beta)} \right\}^{1/2}]^2}$$  \hspace{1cm} (2.14)

Where $K_{pc} \neq 1/K_{ac}$ and lateral earth pressure coefficients are applied to the principal effective stress.

$K_{ac}$ = lateral earth pressure coefficient at the active state

$K_{pc}$ = lateral earth pressure coefficient at the passive state

$\phi'$ = effective angle of friction;

$\delta$ = wall friction angle;
\[ \eta = \text{slope inclination}; \]
\[ \beta = \text{back face inclination of the structure}. \]

\[ \tan \theta = \frac{\left( \sin \phi' \cos \delta \right)^{1/2}}{\cos \phi' \{ \sin (\phi' + \delta) \}^{1/2}} \pm \tan \phi' \tag{2.15} \]

Where the positive sign is refer to the active state (\( \theta = \theta_a \)) and the negative sign refer to the passive state (\( \theta = \theta_p \)).

Both of the active state and passive state has curved slip planes caused by wall friction and the curvature of active state is smaller than passive state. The curved slip surface leaded inaccurate value of \( K_{ac} \) and \( K_{pc} \). However the error for the active state is small and can be neglect, and an overestimation of the passive state using Coulomb’s analysis. So many investigators attempted to find more accurate \( K_p \) using different methods such that Caquot and Kerisel (1948) used logarithm spiral slip surface method and Packshaw
(1969) used circular failure surface method. Table 7 showed below lists factors that Kpc can be used to correct it for logarithm spiral slip surface.

Table 7: Correction factors to be applied to $K_pC$ to approximate a logarithm spiral slip surface for a backfill with a horizontal surface and sloping wall face (Budhu 2010)

<table>
<thead>
<tr>
<th>$\phi'$</th>
<th>-0.7</th>
<th>-0.6</th>
<th>-0.5</th>
<th>-0.4</th>
<th>-0.3</th>
<th>-0.2</th>
<th>-0.1</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>0.96</td>
<td>0.93</td>
<td>0.91</td>
<td>0.88</td>
<td>0.85</td>
<td>0.83</td>
<td>0.8</td>
<td>0.78</td>
</tr>
<tr>
<td>20</td>
<td>0.94</td>
<td>0.9</td>
<td>0.86</td>
<td>0.82</td>
<td>0.79</td>
<td>0.75</td>
<td>0.72</td>
<td>0.68</td>
</tr>
<tr>
<td>25</td>
<td>0.91</td>
<td>0.86</td>
<td>0.81</td>
<td>0.76</td>
<td>0.71</td>
<td>0.67</td>
<td>0.62</td>
<td>0.57</td>
</tr>
<tr>
<td>30</td>
<td>0.88</td>
<td>0.81</td>
<td>0.75</td>
<td>0.69</td>
<td>0.63</td>
<td>0.57</td>
<td>0.52</td>
<td>0.47</td>
</tr>
<tr>
<td>35</td>
<td>0.84</td>
<td>0.75</td>
<td>0.67</td>
<td>0.6</td>
<td>0.54</td>
<td>0.48</td>
<td>0.42</td>
<td>0.36</td>
</tr>
<tr>
<td>40</td>
<td>0.78</td>
<td>0.68</td>
<td>0.59</td>
<td>0.51</td>
<td>0.44</td>
<td>0.38</td>
<td>0.32</td>
<td>0.26</td>
</tr>
</tbody>
</table>

The later force is inclined at an angle $\delta$ from the normal to the wall face. The direction of the friction force depended on whether the wall moves relative to the soil or the soil moves relative to the wall. As can be seen from Figure 2.4, that the active wedge moves downward relative to the wall and the passive wedge moves upward relative to the wall. The active lateral force has the positive sense of the inclination and the passive lateral force negative sense of the inclination, and the application point of these forces is $H_0/3$ from the base of the wall.
2.4.4 Rankine Theory

Rankine earth pressure theory is usually used to determine the lateral earth pressures for vertical, frictionless wall supporting a dry, homogeneous soil with a horizontal surface. According to the Rankine theory, the expressions for $K_{ar}$ and $K_{pR}$ are given below with reference to Figure 2.5

![Figure 2.5: Retaining wall with sloping soil surface, frictionless soil-wall interface, and sloping back for use with Rankine’s method (Budhu 2010)](image)

$$K_{ar} = \frac{\cos(\beta - \eta) \sqrt{1 + \sin^2 \phi' - 2 \sin \phi' \cos \omega_a}}{\cos^2 \eta (\cos \beta + \sqrt{\sin^2 \phi' - \sin^2 \beta})}$$

(2.16)

and

$$K_{pR} = \frac{\cos(\beta - \eta) \sqrt{1 + \sin^2 \phi' + 2 \sin \phi' \cos \omega_p}}{\cos^2 \eta (\cos \beta - \sqrt{\sin^2 \phi' - \sin^2 \beta})}$$

(2.17)

Where

$K_{ar} =$ lateral earth pressure coefficient at the active state

$K_{pR} =$ lateral earth pressure coefficient at the passive state

$\phi' =$ effective angle of friction;
The angles of the failure planes to the horizontal for active and passive states are:

\[ \theta_a = \frac{\pi}{4} + \frac{\phi'}{2} + \frac{\beta}{2} - \frac{1}{2} \sin^{-1} \left( \frac{\sin \beta}{\sin \phi'} \right) \]  
\[ \theta_p = \frac{\pi}{4} - \frac{\phi'}{2} + \frac{\beta}{2} + \frac{1}{2} \sin^{-1} \left( \frac{\sin \beta}{\sin \phi'} \right) \]

The inclinations of the normal of the wall face for the forces:

\[ \xi_a = \tan^{-1} \left( \frac{\sin \phi' \sin \theta_a}{1 - \sin \phi' \cos \theta_a} \right) \]  
\[ \xi_p = \tan^{-1} \left( \frac{\sin \phi' \sin \theta_p}{1 + \sin \phi' \cos \theta_p} \right) \]

In the case of a wall with a vertical face, \( \eta = 0 \),

\[ K_{aR} = \frac{1}{K_{pR}} = \cos \beta \left( \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi'}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi'}} \right) \]  

The active and passive lateral earth forces work on a direction parallel to the soil surface, the inclination of angle \( \beta \) to the horizontal, therefore \( P_{ax} = P_a \cos \beta \) and \( P_{az} = P_a \sin \beta \).
2.4.5 Retaining Wall Design

Generally, there are three classes of retaining walls, i.e. cast-in-place (CIP) gravity and semi-gravity walls, and flexible retaining wall (Budhu 2010). CIP gravity and semi-gravity walls are rigid and constructed by concrete relying on gravity for stability. Flexible retaining walls are constructed by long and slender members and rely on passive soil resistance and anchors for stability (Figure 2.6). The common materials used of flexible retaining wall consist of steel, concrete, and timber.

Figure 2.6: Types of rigid retaining walls (Budhu 2010)
According to Das 2007, there are four types of retaining wall as follows:

1. The gravity wall depends mostly on its own weight for stability. It is the oldest and most frequently used retaining walls. Gravity retaining walls can be built of stone or concrete. It has the advantage of using local materials, construction convenience, and good economic results. Therefore, gravity retaining wall has been widely used in railways, highways, water conservancy, harbor, mining and other projects. As the gravity retaining wall to maintain balance and stability by its own weight, therefore, the volume and weight are large; the construction of such heavy wall on the soft foundation is often limited by the bearing capacity of soil foundation.

2. The structural stability of the cantilever retaining wall is ensured by the weight of the wall and the gravity of the fill above the heel plate, and the wall toe plate also
significantly increases the anti-overturning stability and greatly reduces the stress of the foundation. Its main advantages are simple structure, convenient construction, smaller wall section, its light weight, can play a better strength property of materials, can adapt to lower bearing capacity of the foundation. It is suitable for the lack of stone and earthquake areas.

3. Counterfort retaining walls are similarities with cantilever wall for construction. Counterfort retaining walls can restrict more bearing capacity and shear force comparison with cantilever retaining wall.

