STRESS EFFECTS ON SOIL-WATER CHARACTERISTICS OF UNSATURATED EXPANSIVE SOILS

by

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ABSTRACT

Soil-water characteristic curve (SWCC) is a key property for unsaturated soils. It describes the water retention capacity of unsaturated soils. SWCC can be used to predict other useful soil properties such as water permeability and is an essential property for both numerical and analytical transient seepage and slope stability analyses.

In the thesis, a newly modified triaxial apparatus has been used to investigate the influence of isotropic (ISO) stress conditions on measured SWCCs of some unsaturated expansive soils from China. A one-dimensional (1D) stress controllable volumetric pressure plate extractor and pressure plate/membrane extractors without applying any external stress are also used in the study. The SWCCs of the expansive soils are found to be stress dependent. The difference between SWCCs tested under zero stress and non-zero stresses is significant. Generally, the expansive soil has a lower water retention ability under stress compared with the results from conventional SWCC tests without external stress. However, the difference between 1D and ISO stresses is small except for the part in the matric suction range of 1-10 kPa. There is no difference in soil-water characteristics under stress of 50- and 100-kPa when the suction is higher than 10 kPa. The test results of Scanning Electron Microscopy (SEM) and Mercury Intrusion Porosimetry (MIP) show that the recompacted ZY expansive soil has a bi-modal distribution of pore sizes. The pore spaces are mostly composed by inter-aggregate and intra-aggregate pores. Because inter-aggregate pores can be compressed by external stresses, the SWCCs exhibit clear difference under the low suction range from 1 to 10 kPa. Intra-aggregate pores are left unchanged, therefore, the soil-water characteristics are more or less the same in high suction range of 10-500 kPa, regardless of different stress conditions.

A parametric study has been carried out to investigate the influence of rainfall patterns, amount and duration on groundwater responses in an unsaturated slope of residual soils. As for 24-hr short rainfalls, the advanced rainfall pattern seems to be the most critical rainfall type for the slope stability because it can always induce the highest pore-water pressure distributions along the slope. The higher the rainfall depth is, the higher the pore-water pressure buildup will be. However, in this study, it is found that a further increase in the return period of a rainfall of 100-yr, the pore-water pressure does not necessarily increase. Rainfall patterns are more relevant for short rainfalls, since the rainwater distributes more uniformly for storms with long duration.
NOTATION

\[ \tau_f = \text{shear strength at failure} \]
\[ \tau = \text{shear stress} \]
\[ c' = \text{effective cohesion} \]
\[ \sigma = \text{net normal stress state on the failure plane at failure} \]
\[ u_a = \text{pore-air pressure} \]
\[ u_w = \text{pore-water pressure} \]
\[ \phi' = \text{angle of internal friction associated with the net normal stress state variable, } (\sigma_f - u_a)_f \]
\[ \phi^b = \text{friction angle associated with the matric suction stress state variable, } (u_a - u_w)_f \]
\[ u_a - u_w = \text{matric suction} \]
\[ \psi = \text{total suction} \]
\[ \pi = \text{osmotic suction} \]
\[ h = \text{hydraulic head} \]
\[ k = \text{coefficient of proportionality known as hydraulic conductivity or coefficient of permeability} \]
\[ A = \text{cross-section area} \]
\[ L = \text{length of column (m) where the total hydraulic head loss occurred for } \Delta h \]
\[ Q = \text{total flow rate} \]
\[ m = \text{coefficient of water volume change with respect to a change in ma-} \]
tric suction

\[ \rho = \text{density} \]

\[ g = \text{gravitational acceleration} \]

\[ t = \text{time} \]

\[ F = \text{storage coefficient} \]

\[ K = \text{hydraulic conductivity tensor} \]

\[ z = \text{potential head (elevation head)} \]

\[ \theta = \text{volumetric water content} \]

\[ \alpha' = \text{modified compressibility of the medium} \]

\[ \beta' = \text{modified compressibility of the water} \]

\[ n = \text{porosity of the medium} \]

\[ S = \text{saturation} \]

\[ r = \text{pore radius} \]

\[ d = \text{pore diameter} \]

\[ T = \text{surface tension} \]

\[ \theta = \text{contact angle} \]

\[ w = \text{gravimetric water content} \]
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CHAPTER 1  INTRODUCTION

1.1 BACKGROUND

Soil-water characteristic curve (SWCC) is a key property in unsaturated soil mechanics, playing a role similar to the consolidation curve for a saturated soil (Rahardjo and Leong, 1997; Barbour, 1998). It is defined as the relationship of water content vs. matric suction. The water content could be gravimetric water content, volumetric water content, or degree of saturation. Since direct measurements of unsaturated soil properties are not only time consuming but also very costly, SWCC along with saturated properties is often used to predict water permeability and shear strength (Fredlund et al., 1994; Fredlund et al., 1995).

SWCC, an important hydraulic property indicating the water retention ability, is an indispensable input for both analytical and numerical analysis of transient seepage and slope stability. In both urban and rural areas, natural and engineered slopes can be found everywhere. Canal and river levees, and highway embankments can also be treated as slopes. Any disturbance to its stability could cause slope failure.

People look into slope instability problems from different angles. From the geologist point of view, a landslide, defined as ‘a movement of a mass of rock, earth or debris down a slope’, can be recognized and identified mainly by its geomorphological and geological characteristics, and type of materials (Dikau et al., 1996). However, internal stresses and strains in the soil mass of a slope are of the most interest in the geotechnical engineering field.
Problems involved in slope stability may vary from place to place. For example, in an area with a high risk of earthquake, seismic effects on the slope instability may be the most important, whereas in a tropical or subtropical area, such as Hong Kong, rainfall-induced landslides are the most commonly seen natural disasters in wet seasons.

Conventionally, once the slope geometry and subsoil conditions have been determined, the stability of a slope may be assessed using either published chart solutions or a computer analysis. Most computer programs used for slope stability analyses are based on the limiting equilibrium approach for a two-dimensional model. Some may also allow three-dimensional analysis. Finite element or boundary element methods are also used (Abramson et al., 1996).

In other aspects of geotechnical engineering practices, saturated soil mechanics has been adopted as the theoretical base for slope stability analysis for a long period. Many difficulties are encountered when dealing with unsaturated soils with classical soil mechanics. Numerous efforts have been made for the advancement of knowledge and techniques of soil mechanics for unsaturated soils. The significance of this new branch of soil mechanics has been recognized.

Unsaturated soil mechanics is more relevant in arid and semi-arid areas. In these areas, the annual evaporation from the ground exceeds the annual precipitation, therefore the groundwater table is usually deep below the ground surface and there is an unsaturated zone above the main groundwater table. In the unsaturated soil zone, the pore pressure within soil is negative relative to the atmospheric pressure (Fredlund & Rahardjo, 1993). The negative pore water pressure governs a large variety of special behavior of unsaturated soils, such as stress-strain relationship, hydraulic characteristics, etc. Most slopes are unsaturated on the ground surface. It is necessary to take
matric suction into account in slope stability analyses because not only the shear strength but also the groundwater conditions are associated with matric suction.

If the slope is not saturated, unsaturated soil mechanics should be used when the groundwater condition of a slope is taken into account. Slope geometry and subsoil conditions may not be sufficient to assess the stability of the slope. Therefore, seepage analysis is an essential step conducted prior to the slope stability analyses.

To conduct transient seepage analyses, hydraulic properties of soil are necessary inputs. Soil-water characteristics are among these important input parameters. They can be obtained from laboratory tests. Direct measurement of the water permeability of an unsaturated soil is not only expensive but also time consuming. The most common expediency taken in the current practice is to predict the permeability function based on laboratory measurements of SWCC along with saturated permeability (Fredlund et al., 1994). The method works satisfactorily and it has been used worldwide.

The conventional method to measure SWCCs was bought from soil science, so it is understandable that stress effects have not been taken into account. However, stress effects on SWCCs can no longer be ignored as SWCC is widely used in the geotechnical field.

Some experimental results on stress effects as well as hysteresis on SWCCs of some expansive soils from China are shown and adopted as inputs for the later stage of slope stability analysis. As is well known, the huge infrastructure project – South-to-North (S/N) Water Transfer is to be constructed in this century (Fig. 1.1). More photos taken the in-take of the canal in Hubei Province are shown in Appendix I. The Middle Route Project (MRP) is so far the most practical route for the S/N project, which starts from Hubei and ends in Beijing. The S/N project is aimed at alleviating
the problems caused by the disproportionate water resources distribution over the territory of China. The trunk canal of the MRP of S/N is 1240 km long, in which 180 km is going across some areas with expansive soils (Liu, 1997).

Expansive soil is a term for soil having a potential to swell or shrink as moisture content changes. The expansibility is mainly due to a considerable fraction of a special mineral – montmorillonite - in the soil (Nelson and Miller, 1992). Due to this particular mineral, the volume of soil increases significantly and shear strength decreases upon wetting.

Because of the problematic nature of this type of soil, the slope stability problem of the canal becomes one of the major concerns for geotechnical engineers. It is thus desirable to undertake an extensive study on slope stability for the S/N project. The research of the thesis mainly focuses on stress effects on SWCCs, groundwater responses under rainfall conditions, and slope stability in expansive soils.

Some expansive soils were sampled from Hubei, China. Fig. 1.2 shows the intake of the trunk canal of MRP. Soil samples were sampled from the slopes near this area. The stress effects of soil hydraulic properties have great influence on the slope stability of the canal. Thus, the work in the thesis is of much importance and significance. The results of the thesis, hopefully, will provide pertinent and useful information for a better design and construction of the canal.

1.2 OBJECTIVES

Goals to be achieved through this study, by carrying out experimental work on SWCC and numerical analyses on slope stability, are:

(1) To study the adsorption and desorption characteristics of expansive soils compared with non-expansive soils;

(2) To study stress effects on the SWCCs of ZY expansive soil;
(3) To interpret experimental results from the microscopic point of view based on Scanning Electron Microscopy (SEM) and Mercury Intrusion Porosimetry (MIP);

(4) To investigate the influence of rainfall parameters on the pore-water pressure distribution of a slope subjected to various rainwater infiltration conditions;

(5) To explore the stress effects of hydraulic properties on groundwater flow in an unsaturated expansive soil canal; and

(6) To provide some pertinent and useful information for a better design, construction and maintenance of the canal of the S/N project.

1.3 OUTLINE OF THE THESIS

The thesis is comprised of seven chapters. The present chapter, Chapter 1, is to provide background to the research topic and aims to be achieved. In Chapter 2, previous research work related to the study is reviewed extensively.

The laboratory work of SWCCs is presented in detail in Chapter 3. The test results of SEM and MIP are presented in Chapter 4. From the microscopic point of view, differences in SWCCs reflect the differences in microfabrics of soil samples, such as pore size distributions. SEM is a qualitative tool whereas MIP can demonstrate soil fabric characteristics by showing pore size distribution in a quantitative manner. Different soil fabrics may be brought out by different testing conditions.

A 3D numerical study on the influence of rainfall parameters on an unsaturated slope is reported in Chapter 5. Key results are shown to illustrate that rainfall patterns, rainfall duration as well as return periods have remarkable influence on pore water pressure distributions along the slope after subjection to various rainfall events.
Based on the results obtained in the experimental work on SWCCs, other hydraulic properties are be calculated. Then they are all input into a 2-dimensional (2D) finite element program for seepage analysis. Two series of 2D numerical seepage analyses are implemented to investigate the influence of stress effects in slope stability, by comparing the different groundwater responses with different input parameters, with or without considering stress effects. This numerical part of work is presented in Chapter 6.

The thesis is concluded in Chapter 7, in which major conclusions are summarized and some suggestions for further research are given.

1.4 REFERENCES


Fig. 1.1 Location of the proposed Middle Route of the South-to-North Water Transfer Project
Fig. 1.2  Intake of the trunk canal of the proposed Middle Route for the S/N Water Transfer Project
CHAPTER 2 LITERATURE REVIEW

Every step of advancement of human knowledge is made based on the knowledge of our ancestors. This chapter presents an extensive literature review of previous work relevant to the thesis. Section 2.1 of the chapter starts with the review on the origin and nature of soil, and fundamentals in soil science and soil mechanics are presented. In Section 2.2, some basic concepts relevant to the thesis in saturated and unsaturated soil mechanics are introduced. SWCC and its applications in transient seepage and slope stability analyses are reviewed in detail in Section 2.3. Because expansive soil is the study subject in the thesis, Section 2.4 is to review basic knowledge regarding this particular type of soil. Groundwater modeling and transient seepage analysis is reviewed in Section 2.5. Section 2.6 mainly deals with 2D transient seepage analysis and its implications on slope stability.

2.1 ORIGIN AND NATURE OF ENGINEERING SOIL

As stated by Hillel (1998), a precise definition for soil is 'elusive', because the word means different things to different people. As a scientist in soil physics, he finally defined soil as 'a natural body, engaged in dynamic interactions with the atmosphere above and the strata below, that influences the planet’s climate and hydrological cycle and that serves as a growth medium for a versatile community of living organisms'.
According to Barnes (1995), there are various sources for soil formation. Most soils referred to are naturally-occurring soils, such as decomposed, weathered rocks, water-borne soils, glacial deposits, etc. However, waste materials, such as surplus and residues from construction processes, ashes, slag, quarry waste from industrial processes, are also included as certain particular types of soil.

Conventionally, soil is defined as 'a product of the weathering of rocks, i.e. the deterioration that takes places during geological time span (Ortigao, 1993)'. 'Soil and rock are both conglomerates of mineral grains and the boundary between soil and rocks is somewhat arbitrary and often not clear-cut. Soil is basically uncemented and relatively weak, rock cemented and relatively strong (Milligan and Houlsby, 1984)'.

Weathering processes are caused by chemical and physical agents. If the soil that is weathered remains in at the site of its formation, it can be classified as residual soil; those that have been transported to other places transported by a flow of water are referred to as the sedimentary soils, such as colluvium and alluvium.

Soil particles are composed of various minerals. The most important one is silica (Si), which has strong chemical bonds and a stable crystalline packing resistant to further degradation. Silica particles mainly form sands and silts.

As stated by Milligan and Houlsby (1984), there must be spaces or pores between the soil grains of a soil. These spaces are usually filled with water, air or both. The behavior of soils is significantly affected by the presence of the pore fluid and the ease with which it can move through the soil. The size of the pores is obviously related to the size of the particles, and in clay soils is mostly very small. When water moves through such pores with great difficulty, the soil is nearly impermeable. Capillary forces are very large in such small pores and the resulting 'suction' can hold the material together even if no external stresses are applied. Clay soil therefore appears
to be cohesive and plastic; it can hold itself together in a lump and can undergo substantial distortion without breaking up (Milligan and Housby, 1984).

The arrangement of clay particles greatly affects the soil behavior. Clay mineral particles are so small that their arrangements are referred to as microstructure or microfabric. Scanning Electron Microscopy technology is of great use for microfabric studies. Elementary particle arrangement can be dispersed, or flocculated, or partly discernible, while particle assemblages are more complex. One of the assemblages can be clay coating on silt and sand particles.

The oxygen and hydroxyl ions play a dominating role in the mineral structure due to their great numbers and their large size. They can impart a slightly negative charge because the O²⁻ and OH⁻ ions exist on the surface of mineral sheets, though their negative charges are neutralized by positive cations. When substitution of the cations has occurred, for example, Al³⁺ for Si⁴⁺ or Mg²⁺ for Al³⁺, there is a greater net negative charge transmitted to the particle surface because there are more ions of these elements available at the formation of the soil. Then by adsorption of positive ions (cations) and polar water molecules on the mineral surfaces, the negative charges can be neutralized. Thus, a wide variety of clay mineral structures is possible, due to various combinations of substituted cations, exchangeable cations, interlayer water and structural layers or stacking (Mitchell, 1993).

2.2 UNSATURATED SOILS

As stated previously, the spaces and pores between soil particles can be filled with water or air or a combination of the both fluids. If all the spaces are filled with water, the soil is so-called saturated. Otherwise, it is unsaturated. In arid and semi-arid areas, the groundwater table is usually located deeply under the ground surface. The
soils above the groundwater table have negative pore-water pressure. Changes in climate conditions significantly affect the water contents of soils in the unsaturated zone. Upon wetting, commonly e.g. rainwater infiltration, the negative pore-water pressure increase towards positive values. Many soils exhibit significant swelling or expansion when wetted. Moreover, some soils have great loss in shear strength upon wetting, for instance, expansive soils.

2.2.1 Four-phase materials and suctions

Saturated soils have only two phases, solid and water. It is proposed that unsaturated soils are four-phase materials. In addition to solid, water and air, the air-water interface can be viewed as the fourth phase and is often referred to as the contractile skin. The contractile skin can act like a thin rubber membrane, pulling the particles together. Moreover, it is the source of 'suction' for unsaturated soils.

Total suction $\psi$ is comprised of two components, osmotic suction $\pi$ and matric suction $(u_a-u_w)$. The capillary phenomenon is associated with matric suction. The surface tension on the top meniscus pulls up a certain height of water in the tube and the water under tension has negative pore-water pressure, as illustrated in Fig. 2.1. The vertical force equilibrium of the capillary water in the tube can be written as Eq. 2.1.

$$2\pi r T_s \cos \alpha = \pi r^2 h_c \rho_w g$$

(2.1)

where

$r$ = radius of the capillary tube;

$T_s$ = surface tension of water;

$\alpha$ = contact angle;

$h_c$ = capillary height; and

$g$ = gravitational acceleration.
Equation 2.2 can be rearranged to give the maximum height of water in the capillary tube, $h_c$:

$$ h_c = \frac{2T_s}{\rho_w g R_s} \quad (2.2) $$

where $R_s$ is the radius of the meniscus (i.e. $r \cos \alpha$). The contact angle between the tension force and the glass tube is zero for pure water, therefore, the capillary height is

$$ h_c = \frac{2T_s}{\rho_w g r} \quad (2.3) $$

The radius of the tube is analogous to the pore radius in soils, so clay usually has a greater capillary height than sand, provided that all other external conditions are identical.