4. Sheet piling retaining walls are built in comparatively soft soils such as expensive soils and tight spaces, which the function is for temporary structures like braced cuts. Moreover, they have another usage with a varying purpose for excavation support system and floodwalls also cut-off walls. Cantilever and anchored sheet pile are the two basic types of sheet pile walls can be used for a wall height larger than 4.5m (Day 1994).
There are four modes of failure for a rigid retaining wall: (a) translational failure, (b) rotation and bearing capacity failure, (c) deep-seated failure, and (d) structural failure.

Figure 2.8: The failure modes of rigid retaining wall (Budhu 2010)
The modes of failure for sheet pile walls are shown in as follows:

![Image of failure modes]

Figure 2.9: The failure modes of flexible retaining wall (Budhu 2010)

2.4.6 Stability of rigid retaining walls

Translation

A rigid retaining wall must have adequate resistance against translation, which is the sliding resistance of the base of the wall greater than the lateral force pushing on the wall. The factor of safety against translation is:

\[(FS)_T = \frac{T}{P_{ax}}; \quad (FS)_T \geq 1.5\]  \quad (2.25)

Where
T is the sliding resistance at the base;
\( P_{ax} \) is the lateral force pushing on the wall.

\[
T = R_z \tan \phi'_b , \text{ for an ESA;} \tag{2.26}
\]

\[
T = s_w B , \text{ for a TSA} \tag{2.27}
\]

Where

\( R_z \) is the resultant vertical force;
\( s_w \) is the adhesive stress;

B is the horizontal width of the base;

\( \phi'_b \) is the interfacial friction angle between the base of the wall and the soil.

\[
\phi'_b \approx \frac{1}{2} \phi'_{cs} \text{ to } \frac{2}{3} \phi'_{cs}
\]

Typical sets of forces acting on gravity and cantilever rigid retaining walls are showed in Figure 2.10

![Diagram of forces on rigid retaining walls](image)

**Figure 2.10: Forces on rigid retaining walls**

For an ESA,

\[
(\text{FS})_T = \frac{[(W_w + W_s + P_{az}) \cos \theta_b - P_{ax} \sin \theta_b] \tan \phi'_b}{P_{ax} \cos \theta_b + (W_w + W_s + P_{az}) \sin \theta_b} \tag{2.28}
\]

Where

\( W_w \) is the weight of the wall,
\( W_s \) is the weight of the soil wedge,

\( P_{az} \) is the vertical components of the active lateral force,

\( P_{ax} \) is the horizontal components of the active lateral force.

\( \theta_b \) is the inclination of the base to the horizontal.

If \( \theta_b \) is 0 (the base of a retaining wall is horizontal), then:

\[
(FS)_T = \frac{[(W_w + W_s + P_{az}) \tan \phi_b]}{P_{ax}}
\]  

(2.29)

For a TSA,

\[
(FS)_T = \frac{s_w B / \cos \theta_b}{P_{ax} \cos \theta_b + (W_w + W_s + P_{az}) \sin \theta_b}
\]  

(2.30)

If \( \theta_b \) is 0, then:

\[
(FS)_T = \frac{s_w B}{P_{ax}}
\]  

(2.31)

The embedment of rigid retaining walls is generally small and passive lateral force is not taken into account. For gravity retaining structures, with increasing width \( B \) of the wall, the base resistance can be increased. For cantilever walls, the shear key can be constructed to provide additional base resistance against sliding.

**Rotation**

A rigid retaining wall must have adequate resistance against rotation. The rotation of the wall about its toe is satisfied if the resultant vertical force lies within the middle third of the base. The resultant vertical force at the base is located at

\[
\bar{x}_{\theta} = \frac{W_w x_w + W_s x_s + P_{az} x_a - P_{ax} \bar{z}_a}{(W_w + W_s + P_{az}) \cos \theta_b - P_{ax} \sin \theta_b}
\]  

(2.32)

Where

\( \bar{z}_a \) is the location of the active lateral earth force from the toe;

If

\[
B/3 \leq \bar{x} \leq 2B/3
\]

That is
\[ e = \left| \left( \frac{B}{2} - \bar{x} \right) \right| \leq B/6 \]

Where \( e \) is the eccentricity of the resultant vertical load, and \( \bar{x} = \bar{x} \cos \theta_b \).

If \( \theta_b \) is 0, then:

\[
\bar{x} = \frac{W_w x_w + W_s x_s + P_{az} x_a - P_{ax} \bar{x}_a}{W_w + W_s + P_{az}}
\]  

(2.33)

**Bearing capacity**

A rigid retaining wall must have a sufficient margin of safety against soil bearing capacity failure. The maximum pressure imposed on the soil at the base of the wall must not exceed the allowable soil bearing capacity, that is,

\[
(\sigma_z)_{\text{max}} \leq q_a
\]  

(2.34)

Where

\((\sigma_z)_{\text{max}}\) is the maximum vertical stress imposed,

\(q_a\) is the allowable soil bearing capacity.
3.1 Introduction

A series of laboratory tests were conducted in this research, which include liquid limit, plastic limit, linear shrinkage, X-ray diffraction (XRD) test, shrink-well test, shear box test and soil-water characteristic curve. In this chapter, the laboratory testing is discussed in details, which includes set-up and preparation of experiments, soil used for testing, and the test procedures. The results of tests are presented and discussed as well.

The main objective of this research is to study the swelling pressures of expansive soils. Swelling in expansive clays is a result of changes in the soil suction or water content. The swelling pressure can be defined as the pressure required re-compressing, the fully swollen sample to its original volume.

In this study, oedometer test was used to measure the swelling pressure of expansive soil. The soils samples used in the laboratory test were collected from the fields at either Glenroy (a suburb 13 km north of Melbourne), or Braybrook (a suburb 9 km west of Melbourne).
3.2 Atterberg Limit Test

As discussed in Section 2.2.1.1, the Atterberg limits are a basic measure of the nature of a fine-grained soil. Atterberg Limits Tests consist of Plastic Limit (PL) test and Liquid Limit (LL) test. In this study, the plastic limit test, liquid limit test and linear shrinkage test were conducted in accordance with the Australian Standards AS1289. The soil was firstly dried in an oven at a temperature of 105 °C for two days and was then crushed. The crushed soil was sieved using a 425μm screen to remove all coarse materials and vegetable matter (see Figure 3.1)

![Glanroy Soil after oven dry](image1)

![425μm sieve](image2)

![Soil after sieve](image3)

Figure 3.1: Soil sample preparation
3.2.1 Plastic Limit Test

The moisture content at which soil begins to behave as a plastic material is defined as the plastic limit (PL). PL is determined by rolling out a thread of the soil sample on a flat and smooth surface. The plastic limit is defined as the moisture content where the thread just begin to crumble (as shown in Figure 3.2) at a diameter of 3 mm (Budhu 2007). According to AS 1289.3.2.1-2001, the soil sample is rolled by fingers on a glass plate with rolling rate between 80 to 90 strokes per minutes. A small container that was weighted before the testing was used to collect the soil thread and to determine the moisture content.

![Plastic limit test](image)

**Figure 3.2: Plastic limit test**

The plastic limit $w_p$ can be calculated based on AS 1289.3.2.1 (2001):

$$w_p = \frac{m_b - m_c}{m_c - m_a}$$  \hspace{1cm} (3.1)

Where

$w_p$ = moisture content for plastic limit, in percent

$m_b$ = mass of container and wet soil, in grams

$m_c$ = mass of container and dry soil, in grams

$m_a$ = mass of container, in grams
3.2.2 Liquid Limit Test

The liquid limit (LL) is the moisture content at which soil begins to change from the liquid state to a plastic state. The liquid limit of a soil can be determined by using Four Point or One Point Casagrande Method. According to AS1289.3.1.2, the Casagrande cup needs to be checked that the 10 mm calibration gauge will just slide between the base of the device and the completely raised cup. This gap should be checked each time before using the device to ensure accuracy of results. It is also important to check that the grooving tool tip is 2.5mm to ensure the soil sample is closing over a gap of 2.5mm. Figure 3.3 shows the apparatus used for liquid limit test.

![Figure 3.3: Liquid limit test](image)

LL is defined as the moisture content at which two sides of a groove come close together (Figure 3.3) under the impact of 25 numbers of blows. AS1289.3.1.2 gives the following equation for determination the liquid limit:

\[ w_L = w \left( \frac{n}{25} \right) \tan \beta \]  

(3.2)

Where

\( w = \) water content;
n = number of blows to closure;
\[ \tan \beta = 0.091. \]

### 3.2.3 Linear Shrinkage Test

The shrinkage limit (SL) is the moisture content of a soil at which further loss of moisture does not result in any more volume reduction.

**Test procedures:**

Obtain a sample of at least 250 g from the soil passing the 425 µm sieve which has been prepared in accordance with the procedure prescribed in AS 1289.1 for the preparation of disturbed soil samples. Alternatively for undisturbed samples, crush the sample using either or both a mortar and pestle or a machine designed to crush soils to sizes less than 425 µm.

(a) Perform the liquid limit test as described above and ensure a result close to the liquid limit is achieved.

(b) Lightly grease or oil the inside of a clean shrinkage mold and place the wet soil in it. Take care to thoroughly remove all air bubbles in the soil by lightly tapping the base of the mold. Slightly overfill the mold and then level off the excess material with a palette knife. Remove all soil adhering to the rim of the mold (see Figure 3.4).

(c) Allow the specimen to dry at room temperature for about 24 hours or until a distinct change in color can be noticed. Transfer into an oven and dry at 105°C, or allow to continue air-drying, until shrinkage ceases.

(d) Allow the specimen to cool and then measure its longitudinal shrinkage \( L_s \) to the nearest millimeter. If the specimen cracks into pieces, firmly hold the separate parts together and measure the shrinkage \( L_s \). If the specimen curls (see Figure 3.4), carefully remove it and measure the length of the top and bottom surfaces. Subtract the mean of these two lengths from the internal length of the mold to obtain the shrinkage \( L_s \).
The shrinkage limit (SL) can be determined by the following equation (from AS1289.3.4.1, 2008):

\[
SL = \frac{L_s}{L} \times 100
\]  

(3.3)

Where

L = the length of the mold;

\( L_s \) = the longitudinal shrinkage of the specimen.

The index properties for Glenroy soil and Braybrook soil are summarised in Table 8. Based on the Unified Soil Classification System, both Glenroy soil and Braybrook are classified as clay (Figure 3.5).

Table 8: Results of liquid limit, plastic limit and linear shrinkage tests

<table>
<thead>
<tr>
<th>Location</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>LS (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glenroy</td>
<td>56.54</td>
<td>16.92</td>
<td>39.61</td>
<td>11.20</td>
</tr>
<tr>
<td>Braybrook</td>
<td>80.75</td>
<td>18.02</td>
<td>62.73</td>
<td>22.10</td>
</tr>
</tbody>
</table>
Figure 3.5: Plasticity chart showing soil studied
3.3 X-ray Diffraction

X-ray diffraction is to study the mineral composition of the soil samples and to determine percentage crystallinity of samples.

X-ray diffraction analysis was conducted by using a Bruker AXS D4 Endeavour system, Cu-Kα radiation operated at 40 kV and 40 mA and a Lynxeye linear strip detector. All samples were in powder form. Microscope slides were used to flatten the surface of all samples before loading. Samples were tested between 10 and 90 degrees 2 theta with variable slit v20 with step size 0.02 s.

Figure 3.6: X-ray Diffraction of Glenroy Soil

Figure 3.6 shows the X-ray diffraction result of Glenroy soil. There is 53.1% crystallinity in the soil. The soil is mainly consists of quartz and muscovite.
From Figure 3.7, it can be seen that there is 46.9% crystallinity in Braybrook soil, which is less than that in Glenroy soil. Quartz leads soils to non-expansive. Based on the results of oedometer test, Braybrook soil can cause larger swelling pressure than Glenroy soil because of less quartz content in Braybrook soil.
3.4 The Shrink Swell Test

Australian Standard AS2870 (2011) provides three testing methods for the evaluation of the reactivity of soil, namely the shrink-swell test (AS1289.7.1.1, 2003), the loaded shrinkage test (AS1289.7.1.2, 1998) and the core shrinkage test (AS1289.7.1.3, 1998). The shrink swell test was used for this study because it has two distinct advantages when compared to the other two methods: (a) both swell and shrinkage strains are considered so that the sample may be either very wet or very dry; (b) there is no need to measure soil suction values (Li et al, 2016).

The shrink-swell test, consisting of a core shrinkage test and a swelling test, requires two identical soil samples which have the same initial moisture content (i.e. one for core shrinkage test and another for swelling test).

3.4.1 Test Procedure

The core shrinkage test steps are described as follows:

1. An undisturbed cylindrical core sample of a diameter of 45-50 mm was cut and trimmed to a length within the range of 1.5 to 2 diameters.
2. Initial diameter, length and mass were recorded.
3. Each end of core was placed with a drawing pin that provided a reference mark for measurements to be taken during the drying procedure.
4. The core was allowed to be air dried on a smooth surface for about two weeks. Diameter, length and mass were recorded regularly during this period.
5. After about two week drying in the air, the core was oven dried at a temperature of 105-110 °C for at least 24 hours.
6. After removed out of oven, the length and mass of the core were measured and final moisture content was determined.
The total shrinkage strain $\varepsilon_{sh}$ was calculated by the following equation 3.4:

$$\varepsilon_{sh} = \frac{100 \times (D_0 - D_d)}{H_0}$$  \hspace{1cm} (3.4)

Where

- $D_0$ is the distance between the rounded heads of the pins after their placement (mm)
- $D_d$ is the distance between the rounded heads of the pins after removal of specimen from oven (mm)
- $H_0$ is the initial length of the specimen (mm)
The apparatus used for the swelling test are shown in Figure 3.9

![Apparatus used for the swelling test](image)

**Figure 3.9: Apparatus used for the swelling test**

The swelling test procedure (AS1289.7.1.1, 2003) is as follows:

1. A 50 mm diameter sample was cut with a rigid steel ring of 20mm height and 45mm in diameter and trimmed carefully to ensure both ends were flat. The trimmings were collected and used to determine initial moisture content.

2. The specimen was then placed in a consolidation cell with two porous stone plates at the top and bottom.

3. A seating pressure of 5 kPa was applied for about 10 minutes and the displacement transducer was zeroed under this seating load to allow for a small amount of initial settlement of specimen.
4. The vertical pressure was then increased to a value equal to the overburden pressure or 25 kPa (whichever is greater) for a maximum period of 30 minutes. After recording the initial specimen settlement, which was taken as the datum from which swelling strain was determined, the specimen was inundated with distilled water and allowed to swell.

5. The swelling displacement of the specimen was regularly monitored during the swelling procedure.

6. The testing was terminated when the swelling increment was less than 5% of the total specimen swelling movement for a period of at least 3 hours.

7. Final moisture content and mass of the specimen was measured at the end of swelling test.

\[ \varepsilon_{sw} = \frac{100 \times (D_s - D_i)}{H_0} \]  

(3.5)

Where

\(D_s\) = the total swell of the sample after inundation. (mm);

\(D_i\) = the initial settlement observed prior to inundation (mm);

\(H_0\) = the average initial height of the specimen (mm).
3.4.2 The Results of shrink swell testing

Knowing the shrinkage strain $\varepsilon_{ss}$ and swelling strain $\varepsilon_{sw}$, the shrink-swell index, $I_{ss}$, can be calculated by the following equation.

$$I_{ss} = \frac{\varepsilon_{sw}/2 + \varepsilon_{ss}}{1.8}$$

(3.6)

The calculated shrink-swell index for Glenroy soil is 2.00%/pF while for Braybrook soil $I_{ss} = 4.53%/pF$. 
3.5 Shear Box Tests

In this study, a series of shear box tests was also performed to determine the shear strength of Glenroy soil. The apparatus used for the testing is shown in Figure 3.10. The size of the shear box is 60 mm by 60 mm by 20 mm high.

Figure 3.10: Apparatus used for the direct shear testing

The test procedure is as follows

1. Initial moisture content and dry density were determined for each specimen before the test.

2. The specimen was consolidated in the shear box prior to the shear box testing.

3. Turn on the machine to start the test. During the testing, all readings were taken by a computer, which include the shear stress, horizontal movement and vertical movement. The time taken to reach failure ($t_f$) was calculated from the following equation:

$$t_f = 50t_{50}$$

(3.7)

where $t_{50}$ is the time taken to reach 50% consolidation (minutes)
4. The machine stopped and reserved the direction, when the shear box was reached the limit of travel and the shear box was then bought back to the initial position.