2.2.2 Stress variables and shear strength

The effective stress principal is valid in classical soil mechanics. Effective stress is the unique stress state variable for saturated soil mechanics. The shear strength of Mohr-Coulomb theory, which is a most well-known theory in soil mechanics, can be expressed as (Bardet, 1997):

$$ \tau_{\text{max}} = c + \sigma \tan \phi \quad (2.4) $$

where $\tau_{\text{max}}$ is the shear strength; $c$ is the cohesion or cohesion intercept; and $\phi$ is the angle of shearing resistance (internal friction angle). The Mohr-Coulomb failure theory requires that $|\tau| \leq \tau_{\text{max}}$.

Two most commonly used independent stress variables for unsaturated soils are net normal stress and matric suction (Fredlund and Rahardjo, 1993). Therefore, the
shear strength equation with the combination of \( (\sigma - u_a) \) and \( (u_a - u_w) \) can be written as:

\[
\tau_{eff} = c' + (\sigma - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b
\]  
(2.5)

where \( c' \) = intercept of the extended Mohr-Coulomb failure envelope on the shear stress axis where the net normal stress and the matric suction at failure are equal to zero; it is also referred to as ‘effective cohesion’;

\( (\sigma - u_a)_f \) = net normal stress state on the failure plane at failure;

\( u_{of} \) = pore-air pressure on the failure plane at failure;

\( \phi' \) = angle of internal friction associated with the net normal stress state variable,

\( (\sigma - u_a)_f \);

\( (u_a - u_w)_f \) = matric suction on the failure plane at failure; and

\( \phi^b \) = friction angle associated with the matric suction stress state variable, \( (u_a - u_w)_f \).

It can be observed from the above equation that the reduction in negative pore-water pressure can cause a loss in shear strength. Therefore, it is understandable that changes in negative pore-water pressures associated with heavy rainfalls cause numerous slope failures. The extended Mohr-Coulomb failure envelope is illustrated in Fig. 2.2. There is an implicit assumption in the extended shear strength equation that the shear strength increases linearly with increasing matric suction, which is not the case in fact. The non-linearity is also discussed by Fredlund and Rahardjo (1993).
2.3 SOIL-WATER CHARACTERISTIC CURVE

2.3.1.1 Origin and history of SWCC

SWCC is also known as the water retention curve, which originated from Soil Science (Brooks and Corey, 1966; van Genuchten, 1980; Hillel, 1998). As stated by Hillel (1998), in a saturated soil at equilibrium with a body of free water at the same elevation, soil water is at atmospheric pressure. Hence, the hydrostatic pressure is zero. As suction is applied to water in a saturated soil (suction is a water pressure slightly subatmospheric), the soil may not lose water until a critical suction value is exceeded at which the largest surface pore begins to empty and its water content is displaced by air. The critical suction value is called the air-entry suction. Increasing suction is thus associated with decreasing soil wetness. The amount of water remaining in the soil at equilibrium is a function of the sizes and volumes of the water-filled pores and of the amount of water adsorbed to the particles. Hence it is a function of the matric suction. This function is usually experimentally and represented by a curve called the soil-moisture retention curve, also known as the soil-moisture release curve or the soil-moisture characteristic (Hillel, 1998).

Now, the soil-water characteristic curve (SWCC) describes the relationship between suction and water content or the degree of saturation of a soil. Fig. 2.3 shows a typical SWCC for a silty soil, including drying and wetting curves (Fredlund et al., 1998). The soil-water characteristics indicate the water storage capacity of a soil at a given soil suction. The air-entry value, saturated water content, desorption rate as well as the transition point to the residual state are demonstrated in the figure. They are all very important variables for defining SWCC.
2.3.1.2 Applications of SWCCs

The wide application of SWCC in the unsaturated soil mechanics research field is the result of its clear physical meaning and close relationship to other physical and mechanical properties. As stated by Barbour (1998), SWCC provides a conceptual framework in which the behavior of unsaturated soils can be understood. It is a useful parameter for analyzing transient water flows in unsaturated soils and plays a similar role as a consolidation curve describing the relationship between void ratio (or volume) and effective stress in saturated soils (Rahardjo and Leong, 1997; Barbour, 1998). It has been shown that other engineering properties of an unsaturated soil can be predicted using SWCC and saturated soil properties (Brooks and Corey, 1966; van Genuchten, 1980; Fredlund et al., 1994; Vanapalli et al., 1996). However, the prediction methods need more research before they can be widely used.

The phenomenon of hysteresis is commonly observed and important in a SWCC and hydraulic conductivity of an unsaturated soil (Topp and Miller, 1966; Hillel, 1998). More lately, hysteresis effects in SWCC and hydraulic conductivity were discussed by Lin and Benson (2000). Hydraulic properties are essential inputs for both analytical and numerical analyses in geotechnical engineering practice. Therefore, influence of hysteretic phenomenon on transient seepage and slope stability analyses cannot be ignored (Rahardjo and Leong, 1997; Ng et al., 2000). Hysteresis may be attributed to several causes, such as the geometric nonuniformity of the individual pores (ink-bottle effect), the contact-angle effect, the encapsulation of air in 'blind' or 'dead-end' pores and swelling, shrinking or aging phenomena (Hillel, 1998).
2.3.1.3 Stress effects on SWCCs

The effects of wet-dry cycling, stress history and soil structure on SWCCs are not often investigated. One of the few exceptions is that Vanapalli et al. (1999) conducted a series of experiments for studying the influence of soil structure and stress history on the soil-water characteristics of a compacted clayey till. Compacted fine-grained soil specimens were firstly consolidated under a given load in an oedometer to different void ratios before their SWCCs were measured in a pressure plate extractor. Their results suggest that different water contents for the compaction of soil specimens have significant effects on SWCCs due to different structures. Specimens compacted at dry of optimum water content possess a higher desorption rate than those compacted at wet of optimum. The pre-consolidation process has more significant influence on specimens compacted at wet of optimum than those compacted at dry of optimum.

Another exception was made by Ng and Pang (1999, 2000). Using a 1D modified volumetric pressure plate extractor, they measured SWCCs of a typical residual soil in Hong Kong under $K_0$ stress conditions. It was found in the study that the desorption rate of the measured stress dependent SWCCs (SDSWCC) in the suction ranging from 5 to 200 kPa decreases with increasing stresses applied on the soil. It meant that the higher the applied stress, the higher the air-entry value and the water storage in the high suction range (Fig. 2.4). Thus, it was concluded that slope stability was overestimated by ignoring stress effects. The stress dependent hydraulic properties were then applied in a numerical seepage analysis and it was found that the case using stress dependent parameters predicted a lower matric suction profile (a higher pore-water pressure distribution) in an unsaturated slope of residual soil (Ng and Pang, 2000).
2.4 EXPANSIVE SOILS

2.4.1 General description

Expansive soil is a term used for those soils that have a potential for swelling due to an increase of moisture content (Nelson & Miller, 1992). Expansive soil increases in its volume when the water content increases. Expansive soils can be found on almost every continent of the Earth. Destructive results caused by these types of soil have been reported in many countries worldwide such as United States, Australia, South Africa and China (Steinberg, 1998). Expansive soils are indeed a typical type of unsaturated soil due to climatic conditions in expansive soil areas. The matric suction, i.e. negative pore water pressure, in the surface layer is often high, which leads to a high degree of expansion in surficial soils.

Several indices are commonly used to describe and classify expansive soils, according to Nelson and Miller (1992).

1. **Free swell** is defined as the ratio of a known volume of dry soil passing the No. 40 sieve into a graduated cylinder filled with water and the change of the measured volume of the swelled volume compared with its initial value after it has completely settled. It is expressed as a percentage. A high grade commercial bentonite (sodium montmorillonite) will have a free swell value from 1200 to 2000 %. Soils with free swell values lower than 50 % are normally not considered to exhibit appreciable volume change, however, there might be some exceptions due to extreme climatic conditions.

2. **Swelling pressure** is measured by potential volume change (PVC), a standardized apparatus. It is only applied for measuring swelling pressure of a
compacted sample. The sample is wetted in the device and allowed to swell against a proving ring.

3. **Expansion index** is calculated with Eq. 2.6 based on the volume change measured on a compacted sample at optimum water content wetted to 50% saturation.

\[
EI = 100 \cdot \Delta h \times F
\]  
(2.6)

where \(\Delta h\) = percent swell; and \(F\) = fraction passing No. 4 sieve.

China has a wide distribution of expansive soils over her territory, especially in the southern and southwestern areas. There will be 180 km long excavated canal crossing expansive soil areas along the MRP of the S/N Water Transfer (Liu, 1997). The appraised cost for constructing an expansive soil canal would be much higher due to expansive soil slope failures. Therefore, it is necessary and meaningful to carry out the study on slope instability of expansive soils.