5. The shear stress versus horizontal displacement curve was plotted. The peak shear stress was then identified based this curve.

6. Finally a graph of shear stress versus normal stress can be plotted using Microsoft Excel Spreadsheet.

The relationships between shear stresses and normal stresses for different dry densities of soil are plotted in Figures 3.11- 3.13. The results of shear box testing on Glenroy soil are summarised in Table 9.

Figure 3.11: Shear strength vs normal stress (dry density = 14 kN/m$^3$)
Figure 3.12: Shear strength vs normal stress (dry density = 17.4 kN/m$^3$)

Figure 3.13: Shear strength vs normal stress (dry density = 18 kN/m$^3$)
Table 9: Results of Shear Box Testing

<table>
<thead>
<tr>
<th>Dry density (kN/m$^3$)</th>
<th>$c'$ (kPa)</th>
<th>$\phi'$ (degree)</th>
<th>Shear strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.0</td>
<td>71.51</td>
<td>49.32</td>
<td>$\tau_f = 71.51 + \sigma \tan (49.32^\circ)$</td>
</tr>
<tr>
<td>17.4</td>
<td>117.67</td>
<td>56.06</td>
<td>$\tau_f = 117.67 + \sigma \tan (56.06^\circ)$</td>
</tr>
<tr>
<td>18.0</td>
<td>292.22</td>
<td>76.94</td>
<td>$\tau_f = 292.22 + \sigma \tan (76.94^\circ)$</td>
</tr>
</tbody>
</table>

From Table 9, it can be seen that the effective cohesion ($c'$), effective friction angle ($\phi'$) and shear strength increase with the dry density of soil.
3.6 Soil-Water Characteristic Curve

The soil-water characteristic curve (SWCC) also known as the soil-water retention curve (SWRC) is generally used to present the behavior of deformable unsaturated soil. It provides the connection between the soil suction and the amount of water in the soil such as the degrees of saturation. It also gives the essential characteristics of unsaturated soil. Lots of properties of unsaturated soil such as coefficient of permeability; shear strength and the amount of water contained in the pores in any suction can be obtained via the soil-water characteristic curve (Mualem 1976, Fredlund et al.1994, Wheeler 1996, Assouline 2001).

The air entry value (AEV) is when air begins to go in the greatest pores of the soil. It can be comprehended as the suction required causing desaturation of the greatest pores, specified by Fredlund (1996). It can be obtained from SWCC curve as shown in Figure 3.14 which is an example of the soil-water characteristic curve. It is very significant to know that the desaturation would occur only when the suction is greater than the air entry value.

![Figure 3.14: Example of soil-water characteristic curve (Sillers et al. 2001)]
SWCC normally consist of three stages during drying and wetting process:

- The Saturation zone, also called capillary saturation zone. The pore water is tightness and the soil is seeing as fundamentally saturated due to capillary force in this zone. It continues until the air start to goes in the large pore in the soil sample.
- The Desaturation zone: when soil suction value is exceeding the air entry value, the air will starts to replacing the pore water in the soil, as a result, a significant decrease appears in the degree of saturation. This zone ends when an increase of soil suction does not resulting significant changes in the degree of saturation.
- The Residual zone, it will be completed due to oven dry conditions where the water content equals zero, corresponding to a soil suction of approximately 10^6 kPa (Croney and Coleman 1961).

A number of the empirical SWCC models/equations have been proposed by different researchers, which are discussed briefly in the following sections.

**Gardner Model (1956)**
Gardner proposed an equation which was used for modeling the permeability coefficient of unsaturated soil in 1956. This equation has been used to model the soil-water characteristic curve as well.

\[
S_r = \frac{1}{1 + a\varphi^n} \tag{3.8}
\]

Where \(S_r\) is the degree of saturation, \(\varphi\) is the total suction, \(a\) and \(n\) are the fitting parameters.

**Brooks and Corey Model (1964)**

The very first models used for the soil-water characteristic curve would include the Brooks and Corey equation.
\[
S_r = \begin{cases} 
1 & \varphi < a \\
\left(\frac{\varphi}{a}\right)^{-n} & \varphi > a 
\end{cases}
\]  

The equation specified that SWCC would become an exponentially reducing function when suction over the air entry value and the SWCC remains constant when suction value is less than the air entry value.

**Van Genuchten Model (1980)**

The Van Genuchten model is very popular due to its flexibility and simplicity. Many researchers have modified this model.

\[
S_r = \frac{1}{(1 + (a\varphi)^n)^m} 
\]  

Flexible is a significant advantage of the origin Van Genuchten model, because it consists three parameters and it is continues model where Brooks and Corey (1961) does not create the continuous function for the entire SWCC.

**Fredlund and Xing Model (1994)**

A three parameter continues model for SWCC was created by Fredlund and Xing in 1994:

\[
S_r = \frac{1}{(\ln(e + (a\varphi)^n))^m} 
\]

Where \( S_r \) is the degree of saturation, \( a, n \) and \( m \) are the fitting parameters, \( e \) is void ratio and \( \varphi \) is soil suction.

This model has great flexibility to fit a wide range of data. Each parameter in the equation is significant. The achievement of one parameter in the equation could be distinguished from the effect of the others two parameters, (Sillers and Fredlund, 2001).
3.6.1 Testing Procedure

1. The soil sample was collected from Glenroy field site.

2. A cutting ring with an internal diameter of 50 mm was used to cut/trim the soil sample used for SWCC test.

3. The initial weight and volume of specimen, and the initial soil suction/moisture content were measured. The initial void ratio of specimen was calculated as well.

4. The specimen was submerged by distill water for approximately two weeks until it was fully saturated. A small surcharge pressure was applied on the top of the specimen.

5. After all preparations had been done, sample was placed into the SWCC cell.

6. After a 20 kPa surcharge is applied on the top of the sample, the initial vertical displacement of specimen and initial water discharged were recorded.

7. The water discharged out of the specimen was recorded every day.

8. At the end of test, the specimen was taken out from the sample cell and the final weight and dimensions of the specimen were measured.

3.6.2 Test Result

The SWCC of Glenroy soil in terms of gravimetric water content and degree of saturation are plotted in Figure 3.15 and Figure 3.16 respectively. The suction range above 1500 kPa was obtained by using a Dewpoint Potentiometer (WP4). After SWCC test, the final mass and volume of the specimen were determined. The specimen was placed on a smooth surface and left for air dry for three hours. The length and diameter of the specimen was monitored using a digital vernier caliper. The final suction of the specimen was measured by the use of WP4 potentiometer. It was assumed that fully dry soils (i.e., zero water content or zero degree of saturation) had a suction of around 1,000,000 kPa. This value is supported by experimental evidence and theoretical thermodynamic considerations (Fredlund et al 1994).
Figure 3.15: Gravimetric water content versus soil suction for Glenroy soil
As can be seen from Figure 3.16 at the beginning of applying matric suction, the soil specimen has initial gravimetric water content that is 37.87%. A flat reduction is applied until the suction is increased to 400 kPa, and before 1000 kPa, the moisture content has decreased rapidly. The suction value above 1500 kPa has been obtained using the WP4 potentiometer test. The shape of the SWCC for Glenroy soil shows flat reduction before the suction with 400 kPa, which indicates that expansive soil has a high water retention capability. Composed two plots, the amount of water draining out of the soil during drying from saturated conditions were accompanied by a corresponding reduction in the void ratio of the soil.
3.7 Oedometer Test

As introduced in Section 2.2.2.1, oedometer testing is one mostly used method to measure swelling pressure of expansive soils directly. Constant volume method and the Oedometer test method were studied by Ali and Elturabi (1984). They found that the oedometer test method might give a slightly higher swelling pressure value than that obtained from the constant volume method. The oedometer test method was adopted in this study due to its simplicity. To evaluate the impacts of initial dry density, initial suction/moisture content on the oedometer swelling test, and three groups of oedometer tests were performed in this study, in which the samples were prepared at different initial dry densities and different initial suctions/moisture contents.

3.7.1 Test Procedure

1. After collected from the field site, the soil was firstly dried in an oven at a temperature of about 105°C and was then crushed.
2. The samples were prepared at different initial suctions/moisture contents with different initial dry densities
3. The initial soil suction and moisture content of the specimen were measured;
4. The specimen was inundated with distilled water and then a load was applied on the top of the specimen.
5. Once initial swelling movement decreased, an incremental load was applied to the specimen;
6. After settlement stop rising up, further loads were applied to the specimen;
7. The above procedure was repeated until the specimen was compressed back to initial height;
8. The swelling pressure was taken as the pressure that was required to compress the specimen to its initial height.
3.7.2 Results of Oedometer Tests

3.7.2.1 Swelling pressure versus initial suction

Figure 3.17 shows the measured swelling pressures versus the initial soil suctions. The results show that for both Glenroy and Braybrook soils, the swelling pressure increases with the initial suction of the soil samples. This is expected because soil suction is a measure of a soil’s affinity for moisture. As the initial suction increases, for the samples having the same dry density, the initial degree of saturation will decrease and the affinity of soil to absorb water will increase. Although there is considerable scatter of data for Glenroy soil, a reasonably good correlation can be observed between the initial soil suctions and the measured swelling pressures with $R^2$ of the trend line exceeded 0.8.
3.7.2.2 Swelling pressure versus moisture content

The measured swelling pressure is plotted against the initial moisture contents of soil samples in Figure 3.18. The swelling pressure of both Glenroy and Braybrook soils decreases as the initial moisture content increases. This can be attributed to the fact that at a high water content, the soil is already expanded and has less swelling potential (Erzin and Erol 2004). From Figure 3.18, it can be seen that the slope of swelling pressure vs initial moisture content curve of Braybrook soils is much larger than that of Glenroy soil. This indicates that the impact of the initial moisture content on the swelling pressure is more significant for highly reactive soil than for low or medium reactive soil.

![Swelling pressure versus initial moisture content](image)

**Figure 3.18: Swelling pressure versus initial moisture content**
3.7.2.3 Swelling pressure versus dry density

In this group of oedometer tests, the only one variable parameter is the dry density of expansive soil. Figure 3.19 shows the relationship between the measured swelling pressure and initial dry density. As expected, the swelling pressure increases with the initial dry density of soil.

![Figure 3.19: Swelling pressure versus dry density](image)

To determine if a correlation exists, the values of the swelling pressure were plotted against initial dry density in Figure 3.19 using Excel spreadsheet. The equation of the fitted trend line and $R^2$ values are:

For Glenroy soil  
$$P_s = 159.31 \gamma_d - 2227.3, \quad R^2 = 0.9754$$

For Braybrook soil  
$$P_s = 161.48 \gamma_d - 2048.6, \quad R^2 = 0.8674$$

Where $P_s$ is the swelling pressure and $\gamma_d$ is the soil dry density.
For a correlation to be considered as a reliable mean for estimating the swelling pressure of a soil, $R^2$ of the trend line needs to exceed 0.8. It is apparent that there is a very good correlation between $P_s$ and $\gamma_d$ since the strength of the correlation is 97.54% (i.e. $R^2 = 0.9754$) for Glenroy soil and 86.74% (i.e. $R^2 = 0.8674$) for Braybrook soil. The laboratory results indicate the dry density is the most important parameter for estimating the swelling pressure of a soil. Ali and Elturabi (1984) also reported that dry density gave a more significant indication of swelling pressure than other parameters such as plastic index. From Figure 3.19, it can be seen that the higher the shrink-swell index (i.e. Braybrook soil), the larger the predicted swelling pressure.

### 3.8 Summary

This chapter presents the results of laboratory tests which include the oedometer tests, liquid limit test, plastic limit test, linear shrinkage test, X-ray diffraction (XRD) test, shrink-swell test, shear box test and soil-water characteristic curve. Based on the shrink-swell index, liquid limit and plastic limit, Glenroy soil can be classified as a medium reactive soil while Braybrook soil can be classified as a highly reactive soil. This is confirmed by X-ray diffraction (XRD) test which indicates that Braybrook soil has a larger swelling potential than Glenroy soil. The results of the shear box tests indicate that the shear strength and shear parameters ($c'$ and $\varphi'$) increase with the dry density of the soil. The results of oedometer tests show that the swelling pressure increases with the initial dry density and initial soil suction but decrease with the soil moisture content. There is a reasonably good correlation between the swelling pressure and initial soil suction/moisture content while the correction between the swelling pressure and the soil dry density is more reliable with the strength of this correlation ranging from 86.74% to 97.54%.

The results of laboratory tests presented in this Chapter provide a good understanding of the engineering properties of Glenroy soil that was used for the retaining wall experiments in this study. The retaining wall experiments will be presented in the following chapter.
Chapter 4
Retaining Wall Experiments

4.1 Introduction

Retaining wall has been widely used in civil engineering to support lateral loads from soils. Currently in Australia, it is generally assumed that the backfill behind a retaining wall is non-expansive material. However, a retaining wall constructed on expansive soil may be subjected to significant lateral swelling pressure as a result of soil swelling due to change in soil moisture. A literature review shows that little research has been carried out to predict or estimate the horizontal swelling pressure. No direct reference can be found in the geotechnical literature in Australia and retaining wall model experiments are almost non-existent.

In order to get better understanding of the lateral swelling pressure developed behind of a retaining wall, a model retaining wall was built at RMIT Geotechnical Laboratory and two retaining wall experiments were conducted in this study.

In this chapter, the experimental set-up and procedure are first described, and the testing results are then presented and discussed.
4.2 Soil samples and soil suction measurement

The soil used for laboratory test and retaining wall experiments was obtained from an expansive soil filed site in Glenroy East, approximately 13 km northern of Melbourne CBD (Li et al, 2014). After removing vegetation and topsoil, the soil was collected from a depth of approximately 0.4-0.5 m. After returning to the laboratory, the soil was firstly dried in an oven at a temperature of about 105°C and then crushed.

The smashed dry soil was mixed at different dry densities and moisture contents, and used for the oedometer tests to determine the swelling pressure and the retaining wall model experiments. It was also used for shrink swell test to assess the reactivity of soil and for shear box tests to obtain shear the strength of soil. All of the laboratory tests were conducted according to Australian Standards (see Chapter 3).

The initial and final soil suctions were measured using WP4 Potentiometer and the filter paper method. The Wescor’s in situ soil psychrometers were used to monitor soil suction changes during retaining wall experiments. Before the lab experiments the soil psychrometers were calibrated using sodium chloride solutions (Figure 4.1)

![Image of WP4 Potentiometer and psychrometers calibration](image)

(a) HR-33T Dew Point Microvoltmeter  
(b) Psychrometer calibration in salt solution

**Figure 4.1: Psychrometer calibration with Sodium Chloride solutions**
4.3 Experimental Setup

A model retaining wall apparatus was constructed at RMIT Geotechnical Laboratory to study the swelling pressure developed behind a retaining wall. Two retaining wall experiments were conducted in this study. The initial dry density of soil was 17.43 kN/m$^3$ and 14.13 kN/m$^3$ respectively. The dimensions of the model retaining wall are 1 m $\times$ 1.4 m $\times$ 1 m. Level lines are drawn at 100 mm intervals. The 2 mm thick steel retaining wall is fixed close to one end of the box, which is structurally supported to avoid bending of the steel. The resulting size of the void is 1000 $\times$ 450 mm in surface area and 1000 mm deep. A silicon seal was used to prevent any water from leaking during the test. Five small holes were drilled to fix load cells to the inner side of the retaining wall. Figure 4.2 shows the arrangement of retaining wall and the locations of load cells. Five load cells were used to measure lateral loads at different heights:

- Load Cell 1 (H = 100 mm)
- Load Cell 2 and 3 (H = 300 mm)
- Load Cell 4 (H = 500 mm)
- Load Cell 5 (H = 700 mm)

![Figure 4.2: Retaining wall arrangement (showing location of load cells)](image-url)
A 50 mm sand layer was placed at the bottom of the void and covered with a layer of geofabric (960 × 410 mm) in order to prevent mixing of sand and soil. Four timber bars were placed in the spaces left by the geofabric to further avoid mixing of the different layers. A 20 mm diameter pipe, 750 mm long was placed into the sand layer running to the top of the box in order to infiltrate the sand and soil with water. Expansive soil was then compacted in 50 mm layers. After the first layer was compacted the timber bars were removed and replaced with Bentonite. As more layers were placed, the load cells were inserted. Sand was placed around the load cells in order to protect them. Figure 4.3 shows the photos of the set-up.