2.4.2 Physical, mineralogical properties of expansive soils

According to Whitlow (1995), clay minerals are produced mainly from the weathering of feldspars and micas. They form part of a group of complex aluminosilicates of potassium, magnesium and iron, known as layer-lattice minerals. Fig. 2.5 shows two basic structural units: the tetrahedral unit, comprising a central silicon ion with four surrounding oxygen ions, and the octahedral unit, comprising a central ion of either aluminium or magnesium, surrounded by six hydroxyl ions. Note that, in both, the metal (with positive valency) is on the inside and the negative non-metallic ions form the outside. The lattice layer structures are shown in Fig. 2.6. A silica layer is formed of linked tetrahedra and a gibbsite or brucite layer might be formed by
of the clay particles and the chemistry of the soil water surrounding those particles (Nelson and Miller, 1992). In that book, three important structural groups of clay minerals are described for engineering purposes. They are Kaolinite type, Mica-like type and the Smectite group. The Kaolinite type is generally considered nonexpansive and the Mica-like group generally does not pose significant problems, but the Smectite group is mainly composed by a high proportion of montmorillonite leading to the strong swell-shrink property of expansive soil. The volume changes of these two minerals with different ions are shown in Fig. 2.8 (Barnes, 1995). It can be seen that the montmorillonite with Na⁺ has the highest degree of expansion (Chen, 1988). It implies that the soil containing this type of montmorillonite has high swell potential.

Particularly for the S/N project, the mineral components of the soils studied were mainly made up of montmorillonite and illite (Liu, 1998). The content of the former is more than 15%, and up to 55%. Therefore, it is understandable that these soils exhibit substantial shrinkage/swelling due to change in moisture content.

2.4.3 Engineering properties of expansive soils

According to the detailed investigation on the expansive soils along the middle route of S/N (Liu, 1998), considerable detrimental results may occur in those soils with a free swell index greater than 80, based on an extensive investigation for expansive soil engineering projects. Swelling pressure varies with water content and dry density. The approximate range is 80-246 kPa for Nanyang expansive soils reported in Liu’s book. It is also shown that the expansibility is somehow anisotropic. For the sample from Nanyang, Henan, its swell-shrink expansibility in the vertical direction is
linked octahedral units. The spacing between the outer ions in the tetrahedral and octahedral layers is sufficiently similar for them to link together via mutual oxygen or hydroxyl ions. Two stacking arrangements are possible, giving either a two-layer or a three-layer structure. More details are given by Mitchell (1993) about clay minerals and structures.

Clay minerals are those members of the layer-lattice group and four main groups of clay minerals may be identified: Kaolinite, illite, montmorillonite and vermiculite. The kaolinite structure consists of a strongly bonded two-layer arrangement of silica and gibbsite sheets. Kaolinite itself is a typically flaky mineral, usually with stacks of about 100 layers in a very regular structure. The structure of illite group consists of three-layer gibbsite sheets with K⁺ ions providing a bond between adjacent silica layers. The linkage is weaker than that in kaolinite, resulting in thinner and smaller particles. As for the montmorillonite group, the minerals are also referred to as smectites. Essentially the structure consists of three-layer arrangements in which the middle octahedral layer is mainly gibbsite but with substitution of Al by Mg. A variety of metallic ions other than K⁺ provides weak linkage between sheets. As a result of this weak linkage water molecules are easily admitted between sheets, resulting in a high shrinking/swelling potential. The vermiculite group has a similar structure to montmorillonite, except that the cations providing inter-sheet linkage are predominantly Mg, accompanied by some water molecules. The shrinkage/swelling potential is therefore similar, but less severe, than that of montmorillonite. Their respective properties are listed in Fig. 2.7 (Whitlow, 1995).

The mechanism of swelling in expansive clays is complex and is influenced by a number of factors. The swelling capacity of an entire soil mass depends on the amount and type of clay minerals in the soil, the arrangement and specific surface area
greater than the lateral one. It is stated that collapse may take place when overly wetted (Liu, 1997).

Plasticity and density can provide a great deal of insight regarding the expansive potential of soils. Moisture content relative to limiting moisture contents such as the plastic limit and shrinkage limit indicates the shrink-swell potential. Generally, the lower the initial moisture content, the higher the potential will be. As stated in Liu’s book, the expansive soil from Jiangzhuang, Henan has a swelling ratio of 29.5 % under a water content of 13.8 %, whereas the ratio decreases to 5.1 % when the water content is 27.4 %. The swelling potential is proportional to the dry density of an expansive soil.

Similar to other clay soils, the permeability of expansive soils along S/N water transfer canal is relatively low, and groundwater mainly flows through the cracks (Liu, 1997). The approximate permeability of soil block is $10^{-8} - 10^{-10}$ m/sec, and the value measured in situ is about $10^{-7} - 10^{-8}$ m/sec. Besides, the permeability of expansive soils is affected greatly by the distribution intensity, and opening widths of existing cracks in field.

Shear strength of expansive soils is significantly influenced by moisture content and stress history. Some unconfined undrained test results were presented by Liu (1997). It was found that with an increment of 10% in water content, the cohesion $c$ of expansive soils decreases about 90%, and the internal friction angle $\phi$ decreases 28%. It was also found that the shear strength was reduced by 17%-22% after the first two wetting-drying cycles, but it ceased to further decrease in subsequent cycles. In addition, the long-term shear strength is 85% of the peak strength.

Alonso (1998) summarized previous work from 1950’s on the experimental behavior of expansive soils. It was stated that the swelling strain or swelling pressure for
a simple wetting path decreases with the initial water content, so it does with the confining pressure. For a recompacted soil, the quantities related to swelling increases with its compacted dry density. The swelling pressure can increase from 10 to 18 psi, when the dry density increases from 1480 to 1540 kg/m$^3$. As for cyclic wetting-drying paths, stress dependency is clearly demonstrated and irreversible accumulation of expansion or shrinkage strains may take place. The accumulated volumetric strain is up to 1 % after 10 cycles after Pousada-Presa (1984), according to Alonso (1998).

Habib et al. (1995) studied various stress paths on the swelling and consolidation behavior of a compacted expansive soil under controlled suction in a newly modified oedometer. Similar modes of changing void ratio and water content were observed. It was claimed that the tested soil behaved in an elasto-plastic manner during loading and unloading. Habib (1995) also observed that the lateral pressure of unsaturated expansive clay varied to a given vertical stress in a stress loop. Some experimental work has also been done on unsaturated expansive soils. As for the South-to-North project, a detailed investigation was carried out on the expansive soils along its route (Liu, 1997). Similar behavior has also been observed. For example, the swelling pressure or swelling strain increases with the initial water content and decreases with its dry density, either for an intact sample or for a compacted soil. According to Liu (1997), the swelling pressure decreases from 250 to less than 20 kPa when the water content increases from 16 to 30 % for some natural Nanyang expansive soil. The swelling pressure increases from 20 to 250 kPa, as the dry density increases from about 1500 to 1800 kg/m$^3$.

Direct measurements on unsaturated soil properties are costly and time-consuming, thus SWCCs are also often used for predicting permeability function and shear strength of expansive soils, for the sake of ease and convenience and reliability.
However, little literature could be found on the prediction of shear strength using SWCCs with the effect of volume change considered. One exception was made by Zhan et al. (1998) proposing a modified equation for predicting shear strength of unsaturated expansive soils by using SWCC and shrinkage curve. To verify the equation, triaxial shear tests for unsaturated expansive soils were performed on a modified triaxial apparatus. The results had good agreement with the prediction from that modified equation. Nevertheless, systematic work on stress-strain behavior of expansive soils under controlled suction so far can hardly be found in literature.

2.5 GROUNDWATER MODELING

As stated by Bear and Verruijt (1987), in management of groundwater resources, a tool is needed in order to predict the outcome of implementing management decisions. Depending on the nature of the management problem, its decision variables, its objective functions, and its constraints, the outcome may take a variety of forms such as future spatial distributions of water levels, water quality, land subsidence. The tool is the model. Defined by Wang and Anderson (1982), a model is a tool designed to represent a simplified version of reality.

Mathematical models of groundwater flow have been used since 1800s, as stated by Wang and Anderson (1982). A mathematical model consists of a set of differential equations that are known to govern the flow of groundwater. According to Wang and Anderson (1982), using a groundwater model, it is possible to test various management schemes and to predict the effects of certain actions. The validity of the predictions depends on how well the model approximates field conditions. Good field data are essential when using a model for predictive purposes. However, an attempt to model a system with inadequate field data can also be instructive as it may serve to
identify those areas where detailed field data are critical to the success of the model. In this way, a model can help guide data collection activities.

The basic law used in groundwater flow is Darcy’s Law (Darcy, 1856), written as Eq. 2.10 (after Zaradny, 1993):

\[
q = \frac{Q}{A} = -k \Delta h / L \quad (m \cdot s^{-1})
\] (2.7)

in which \( k \) = coefficient of proportionality known as hydraulic conductivity or coefficient of permeability (m/s); \( A \) = cross-section area perpendicular to the direction of motion (m\(^2\)); \( L \) = length of column (m) where the total hydraulic head loss occurred for \( \Delta h \); and \( Q \) = total flow rate (m\(^3\)/s).

The coefficient of permeability may vary with space (heterogeneity) and direction (anisotropy) in an unsaturated soil. However, in most cases, we assume the coefficient of permeability is isotropic for simplicity. The simplified governing equation for 2D groundwater flow with isotropic permeability is as Eq. 2.11 (Fredlund and Rahardjo, 1993):

\[
\frac{\partial}{\partial x}\left( k_w \frac{\partial h_w}{\partial x} \right) + \frac{\partial}{\partial y}\left( k_w \frac{\partial h_w}{\partial y} \right) = m_w \rho_w g \frac{\partial h_w}{\partial t}
\] (2.8)

in which \( m_w \) = coefficient of water volume change with respect to a change in matric suction, \( (u_w - u_m) \); \( h_w \) = hydraulic head (i.e., gravitational plus pore-water pressure head); \( k_w \) = coefficient of permeability with respect to water as a function of matric suction; \( \rho_w \) = density of water; \( g \) = gravitational acceleration; and \( t \) = time.

The equation for 3D flow can also be written in a similar form, as (FEMWATER, 1997):
\[
\frac{\rho}{\rho_0} \frac{\partial h}{\partial t} = \nabla \left[ K \left( \nabla h + \frac{\rho}{\rho_0} \right) \right] + \frac{\rho^*}{\rho_0} q \tag{2.9}
\]

\[
F = \alpha' \frac{\theta}{n} + \beta' \theta + n \frac{ds}{dh} \tag{2.10}
\]

where

F = storage coefficient;

h = pressure head;

t = time;

K = hydraulic conductivity tensor;

z = potential head;

q = source and/or sink;

\( \rho \) = water density at chemical concentration C;

\( \rho_0 \) = referenced water density at zero chemical concentration;

\( \rho^* \) = density of either the injection fluid or the withdrawn water;

\( \theta \) = moisture content;

\( \alpha' \) = modified compressibility of the medium;

\( \beta' \) = modified compressibility of the water;

n = porosity of the medium; and

S = saturation.

For most geotechnical engineering projects, it is assumed that the chemical concentration of groundwater is zero, so that \( \rho / \rho_0 \) is equal to unity. Hence, the equation can be simplified into (Eq. 2.14):
in which $H$ = total head (hydraulic head).

It is impossible to solve these differential equations without the help of numerical methods. Numerical analyses are now widely applied in groundwater modeling. Based on governing equations, a finite difference or finite element program can be applied. The commercial program – FEMWATER is a typical 3D finite element program. The groundwater responses under various rainfall conditions were investigated by FEMWATER is to explore the possible causes for the massive landslide that occurred in the slope at Laiping Road, Hong Kong, 1997 (Tung et al., 1999).

Flow-deformation coupled analysis has attracted a lot of attention recently. (Thomas and He, 1994; Alonso and Battle, 1995; Alonso et al., 1998). Researchers believe that coupled analyses lead to better understanding of groundwater responses under various conditions and hence mechanisms for slope failures. For expansive soils, a considerable volume change will occur due to the variation of moisture content of a soil mass. Flow-deformation coupled analysis may be more pertinent and necessary for groundwater modeling in expansive soils.

2.6 RAINFALL-INDUCED SLOPE INSTABILITY

Many problems in soil mechanics involve the stability of slopes. These may be natural slopes, embankments, canal levees, man-made cuts, whose stability is perhaps threatened by natural erosion, or man-made excavation, or the side of slope of cuttings or rainwater infiltration, even leakage from drainage trunks.

According to Terzaghi (1950), there are two ways in which landslides can be set in motion:
1. External causes which result in an increase in shearing stress. These shearing stresses increase along the surface of failure until the time of failure;

2. Internal causes which result in a decrease in the shearing resistance of the material.

In addition to the two main causes, there may be an intermediate group, with a combination of both internal and external causes. Regarding a potential slope failure, the factor of safety of a slope can be expressed as:

\[
\text{FOS} = \frac{\text{Shear resistance}}{\text{Shear stress on the potential slip surface}}
\]  \hspace{1cm} (2.12)

The slope is considered safe only when FOS is equal to or greater than unity.

As stated by Kenny (1984), the most important soil property among the properties and behavior of soils related to mass movements is shear strength, more precisely, effective shear strength. The shear strength developed along a surface in soil or rock is dependent on the normal effective stress acting on the surface and the effective shear strength properties of the materials through which the surface passes. There are several approaches such as Bishop for calculating the Factor of Safety of a slope with the same principle behind (Fredlund and Rahardjo, 1993).

Rainfall-induced slope instability is a major concern for many regions worldwide such as South Africa and Hong Kong (Brand, 1995; Fourie, 1996). Because slopes are mostly unsaturated near the ground surface, the negative pore-water pressure plays an important role in slope stability. Wetting causes strength reduction and increase in shear stresses on the potential slip surface. The results of seepage analyses of groundwater flow are essential inputs for the slope stability analyses (Sammori and Tsuboyama, 1991; Ng and Shi, 1998; Ng et al., 1999). Matric suction, which contrib-
utes to the shear strength of unsaturated soils is easily destroyed by the rainwater infiltration. As a result, a landslide is likely to occur due to the reduction of the FOS of the slope.

A field instrumentation program was carried out to continuously and simultaneously monitor the in situ matric suction and the rainfall on a residual soil slope (Lim, et al., 1996). According to their observations, a significant reduction in matric suction occurred on the slope due to rainwater infiltration. As reported by Fourie (1996), many slope failures observed were not caused by an evident rise in the groundwater table, but could be attributed to the migration of a wetting front into the slope. Infiltration of rainfall may be sufficient to reduce the matric suction in the surficial soil to a value that is low enough to trigger a shallow failure (Fourie et al., 1999). Anderson and Sitar (1995) observed the debris flow induced by rainwater infiltration involves both drained initiation and undrained mobilization. Because of the obvious correlation of landslides and rainfall events, some researchers made some effort to establish certain kinds of relationship between these two phenomena (Fourie, 1996; Finlay et al., 1997; Ried, 1998)

More specifically, rainwater infiltration may have a more serious adverse influence on slope stability for unsaturated expansive soils, when this type of problematic soils is of interest. As stated before, the shear strength of unsaturated expansive soils decreases significantly with moisture contents, due to the special mineral compositions and fabrics of this type of soils. Thus, the FOS of a slope may be reduced abruptly and a landslide might occur suddenly.

However, there is so far not much literature available on slope stability particularly for expansive soils. Some aspects on expansive soil slope stability were addressed in Bao and Ng (2000). In that paper, the possible causes and failure mecha-
nisms particularly for expansive soil slopes were elaborated and some thoughts were given for early prediction on slope instability in expansive soils. A comprehensive site investigation for tertiary expansive clays was carried out and involved laboratory and in situ tests (Ortigao et al., 1997). In the study, slope failures were attributed to two proposed causes, namely, shallow failure due to expansion and surface degradation and lack of shear strength to resist stresses.

2.7 REFERENCES


Fig. 2.1  Physical model and phenomenon related to capillarity (after Fredlund and Rahardjo, 1993)
Fig. 2.2  Extended Mohr-Coulomb failure envelope for unsaturated soils (after Fredlund and Rahardjo, 1993)
Fig. 2.3 Definition of variables associated with the soil-water characteristic curve (after Fredlund et al., 1998)
Fig. 2.4  Stress effects (K₀ stress conditions) on SWCC of a CDV in Hong Kong (after Ng and Pang, 2000)
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<th>Mineral name</th>
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<th>Between layers</th>
<th>Approximate size (μm)</th>
<th>Specific surface (m²/g)</th>
<th>Approx. exchange capacity (me/100g)</th>
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<td><img src="image" alt="Kaolinite Structure" /></td>
<td>H-bond linkage</td>
<td>(l = 0.2 \text{--} 2.0) (t = 0.05 \text{--} 0.2)</td>
<td>10\text{--}30</td>
<td>5</td>
</tr>
<tr>
<td>halloysite</td>
<td><img src="image" alt="Halloysite Structure" /></td>
<td>(\text{H}_2\text{O})</td>
<td>((\text{tubular})) (l = 0.5) (t = 0.05)</td>
<td>40\text{--}50</td>
<td>15</td>
</tr>
<tr>
<td>illite</td>
<td><img src="image" alt="Illite Structure" /></td>
<td>K⁺ linkage</td>
<td>(l = 0.2 \text{--} 2.0) (t = 0.02 \text{--} 0.2)</td>
<td>50\text{--}100</td>
<td>30</td>
</tr>
<tr>
<td>montmorillonite</td>
<td><img src="image" alt="Montmorillonite Structure" /></td>
<td>weak cross-linkage between Mg/Al ions</td>
<td>(l = 0.1 \text{--} 0.5) (t = 0.001 \text{--} 0.01)</td>
<td>200\text{--}800</td>
<td>100</td>
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<tr>
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Fig. 2.7  Structure and size of the main clay minerals (after Whitlow, 1995)
Fig. 2.8 Volume changes of montmorillonite and kaolinite clays during hydration (after Barnes, 1995)
CHAPTER 3 LABORATORY TESTS ON SWCC UNDER DIFFERENT STRESS CONDITIONS

3.1 INTRODUCTION

Head (1992) stated from the laboratory testing point of view that soil mechanics is that branch of engineering science, which applies the principles of mechanics, hydraulics and geology to the solution of engineering problems in soils. The study of soil mechanics covers the investigation, description, classification, testing and analysis of soils to determine their inter-reaction with structures built in or upon them, or built with them.

Soils are regarded as engineering materials. Their physical characteristics can be determined by experiment, and these properties can be used to predict their likely behavior under defined working conditions. But unlike other engineering materials such as metals and concrete, over which control can be exercised during manufacture, soils are natural materials, which more often than, not have to be used in their natural condition. Inside the scope of geotechnology, even when some kind of processing is possible, either in situ or by using excavated material, the soil can be modified only to a limited extent by relatively simple procedures on site. The variety of soils is very wide indeed, and no two sites have identical soil conditions. It is therefore necessary to evaluate the physical properties and engineering behavior of the soils present at every site that is developed in any way.