![Figure 4.3: Retaining wall set-up – bottom layer](image)

The filter papers were placed at every two layers (i.e. every 100 mm) from bottom 150 mm. Layers at 150 mm, 250 mm, 350 mm, 450 mm, 550 mm and 650 mm each contain four pieces of filter paper (24 papers). Larger diameter papers of 70 mm were placed at
top and bottom to protect the 47 mm diameter filter paper. The locations of filter papers are shown in Figure 4.4.

![Diagram of retaining wall arrangement](image)

**Figure 4.4: Retaining wall arrangement (showing location of filter papers)**

Wescor’s *in situ* soil phychrometer were placed at heights of 200 mm, 450 mm and 650 mm. Another layer of geofabric and timber bars were placed on top of the soil at 750 mm. A final 50 mm sand layer is placed on top of the geofabric. The timber was removed and again replaced with bentonite. The setup procedure is illustrated in Figure 4.5.

A concrete block was prepared and placed on the top of the soil to apply a 5 kPa surcharge. The block had three pipes inserted in it to allow water to readily seep into the soil. A sixth load cell was placed on top of the concrete block and clamped to the box with the assistance of a steel beam. This was to restrict the vertical swelling pressure from the expansive soil, causing all pressure to be lateral pressure. This load cell was used to measure the vertical swelling pressures. A laser level was placed on top of the steel beam to measure any vertical movement of the concrete block. Two LVDT devices placed at the rear of the retaining wall were used to monitor lateral movement of the wall.
Figure 4.5: Retaining wall set up – second layer

(a) Placed filter paper
(b) The last soil layer
(c) The last geofabric layer
(d) The last sand layer

Figure 4.6: Retaining wall set up – top concrete block, laser and LVDT devices

(a) Concrete block
(b) Concrete block put on top of the sand layer
(c) Top load cell set up
(d) Load cell
(e) Laser device
(f) LVDT device
A data logger was used to take readings from the six load cells, the laser level and the two LVDT’s every minute. After initial data was taken, 16 liter of water was inserted through the pipes to infiltrate from top and bottom.

The retaining wall experiment was run for 80 days and results from the data logger are plotted using Microsoft Excel. The set up schedule of retaining wall experiment 2 was the same as the first experiment, but there were some differences with the first experiment. The density of soil was less than that of the first experiment. Only five load cells were used in the second experiment. There was not a tube at the right hand side of steel box. Thus the filled water can only seep into soil from the top surface. Figure 4.7 shows the photos of model retaining wall which had been set up and was ready for the testing.

Figure 4.7: Model retaining wall set up for experiments
4.4 Result of Retaining Wall Experiments

Two retaining wall experiments with the same initial moisture content but different initial dry densities were carried out in this study. The average initial dry density of Experiment 1 and 2 was 17.43 kN/m$^3$ and 14.13 kN/m$^3$ respectively. The measured results are presented in the following sections, which include lateral and vertical swelling pressures, horizontal displacement of retaining wall and suction changes.

4.4.1 The Load Cell Data

*Retaining Wall Experiment 1*

Figure 4.8 shows the measured total horizontal pressure acting on the wall. As discussed in Section 4.3, load cell 1 was placed at the center of the retaining wall and 150 mm from bottom of the wall. For Experiment 1, the water was filled not only from the top surface but also from bottom side. As can be seen from Figure 4.8, the lateral pressure measured by load cell 1 increased rapidly at beginning. The pressure reduced slightly after 8 days and then increased again after 30 days, finally stayed around 40 kPa.

The load cells 2 and 3 were installed at the same height (see Figure 4.2), 200 mm higher than load cell 1. Load cell 3 was 200 mm left hand side of load cell 2, closer to edge of the wall. Because of a little gap between the soil and steel wall, water leaked into the soil from the side of the wall, leading to the lateral pressure measured by load cell 3 which was closer to edge of the wall was much higher than that measured by load cells 1 and 2. From Figure 4.8, it can be seen that lateral swelling pressure recorded by load cell 3 increased steadily during the first 20 days and then decreased due the fully saturation of the surrounding soils.
Figure 4.8: Measured lateral swelling pressure versus time for load cell 1-3
Figure 4.9 shows the data collected from load cell 4 and load cell 5. Load cell 4 was located in the middle soil layer (Figure 4.2) which was saturated during the last stage of Experiment 1 since water was added to soil from the top and bottom. From Figure 4.9, it can be seen that the measured lateral pressure increased sharply after 60 days due to water seeping into the middle soil layer. The lateral pressure of the top soil layer (measured by load cell 5) increased steadily from 0 to 80 kPa during the first four days and then dropped slowly to 20 kPa. It started to increase again after 32 days and reached to 250 kPa at the end of experiments (Day 81).

Vertical swelling pressure versus time is plotted in Figure 4.10. The vertical swelling pressure measured by the top load cell increased significantly during the first six days with a steep slope and then continued to increase until it reached to 500 kPa at day 81.
Figure 4.9: Measured lateral swelling pressure versus time for load cell 4 and 5
Figure 4.10: Measured vertical swelling pressure versus time
Retaining Wall Experiment 2

During retaining wall Experiment 2, water was filled into the soil from the top only. Figure 4.11 shows the lateral pressure measured by load cells 1-3. The measured lateral pressure varied with time. During experiment, water seeped into soil through the gap between soil and the steel wall, leading the soil at the bottom and edge to swell first. This is why the swelling pressure measured by load cell 1 (at the bottom) and load cell 3 (close to the edge) increased remarkably during the first two days. The soil surrounding load cell 3 became fully saturated at day 28 and the swelling pressure started to fall down after 29 days. The swelling pressured measured by load cell 2 increased slowly and reached to 28 kPa after 82 days. Obviously the soil at the middle of the retailing wall was not fully saturated at the end of Experiment 2. This is mainly attributed to the fact that water was only allowed to infiltrate from the top of soil. After the experiment, the soil samples were taken for soil suction and water content measurements at various locations. The results show that the soil surrounding load cell 2 was much drier than that of other locations.

It should be pointed out that only four load cells were used to monitor the lateral pressure behind of the retaining wall in Experiment 2. As can be seen from Figure 4.12, the lateral swelling pressure of the top soil (measured by load cell 4) increased remarkably during the first two days after water was added to soil through three pipes inserted into the top concrete block. The measured pressure decreased after two day due to that water infiltrated into the bottom soil layers and increased again at Day 6 when water was added to the top surface. More water was added at Day 17 and 21, causing the lateral swelling pressure to reach a peak value of 570 kPa at Day 24.

From Figure 4.13, it can be seen that the vertical swelling pressure increased steadily with time and reached to a maximum value of 290 kPa at Day 40, which was much lower than the maximum value of 500 kPa in Experiment 1. This is because that the initial dry density of the soil used in Experiment 2 was much lower than that used in Experiment 1.
Figure 4.11: Measured lateral swelling pressure versus time for load cells 1, 2 and 3
Figure 4.12: Measured swelling pressure versus time for load cell 4
Figure 4.13: Measured vertical swelling pressure versus time
4.4.2 The Displacement Data

The displacement measurement was used to prove that the wall was not bending. So the data measured from load cells was the exactly force that could be used.

Retaining Wall Experiment 1

During the model retaining wall experiments, the lateral deformation of the steel wall was monitored by the use of two LVDT (Linear Variable Differential Transformer) devices. LVDT 1 was installed at the centre of the retaining wall while LVDT 2 was located 200 mm below LVDT 1. To restrict the vertical swelling displacement, the top concrete block was clamped to the steel box with the assistance of a steel beam. The vertical movement of the concrete block was monitored using a laser device placed on the top of the steel beam.

Figure 4.14 shows the displacement-time graph for Experiment 1. The measured displacement generally increased with time. The maximum lateral and vertical displacements was approximately 0.8 mm and 2 cm respectively.

The displacement versus time for Experiment 2 is plotted in Figure 4.15. While there is a generally increasing trend in displacements with time, the measured lateral displacements may be ignored since they are very small. The maximum lateral displacement was 1.3 mm, slightly larger than that of Experiment 1 while the maximum vertical displacement of 1.3 mm was lower than that measured in Experiment 1.
Figure 4.14: Displacements of retaining wall versus time (Experiment 1)
Figure 4.15: Displacements of retaining wall versus time (Experiment 2)
4.4.3 Suction Measurement

A retaining wall backfilled with expansive soils may be subjected to a lateral swelling pressure due to soil suction and volume change. The lateral swelling pressure is dependent upon soil suction change of the soil media. Therefore the soil suction measurement is an important part of laboratory experiments.

Soil suction has been introduced in details in Section 2.3, which can be measured directly or indirectly. Several methods/instruments are valuable for the measurement of soil suction, which include psychrometers, filter papers, pressure plates, pressure membranes, tensiometers, and thermal conductivity sensors. Each method has its own limitations. Tensiometers only work in the low suction range and require frequent maintenance. Psychrometers are less sensitive in the low suction ranges but sensitive with the temperature of surrounding environment, and must be calibrated prior to testing. Pressure membranes, pressure plates and thermal conductivity sensors can only measure the matric suction. The filter paper method is based on the assumption that a filter paper will come to equilibrium with soil suction.

In the laboratory, the filter paper method is treated as a reliable test method for measure of the matric and total suctions of soil sample. Thermal conductivity sensors, membranes, and pressure plates could only measure the matric suction (Manosuthkij et al. 2008). Psychrometers need recalibration and maintenance frequently, less sensitive in low suction range, but sensitive with the temperature of surrounding environment, and could only measure the total suction. Tensiometers work in low suction range and daily maintenance is required. The filter paper method of suction measurement is based on the assumption that a filter paper will come to equilibrium with soil suction. When a dry filter paper is placed in direct contact with a soil specimen a closed space, there is direct liquid connectivity between the paper and the soil. It is assumed that equilibrium is reached by liquid moisture exchange between the soil and the filter paper. Hence, it is considered that contact filter paper method gives values of matric suction. When a dry filter paper is suspended above a soil specimen in a closed container, equilibrium is
reached by vapor moisture exchange between the soil and the filter paper. The concentration of this moisture vapour is controlled by the total water potential of the soil (i.e., total suction).

\[ \text{4.4.3.1 Soil Suction Measured Using Filter Paper Method} \]

The filter paper method was adopted in this study because it is an inexpensive and relatively simple laboratory test method.

At the end of experiments, the moisture content of the filter papers was determined, and the suction values of the soil were then estimated by using the calibration curve recommended by ASTM D5298 (2010).

Matric suction

\[ \log \phi = 2.909 - 0.0229w_f, \ (w_f \geq 47\%) \]  
\[ \log \phi = 4.945 - 0.0673w_f, \ (w_f < 47\%) \]  

Total suction

\[ \log \phi = 8.778 - 0.222w_f, \ (w \geq 26) \]  
\[ \log \phi = 5.31 - 0.0879w_f, \ (w < 26) \]  

\[ \text{Retaining Wall Experiment 1} \]

The soil suction profiles at the end of Experiment (i.e., after 80 days) are shown in Figure 4.16. The horizontal locations of filter papers relative to the retaining wall are also illustrated in Figure 4.16 while the vertical locations of filter papers are shown in Figure 4.4. The soil suction measured by the filter paper method in this study was matric suction of the soil, because the filter papers were buried within the soil without gaps between the soil and papers. As discussed previously, in Experiment 1, water was
filled into the soil from both top and bottom. Thus the soil at the top and bottom would reach a fully saturated state quickly.

From Figure 4.16, it can be seen that the shapes of the soil suction profiles are similar, i.e. the suction values at the top and bottom were approached to zero while the soil at the middle had a much higher suction ranging from 2000 kPa to 3300 kPa. In other words, the retaining wall experiment should have been run longer than 80 days to allow the maximum lateral swelling pressure to develop behind the retaining wall. However as the RMIT Civil Engineering Laboratory had to be moved from the City Campus to Bundoora campus, the retaining wall experiment must be completed within three months.

Figure 4.16: Matric suction measured using the contact filter paper method (Experiment 1)
It should be noted the soil suction shown in Figure 4.16 is matric suction of the soil since the contact filter paper method (i.e., the filter papers and soil were in contact) was used in the lab experiment. In most cases, the water content of the soils can generally be assumed to correspond to matric suction or total suction when the suctions are greater than 1500 kPa, i.e., the low suction range up to 1500 kPa represents matric suction and the high suction range beyond 1500 kPa represents total suction (Li et al 2007).
In Experiment 2, the water was added to the soil from the top surface only. As shown in Figure 4.17, the top soil was fully saturated with a very low suction value. The soil suction increased with the depth. It can be seen that the suction value of the bottom soil at the right hand side of the retaining wall (the red line) was 322 kPa, much lower than the suction value of 1740 kPa obtained at the left hand side of the wall (the blue line). A close inspection revealed that there was a little gap between the wall and soil along the right hand side corner which allowed water to leak into the soil.

Figure 4.17: Matric suction measured using the contact filter paper method (Experiment 2)
4.4.3.2 Soil SuctionMeasured Using WP4 Dewpoint Potentiometer

Retaining Wall Experiment 1

After the experiments, the soil samples were collected from the center of soil behind the retaining wall at the different depths. The soil suctions were measured using a Decagon WP4-T Dewpoint Potentiometer which uses the chilled-mirror dewpoint technique to measure total suction. The WP4 was calibrated using a 0.5 Molal/kg solution of potassium chloride as per the guidelines provided by the manufacturer. The measured final suction values are plotted in Figure 4.18. It can be seen that the pattern of soil suction distribution is similar to those plotted in Figure 4.16. The total suction of the top and bottom of soil was lower because water was filled from both top and bottom of the soil in Experiment 1. At the end of the experiment (i.e. after 80 days), the water had not reached to the middle part of the soil.

![Figure 4.18: Total suction of the soil behind the center of the wall measured using WP4 (Experiment 1)](image-url)
Retaining Wall Experiment 2

Figure 4.19 shows the total suction of the soil behind the center of the retailing wall measured by the use of WP4 at the end of Experiment 2. As discussed previously, in Experiment 2, the water was filled into the soil from top surface only. Therefore the soil suction at the top 300 mm was remarkably lower than that of the bottom soil.

![Graph showing total suction of soil behind the center of the wall](image)

Figure 4.19: Total suction of the soil behind the center of the wall measured using WP4 Experiment 2)
4.4.3.3 Soil Suction Measured Using Wescor in situ soil Psychrometer

In the retaining wall experiments, Wescor's *in situ* soil psychrometers (model PST-55) were used to monitor the change in soil suction with time during the laboratory experiments. Three psychrometers were buried within the soil at the different depths behind the retaining wall. The PST-55 has a stainless steel screen to allow only the water vapor to enter the sensor.

*Retaining Wall Experiment 1*

The location of three psychrometers and the measure soil suctions are shown in Figure 4.20 and Figure 4.21 respectively.

![Figure 4.20: Location of Psychrometer sensors embedded behind of retaining wall (Experiment 1)](image-url)
From Figure 4.21, it can be seen that the measured soil suction decreased with time. The suction value of the bottom soil (red line) and top soil (black line) dropped much quicker than that of the middle soil (blue line) since water was filled into the soil from both top and bottom sides. The total soil suctions after 80 days measured by the use of Wescor’s psychrometers (Figure 4.21) agree reasonably well with those obtained by Decagon WP4-T Dewpoint Potentiometer Figure 4.18.

Retaining Wall Experiment 2

In Experiment 2, three PST-55 psychrometers were installed behind the center line of the retaining wall as shown in Figure 4.22. The total soil suctions are plotted versus time in Figure 4.23. It can be seen that the suction of the top soil (black line) dropped remarkably during the first week and was maintained at approximately 270 kPa after 20 days. This is because that water was added into the soil from top surface only. The
suction of the middle soil (400 mm from the bottom, blue line) decreased gradually with time as water slowly seeped into the middle soil layer. The suction state of the soil at the bottom of retaining wall remained almost unchanged.