There are two main categories of soil testing:
1) Classification tests, which indicate the general type of soil and the engineering category to which it belongs.

2) Tests for the assessment of engineering properties.

In a site investigation for a construction project, the field operations, which includes studies of the geology and history of the site, subsurface exploration and in situ testing, are of prime importance. The determination of the ground characteristics by in situ testing can take into account large-scale effects, such as soil fabric, structure and discontinuities of strata, which cannot be represented in small laboratory specimens. Nevertheless, the measurement of soil properties by means of laboratory tests offers a number of advantages, which can be summarized as follows:

(1) Full control of the test conditions, including boundary conditions, can be exercised.

(2) Laboratory testing generally permits a greater degree of accuracy of measurements than does field testing.

(3) Control can be exercised over the choice of material which is to be tested.

(4) A test can be run under conditions which are similar to, or which differ from, those prevailing in situ, as may be appropriate.

(5) Changing in conditions can be simulated, as can the conditions which are likely to occur during or after completion of construction.

(6) Testing can be carried out on soils which have been broken down and reconstituted, or processed in other ways.

In this chapter, firstly some basic laboratory tests including their principals and methods are presented. Emphasis is imposed on SWCCs laboratory measurement. Testing apparatus, sample preparation, procedures and test results are presented in detail. Results are compared and discussed with other non/expansive soils that exist in
the literature. Fitted curves and predictions from the database of Soilvision – a software developed for geotechnical engineers working with unsaturated soils (SoilVision Systems Ltd., 1997) are also presented.

3.2 GENERAL DESCRIPTION OF LABORATORY TESTS AND SOIL SAMPLES

1. The soils tested in the lab are from ZaoYang (ZY) and Liang Zhuang (LZ), Hubei Province in Central China. They are both in the category of Nanyang expansive soils. These two sites are both along the middle route project (MRP) of the S/N water transfer project. Some photographs on the sampling sites of ZY and LZ are shown in Appendix II.

2. Both soils are expansive soils, whose volumes tend to increase during wetting. The specimens taken to the lab include both intact samples from the field and disturbed samples. These two samples are similar in terms of soil properties.

3. In this chapter, the test principles and procedures for some basic properties are presented, namely, grain size distribution, particle density and plastic index. The test of SWCC is presented in detail in the later part of this chapter.

The natural soil properties of LZ and ZY are listed in Table 3.1. The natural sample of ZY is available. One photograph is shown in Fig. 3.1. It is brown-yellow colored with some black nodules, as seen from this figure. These black nodules are mainly composed of Calcium-Manganese. It can be observed that the soil is mainly made up of fine particles.
3.3 GRAIN SIZE DISTRIBUTION

A soil consists of an assemblage of discrete particles of various shapes and sizes. The object of a particle analysis is to group these particles into separate ranges of sizes and to determine the relative proportions, by dry mass, of each size range.

Grain size analyses are also referred to as particle size distribution (PSD) tests, sizing tests or mechanical analysis (MA) tests. The sieving and the sedimentation are the two distinctive procedures. These two types of tests differ in methods and procedures, in order to cover the very wide range of particle sizes which are encountered. Sieving is used to group gravel and sand particles that can be separated into different size ranges with a series of sieves of standard aperture openings. Sedimentation is used for silt and clay size particles, when it is not possible to use sieving. For soils containing both coarse and fine particles, composite tests using both sieving and sedimentation methods have to be carried out if a full particle size analysis is required.

Considering the appearances of the soils concerned in this project, composite tests are not required, since it is apparent that the soil samples are mostly composed of very fine clay particles.

3.3.1 General principles of sedimentation test

The theory of sedimentation is based on the fact that large particles in suspension in a liquid settle more quickly than small particles, assuming that all particles have similar densities and shapes. The velocity which a falling particle eventually reaches is known as its terminal velocity. If the particles are approximately spherical, the relationship between terminal velocity, V, and particle diameter, D, is given by Stokes Law, named after Sir George Stokes (1891). It is stated that the terminal velocity is proportional to the square of the particle diameter. Although clay particles are
far from spherical, the application of Stokes Law based on diameters of equivalent spheres provides a basis for comparison of the particle size distribution in fine soils which is sufficiently realistic for most practical purposes. In a suspension of a known mass of fine soil particles of various sizes, the particles are allowed to settle under gravity. This process is known as sedimentation. From certain measurements made at known intervals of time, the distribution of particle sizes can be assessed. At the very beginning, all particles are assumed to be uniformly suspended in the suspension. In the process of sedimentation, the larger the particle is, the shorter time it takes to settle down to the bottom of the water container. The density of the suspension in the upper part of the container varies with sedimentation. As large particles settle down, the density in the upper portion decreases. The density is measured by a hydrometer.

3.3.2 Testing procedure

1. Calibration of the hydrometer and meniscus correction. The correction of the meniscus for the tests is found to be 0.22 cm.

2. Pretreatment

Pretreatment is necessary if the soil sample contains organic matters. However, these two samples are inorganic (Liu, 1997), so the pretreatment is omitted.

3. Dispersion

a) Weigh 3.3g of sodium hexametaphosphate and 0.7g of sodium carbonate in distilled water to make 100ml dispersant solution.
b) Weigh about 30 g oven-dried clay sample (40°C – 50°C). (Note: Oven drying of some soils, in particular some tropical soils, can change the particle size properties. Since these two samples are both unsaturated expansive soils, oven drying at 100°C to 110°C is avoided.)

c) Add the soil to the dispersant solution in a conical flask. Shake the mixture thoroughly until all the soil is in suspension.

d) Transfer the suspension to a glass bottle sealed with a rubber lid. Shake the bottle in the mechanical shaking device for at least 4 hr.

e) Transfer the suspension from the bottle to the 63 μm sieve placed on the receiver and wash the bottle and the sieve with a jet of distilled water from the wash bottle.

f) Transfer the suspension from to the 1000ml measuring cylinder and make up to the 1000ml mark with distilled water.

4. Sieving

a) Transfer the soil particles retained on the 63 μm test sieve to an evaporating dish and dry in the oven of 40°C to 50°C.

b) Resieve the material on the sieves down to the 63 μm size. Weigh the material retained on each sieve.

c) Add any material passing the 63 μm sieve to the measuring cylinder.

5. Sedimentation

a) Add 100ml of the dispersant solution to the second 1000 ml measuring cylinder and make up to 1000ml solution with distilled water.

b) Insert a rubber bung and shake the dispersant solution thoroughly and place it upright on the table.
c) Insert a rubber bung into the cylinder containing the soil suspension and shake it vigorously end-over-end about 60 times and immediately place it on the table.

d) Start the stopwatch as a timer to record elapsed time. At the instant, immerse the hydrometer into the soil suspension to a depth slightly below its floating position and allow it to float freely.

e) Take a reading from the hydrometer at the upper rim of the meniscus after periods of 0.5 min, 1 min, 2 min and 4 min.

f) Remove the hydrometer slowly, rinse it in distilled water and place it in the cylinder of dispersant solution. Observe and record the top of the meniscus reading.

g) Reinsert the hydrometer in the soil suspension and take and record readings after periods of 8 min, 30 min, 2 h, 8 h, and 24 h from the start of sedimentation.

3.3.3 Plotting and presentation

The particle size distributions of LZ, ZY and CDV (from Peak Road, Hong Kong) are shown in Fig. 3.2(a). It can be seen from the plot that no particle has a diameter greater than 2 mm. LZ and ZY both have a high portion of fine particles (d < 0.002 mm), up to 54% and 49%, respectively, according to British Standard (BS). Detailed results are shown in Table 3.2. The measured grain size distributions of ZY and LZ are very close to each other, and they both have a high percentage of fine particles. The particles of the CDV from Peak Road are generally coarser. Fitted curves obtained using SoilVision (SoilVision Systems Ltd., 1997) are presented in Fig. 3.2(b). It can be seen that the fitted curves have very agreement with the measured data. However, as the particle diameter is smaller than 0.002 mm, the fitted curves of LZ and ZY give the same distribution for the two soil samples.
Referring to the triangular chart distributed by the US Bureau of Reclamation as shown in Fig. 3.3, LZ and ZY are both identified as clay and Peakroad CDV is clayey silt (Bardet, 1997). According to the program of Soilvision, LZ and ZY are both classified as inorganic clay, while CDV is clayey silt. Summarizing the above results, LZ and ZY are clay while the CDV is clayey silt.

3.4 GRAIN DENSITY

The density bottle method (small pykometer) is suitable for these two expansive soils and the Peak Road CDV, which are all mainly composed of fine particles. This is a traditional method for the accurate measurement of the density of particles heavier than water, or of the density of liquids.

Test procedure:
(a) Preparation of density bottles: Wash each density bottle with stopper. Rinse with alcohol-ether mixture and dry by blowing air through them for 1 day.
(b) Obtain 20g sample that has already been ground with pestle and mortar to pass a 2mm BS sieve. Oven dry at 40°C - 50°C.
(c) Weigh each dry density bottle with its own stopper and record the weight for each bottle, $m_1$.
(d) For each sample 3 specimens are tested. Add about 8 g dry soil to each density bottle.
(e) Weigh each density bottle with soil and the stopper and record the data, $m_2$.
(f) Add de-aired distilled water to the density bottle until the water can cover all the soil in the bottle.
(g) Put the bottles in a desiccator and apply a vacuum to move air bubbles trapped in the soil. Leave the bottles in the vacuum desiccator for 2 hours.
(h) Release the vacuum and remove the bottles out from the desiccator. Add some de-aired distilled water into the density bottles and stir the suspension carefully.

(i) Repeat the above step until no more air is evolved from the soil.

(j) Remove the density bottles and add water to fill the bottle. Insert the stopper and leave for one hour.

(k) Ensure the bottles full of water and no excessive water drop outside the bottles. Weigh each bottle with soil and water and record the data, \(m_3\).

(l) Clean out the bottle, and fill it with de-aerated water. Insert the stopper and leave it in the room for one hour.

(m) Weigh the bottle and record the mass of the bottle full with de-aired water, \(m_4\).

The particle density can be calculated by the use of Eq. 3.1

\[
\rho_p = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)}
\] (3.1)

The particle densities of LZ and ZY are 2.72 and 2.73, respectively. Peak Road CDV has a relatively lower particle density of 2.66.

3.5 LIQUID AND PLASTIC LIMIT TESTS

The use of Atterberg limits to predict the swell potential is definitely the most popular approach. The liquid limit is the water content that defines the boundary between the plastic state and liquid state. The boundary of plastic state and semi-solid state is defined by the plastic limit.
3.5.1 Liquid limit

Liquid limits are also known as Atterberg limits. In this test, since only a small amount of soil is available, the one-point method is used.

3.5.1.1 Sample preparation

The soil is used in the natural state without drying before testing. (The ZY sample is intact in that stage and the LZ is disturbed. Thus a representative sample of LZ is used in its natural state. The ZY sample is sifted with a 425 μm test sieve and only the particles passing the sieve are collected for testing.) A certain amount of soil is mixed thoroughly with distilled water on a glass plate, using two palette knives. During this process, all the coarse particles are removed from the sample. The soil sample is ready until the mixture of soil and water becomes a homogeneous paste and no surplus water is visible. After that, it is left in an airtight container for 24 hours. Since both of the soils used are high plasticity clays, enough time to mature is necessary to provide reliable results.

3.5.1.2 Test procedures

a) Check the apparatus condition.

b) Take a certain amount of soil paste with the spatula and press it against the side of the cup. It is of great importance that no air should be trapped in the paste. Press more paste into the cup until the cup is filled with soil paste. Finally smooth the top surface.

c) Lock the cone and lower the supporting assembly carefully so that the tip of the cone is within a few millimeters of the surface of the soil in the cup. Adjust the
cone tip so that the tip just touches the soil surface. The tip of the cone should be as clean as possible.

d) Press the button and let the cone fall under its gravity.

e) Take a reading of the dial gauge 5 s after.

f) Remove the soil paste and add a little water and mix again (It depends if the reading is less or more than 20 mm. Clean the cone and repeat penetration until the penetration depth is 20 mm.

g) Take about 10 g moisture content soil sample from the area penetrated by the cone at that instant and transfer it to a water content box. Weigh the box before taking it to the oven of 100°C -110°C.

h) The water content at which the penetration depth is 20mm is the Atterberg limit, i.e. liquid limit.

3.5.2 Plastic limit

The test is to determine the lowest moisture content at which the soil is plastic. The test may be carried out either on soil in its natural state or on soil prepared by the wet preparation method. It is usually carried out in conjunction with the liquid limit test.

3.5.2.1 Preparation of sample

The preparation procedure is skipped, since the soil used for the plastic limit is the soil left from the liquid limit test. Leave the wet soil open to the air after the liquid limit test for 24 hr when necessary.

3.5.2.2 Procedure

a) Mix the soil sample thoroughly and take about 20 g of the paste.
b) Spread the paste on a glass mixing plate to further dry the soil.

c) When it is plastic enough, it is well kneaded and then shaped into a ball. Mould the ball between the fingers and roll between the palms of the hands.

d) Roll a small amount of soil into a thread of about 3mm diameter, at that point the crumbling of the thread occurs. (The metal rod serves as a reference.) The soil thread is put into a closed container.

e) Repeat the above stage for several more times and place in the same container.

f) Weigh the container and dry in the oven. *** 6.19/2000

g) Calculate the moisture content at the plastic limit. Two containers are needed to make a comparison between each other. The difference should be within 0.5% moisture content.

3.5.3 Classification based on soil properties

The measured liquid limits and plastic limits as well as their particle density are listed in Table 3.3. It can be seen that LZ and ZY are typical clay of high plasticity (CH) and silt of low (ML) plasticity, according to the USCS system (Bardet, 1997). So far, since most of the soil properties for classification have been obtained, the author would like to classify soil samples with proper names. However, until now, there has not evolved a standard classification procedure for identifying expansive soils (Nelson and Miller, 1992). Different schemes are used in different locations. The most confusing aspect of expansive soil classification is the lack of a standard definition of swell potential.

A number of classification methods were addressed in the book of Nelson and Miller (1992). It has been found that different methods use different soil properties. Parameters such as plastic index (PI), liquid limit (LL), shrinkage limit (SL) and col-
loid content are most frequently used. Fine-grained soils shrinks continuously when the water content decreases. Shrinkage limit is the water content at which the soil volume cease decreasing along a drying path (Bardet, 1997). Some of representative schemes are adopted here to classify the soil samples. Both ZY and LZ can be classified as having medium to high swelling potential if only plasticity index is used, whereas they are clay soils of high volume change potential using PI and clay particles ($2\mu$) (Chen, 1988). When the percentage passing No. 200 sieve combined with LL is used, LZ and ZY seem to possess high degree of expansion (Chen, 1965). If colloid content is used, together with PI and SL, LZ and ZY are found to have medium to high degree of expansion according to Holtz and Gibbs (Nelson and Miller, 1992). According to Liu (1997), LZ and ZY can be identified as expansive soils of medium degree of expansion, based on the plasticity chart initially developed by Casagrande and further improved by Li Shenglin (Liu, 1997). More specifically, based on particles smaller than 0.002 mm, LL and SL, these two samples are classified as an expansive soil of medium to high degree of expansion, based on the investigation carried out particularly for Nanyang expansive soils (Liu, 1997). Therefore, based on the above classification schemes discussed, it may be said that these two samples are expansive soils of medium to high degree of expansion. Or in other words, their swell potential is about medium to high.

3.6 SOIL WATER CHARACTERISTIC CURVE (SWCC) TESTS

3.6.1 Introduction for SWCC

Conventionally, SWCCs are measured under zero external stress through conventional apparatus such as the volumetric pressure plate extractor and soil is assumed
to have no volume change. However, it is not the case for the real world. Pang (1999) and Ng and Pang (1999, 2000a) reported their research work on the stress dependency of SWCCs (see Fig. 2.4). In this study, in order to measure stress dependent SWCCs, a conventional triaxial apparatus has been modified into a stress-controllable device, through which isotropic external stresses can be applied onto the soil sample and volume change can be recorded. Therefore, it is expected that the measurements obtained through test can assist better understanding of the soil behavior and provide more suitable input parameters for the subsequent seepage and slope stability analyses.

3.6.2 Description of apparatus

Totally, there are mainly three apparatuses used in the study, namely, conventional volumetric pressure plate extractor, modified 1-dimensional (1D) volumetric pressure plate extractor and newly modified triaxial apparatus for measuring SWCC under isotropic (ISO) stress condition.

1. Conventional volumetric pressure plate extractor (CAT. No. 1250, Soil Moisture): SWCC is obtained through a pressure plate using a pressure plate extractor shown in Fig. 3.4. The difference between the pore-air and pore-water pressures is called matric suction $S_w (u_a - u_w)$, which is one of the stress state variables for unsaturated soils. The matric suction can be applied on the soil specimen by controlling the difference in the pore-air and pore-water pressures with both being positive. In the laboratory, the pore-water pressure is usually controlled at an atmospheric pressure while the pore-air pressure is varied depending on the specified matric suction value. The main component of the pressure plate is the high air-entry value ceramic disk that remains saturated for matric suction applications below the air entry value of the disk. The disk is always saturated and in contact with
water in a compartment below the disk. The water in the compartment is connected to the atmosphere so that the water pressure is equal to the atmospheric pressure (gage pressure). A good contact between the specimen and the disk results in the pore-water pressure in the soil being controlled at the same pressure as the water pressure in the compartment. The diffused air going through the disk can be driven out from the tube by rolling before taking every reading.

2. **1D pressure plate extractor:** The schematic diagram of the setup is illustrated in Fig. 3.5. The modified part of the new apparatus is the loading ram and the dial-gage installed on the top cap of the chamber. Through the loading ram, a certain value of external vertical stress could be applied to the top of the sample. The load is controlled by the reading from the load cell on the ram. The weights are added on to the loading frame. The other components and attachments are similar to conventional volumetric pressure plate extractor. For more details, please refer to Pang (1999).