Figure 4.22: Location of psychrometer sensors embedded behind of retaining wall (Experiment 2)
Figure 4.23: Soil suction measured using psychrometers (Experiment 2)

4.5 Summary

In this study, a model retaining wall was built, and two retaining wall experiments were carried out to study the horizontal swelling pressure developed behind the retaining wall due to change in soil moisture. In retaining wall Experiment 1, water was allowed to seep into the soil from both top and bottom while in Experiment 2, the soil was wetted from the top surface only. During the experiments, The Wescor's *in situ* soil psychrometers were used to monitor the suction variation of the soil behind the retaining wall. The final suctions of the soil were measured using the contact filter paper method and Decagon WP4-T Dewpoint Potentiometer.

The results show that the swelling pressure increased with a decrease in soil suction and an increase in the soil density. The soil suction profiles obtained from WP4 Potentiometer are found to compare very well with those obtained by the use of the filter paper method. The soil suctions measured by Wescor's psychrometers agree
reasonably well with those measured by using WP4 Potentiometer and the filter paper method.

Due to the lab relocation, the retaining wall experiments had to be completed after 80 days. The measured final soil suction values clearly show that 80 days are not long enough for the soil behind the retaining wall to reach a full saturation state.
Chapter 5

Development of New Model for Prediction of Lateral Swelling Pressure

5.1 Introduction

Based on the laboratory tests, an empirical equation is proposed for predicting the lateral swelling pressure of a retaining wall. Three parameters, namely the initial dry density, initial moisture content and soil shrink-swell index are taken into account. The constants adopted in the proposed empirical equation are obtained through multiple regression analysis (MRA). Although a few empirical equations for the prediction of horizontal swelling pressure are available in literature internationally, no research has been done in Australia. Therefore there is a need to develop a new method/model for local engineers so the lateral swelling pressure can be estimated based on the soil indices widely used in the routine practice in Australia.

There are several empirical equations available for prediction of swelling pressure. Erzin and Erol (2007) carried out multiple regression analysis based on the results of experiments on statically compacted samples of Bentonite-Kaolinite clay mixtures. They found that following equations have strong correlations between the swelling pressure and the soil properties.

For $0 < P_s \leq 100$ kPa;

$$P_s = -3.72 + 0.0111PI + 2.077\gamma_d + 0.244 \log s \quad R=0.96$$

(5.1)

For $100 < P_s \leq 350$ kPa;

$$P_s = -16.31 + 0.0330PI + 8.253\gamma_d + 0.829 \log s \quad R=0.97$$

(5.2)

Where

$P_s = Swelling\ pressure$ ($kg/cm^2$);

$PI = Plastic\ index$ ($\%$);

$\gamma_d = Dry\ density$ ($g/cm^3$);

$s = soil\ suction$ (bar).
Elisha (2012) performed one-dimensional swell tests on soil samples specimens with variable properties in oedometers. The relationship between swelling pressure and plasticity index (PI), water content (w) and dry density ($\gamma_d$) were examined. The multiple regression analysis was used to develop the following equation to estimate the logarithm of the swelling pressure:

$$\log P_s = -5.423 + 0.01458PI + 2.563\gamma_d - 0.0168w \quad R^2 = 0.95$$

(5.3)

5.2 The proposed new model

In this study, a new model/equation relating swelling pressure, in terms of shrink-swell index, initial dry density and initial moisture content, was proposed:

$$\log P_s = a + b I_{ss} + c \gamma_d + d w_i$$

(5.4)

Where

$P_s = Swelling$ pressure $($kPa$);$  
$w_i = initial$ moisture content ($\%$);  
$\gamma_d = Dry$ density ($kN/m^3$);  
$I_{ss} = Shrinkage$ and$ swell index ($\%/pF$);  
$a, b, c, d$ are constants;

The multiple regression analysis was carried out using Metlab to obtain the fitting parameters a, b, c and d (see Figure 5.1 and Figure 5.2):

$$\log P_s = -0.2849 + 0.0686 I_{ss} + 0.1851 \gamma_d - 0.0318w_i \quad R^2 = 0.94$$

(5.5)

The result shows that the logarithm swelling pressure has strong correlations with soil properties that consist of shrink swell index, initial dry density and initial moisture content as the strength of this correlation is 94% (i.e. $R^2 = 0.94$).
Figure 5.1: Multiple regression analysis by Metlab
Figure 5.2: Regression equation obtained using Metlab
Figure 5.3 shows the effect of initial dry density on the swelling pressure for the proposed method. The swelling pressure increases with increasing initial dry density for the soils having the same initial moisture content and shrink-swell index. An increase in soil dry density leads to smaller volume for water particles to move. During water absorption, water particles can exert greater force to the surrounding soil particles to achieve the complete saturation. Consequently the soil swelling pressure will be higher as results of these increased inter-particle forces.

![Graph showing the effect of initial dry density on the predicted swelling pressure](image)

**Figure 5.3: Effect of initial dry density on the predicted swelling pressure**
Figure 5.4 shows the relationship between the swelling pressure predicted by the proposed equation and the initial moisture content for Glenroy soil having the same initial dry density. The swelling pressure decreases as the initial moisture content increases.

![Graph showing the relationship between swelling pressure and initial moisture content.](image)

**Figure 5.4: Effect of initial moisture content on the predicted swelling pressure**

Figure 5.5 shows the effect of shrink-swell index ($I_{ss}$) on the swelling pressure for the proposed method. It can be seen that for the soils having the same initial dry density and initial moisture content, the predicted swelling pressure increases with the shrink-swell index.
5.3 Summary

A new model/equation has been proposed for prediction of the lateral swelling pressure using the initial dry density, initial water content and shrink and swell index. The main advantage of this new model is its simplicity as all parameters can be obtained by conventional laboratory geotechnical tests.
Chapter 6  
Conclusions and Recommendations

6.1 Conclusions

In this study, a model retaining wall was built, and two retaining wall experiments were carried out to evaluate the lateral swelling pressure developed behind the retaining wall due to change in soil moisture content (suction). Fifteen groups of oedometer swelling tests were performed on samples with different initial moisture contents/soil suction values and different initial dry densities. In addition to retaining wall model tests and constant volume swell tests, a series of other laboratory tests were also performed, including liquid limit test, plastic limit test, linear shrinkage test, X-ray diffraction (XRD) test, soil-water characteristic curve, shear box test, shrink-swell test, and soil suction measurement.

The following conclusions can be drawn from the testing results:

- Swelling pressure developed within soil depends on the initial moisture content of the soil mass. Swelling pressure decreases with an increase of initial moisture content for the soil having the same initial dry density.
- Both the model retaining wall experiments and constant volume swell tests show that swelling pressure increases with the initial dry density for the soil having the same initial moisture content.
- Swelling pressure increases with the initial soil suction for the soil having the same initial dry density.
- Swelling pressure increases as the shrink-swell index $I_{ss}$ increases.

Based on the laboratory tests, an empirical equation has been developed for estimating the lateral swelling pressure of unsaturated expansive soils behind of retaining walls. This equation can be used for prediction of horizontal swelling pressure based on initial moisture content, initial dry density and the shrink-swell index which is widely used in Australia for classification of expansive soil sites.
6.2 Recommendations for Further Work

It is recommended that further research be carried out in the following areas:

1. Due to laboratory relocation, only two retaining wall experiments were carried out and each experiment was run for 80 days. The results clearly indicate that at the end of the experiments, the soil behind the retaining wall had not been fully saturated yet. More retaining wall experiments are required to further evaluate the lateral swelling pressure developed behind the wall as a result of a change in soil moisture content (suction). The experiments should be allowed to run more than six months if possible.

2. Due to time limitation, only fifteen groups of oedometer swelling tests were performed on two types of soil. More tests are required for different soils with various initial dry densities, various initial moisture contents (suction values) and different shrink-swell indices.

3. Further study of the effect of initial dry density, initial moisture content (suction) and shrink-swell index on the swelling pressure is also needed.
References


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