3. **Modified triaxial apparatus with ISO stress conditions:**

   The schematic diagram of the newly modified triaxial apparatus is illustrated as Fig. 3.6 (a). The major feature can be described as follows:

   (a) The soil sample is sitting on the basement with a high air entry disk. The basement and the sample in the middle and the top cap are all wrapped by the cylindrical membrane, assuming that water cannot go into the sample during the test.

   (b) Beneath the ceramic, the surface of the base is engraved with a spiral groove. The water compartment can store some water and it can flow out through the two small apertures connected to the water outlets. The groove is for driving out accumulated air bubbles. Ideally, all the air bubbles should be driven out
by rolling the tube between the two outlets. Driving out all the bubbles is one of the requirements to get reliable reading for a given matric suction.

(c) The cell is filled with water and cell pressure $\sigma_3$, i.e. the confining pressure acting on the soil, is controlled by an air-pressure source, which is converted into water pressure after passing through the air-water interface.

(d) The air pressure $u_a$ is directly applied to the top of the sample through the top cap. Same as the conventional pressure plate, the water pressure $u_w$ is equal to the atmospheric pressure.

(e) The volume change of the sample can be measured through the volume-change, by measuring the volume of inflow/outflow of water and assuming the cell deformation can be corrected by calibration.

(f) Other attachments are the same with the conventional one.

(g) During the whole process of the test, the net normal stress $\sigma_3 - u_a$ is always kept constant, by controlling $\sigma_3$ and $u_a$ simultaneously.

The real setup in the laboratory is shown in Fig. 3.6(b).

Apart from the above three apparatuses, 5-Bar pressure plate extractor (CAT. NO. 1600, Soil Moisture, Fig. 3.7) and 15-Bar pressure membrane extractor (CAT. NO. 1000, Soil Moisture, Fig. 3.8) were also used to measure SWCCs with higher suctions. The working principle is the same as the volumetric pressure plate extractor: the matric suction is controlled by applying a positive air pressure and maintaining pore-water pressure equal to atmospheric pressure through the saturated ceramic disk (or cellulose membrane) at the base. The difference is that a number of duplicated samples are required for completing a curve, because no other attachments are used, such as air-trap and burette. When the equilibrium is achieved, one sample is taken
out to weigh the mass. By comparing the initial weight and assuming the initial 100% saturation, the water content under a certain value of matric suction can be calculated from the initial condition. Another way of calculation is to oven-dry every sample after its weight is recorded. Comparison of the results from these two different ways can ensure the reliability of the experiments, if the difference between them is small. However, some of the samples were destroyed after being weighed thus could not be dried for back-calculation. Only a few samples were oven-dried and could be used for back-calculation. Therefore, in order to be consistent, data presented hereinafter were calculated from the initial state since their validity was proved by some back-calculations.

Only drying curves can be measured in the 5-Bar pressure plate extractor and the 15-Bar pressure membrane extractor, whereas the rest types are able to measure SWCCs of wet-dry cycles.

3.6.2.1 Pre-test examination on ceramic disks

A valid air-entry value of the ceramic disk is the most important requirement for obtaining reliable test results. However, it is not always ensured for all products purchased. Therefore, before making use of these commercial products, their air-entry values were checked by applying a certain value exceeding their capacity, i.e. air-entry values. It was found that, in fact, each apparatus, including pressure plate/membrane extractors and individually purchased ceramic disks, used in the study has a slightly higher air-entry value with respect to the air-entry value provided by the manufacturer. For 2-Bar volumetric pressure plate extractor, the air-entry value is slightly above 220 kPa. The 5-Bar pressure plate extractor has an air-entry value of about 540 kPa. Those ceramic disks individually installed onto the modified triaxial
apparatus were found to have an air-entry value of 550 kPa, higher than 500 kPa. The 15-Bar membrane does have 1500 kPa for the air-entry value, however, some cracks form and its performance deteriorates greatly under high suctions when it is in a relative dry state. So normally, the results beyond 1000 kPa may involve some error.

3.6.3 Testing procedure

3.6.3.1 Sample preparation

Both intact and recompacted soil samples were used in the laboratory SWCC tests. Intact samples help investigate the characteristics of the soil sample in its natural state, whereas recompacted samples are also necessary and important. Recompacted samples can easily be prepared into any desired state, with the moisture content and dry density controlled. Moreover, duplicated samples can also be produced, however, identical samples are hardly available from natural samples due to great variation in soil properties with respect to space.

Intact sample preparation

a) For natural soil sample, a special cutting ring, which is made up of stainless steel, is used to trim the sample from a block of intact soil. The trimmed soil sample should have a good contact with the internal side of the ring.

b) Trim the sample carefully and make the two end surface as smooth as possible. There should be no crevice between the soil and the interior wall of the cutting ring.

c) Weigh the sample with the cutting ring and record the mass of both the sample and the ring.
Re-compacted sample preparation

d) For already disturbed soil, the soil is dried in an oven of a temperature about 40°C to 50°C and ground in a mortar with a pestle.

e) All the disturbed sample used for testing should be allowed to pass through a test sieve of 2mm-diameter aperture openings before it is wetted and compacted.

f) After dry sieving, the soil is mixed with a certain amount of de-aerated water so that the soil has a particular value of moisture content. (The prepared moisture contents are determined by the purpose of the test and the soil category.) The sample should be mixed thoroughly by a spatula until no big soil clots or dry soil particles can be seen. In this study, the moisture content at compaction is equal to the natural water content (30.3 %). The compaction curve of ZY (obtained from the geotechnical laboratory of YRSRI, Wuhan) is shown in Appendix II. It can be seen that the soil is compacted at wet of optimum. The solid curve represents the standard compaction effort curve. The dot line is postulated, which represents the compaction curve of certain compaction effort used in this study.

g) The soil-water mixture is left statically in a sealed plastics bag for 24 hours to equalize the moisture of the soil. After that, a small amount of soil is taken out and the real moisture content of the prepared sample is measured in an oven at 105°C. The measured water content is used for compaction. During the period, the soil is again sealed in the bag for 24 more hours.

h) According to the dry density required by a given test, a fixed mass of wet soil is weighed according to its water content measured and compacted into a stainless steel ring to make a desired sample with a specific water content and a specific dry density. Totally there are two types of steel ring that have been used, hence two different ways of compaction are adopted. One type is the oedometer ring, which
is 7 cm in diameter and 2 cm in height, and the other type is self-manufactured ring that is 5 cm in diameter and 1 cm in height. The samples compacted in oedometer ring are used for 2-Bar volumetric pressure plate extractor and the modified triaxial apparatus. The samples compacted in self-manufactured ring are used in 5-Bar pressure plate extractor and 15-Bar pressure membrane extractor, because this type of sample is smaller in size and more suitable for these two types of apparatus, where a number of samples are needed for completing a curve. For the oedometer ring, the soil was compacted statically in oedometer, the settlement of the soil during compaction could be observed from the dial gage on the top cap to make sure that the sample volume is exactly the same with the ring. The maximum applied weight was about 5-10 kg. Therefore, the maximum mean net normal stress \((p-u_0)\) a sample experienced in compaction was about 50-100 kPa. For the self-made stainless steel ring, when the ring is fixed on the table stably, a steel weight with a hole in its center is allowed to fall down from a certain height along a steel rod to compact the sample underneath the bottom of the rod. Ideally, the compaction effort is the same for every sample to achieve identical soil samples but the maximum stress can by no means be measured. However it might be roughly estimated by inferring from the calculated value on a sample compacted in oedometer. Since soil samples were compacted (30.3%, wet of optimum) into the same density (1370 kg/m\(^3\)), the maximum value may be estimated to be about 100 kPa.

i) Weigh each sample with its ring and record the value.

Saturation of the samples
j) The saturation processes for intact and recompacted samples are the same. Swelling after compaction should be completely constrained. Each specimen was sandwiched between two coarse porous stones along with two pieces of filter paper. All the specimens and porous stones were fixed with two steel plates by two clamping rods (Fig. 3.9). Provided that all coarse porous stones and steel plates were rigid, all the samples could be assumed to have no swell during this procedure.

k) All prepared samples contained in steel rings along with the constraining device were then placed into a transparent plastic desiccator in which a vacuum of about 90 kPa is applied. 6 hours later, de-aerated water is added to the desiccator under vacuum until all the specimens are immersed in water. The setup is shown as Fig. 3.10. The applied vacuum was maintained for 12 more hours. Release the vacuum and open the desiccator. While under water, the specimens were left open to the atmosphere for 6 more hours. The process of saturation was thus completed.

l) Weigh each sample with its containing ring and record the readings from the balance.

m) From calculation, it was found that the degree of saturation of some samples could be up to nearly 100%, while some others can only have a degree of saturation at about 90-95%. The specimens are now ready for testing.

Expansive soil swells significantly upon wetting, therefore, to maintain a desired dry density, swell during the process of saturation of all soil samples in the study should completely constrained. A comparison between two samples was presented by Ng et al. (2000). One of the two samples was restrained from swelling during saturation, the other one was free to swell during saturation. It was shown that the two dif-
ferent saturation methods, restrained and unrestrained, caused different initial water contents. The unrestrained sample had a higher water content, in terms of gravimetric water content. It was due to a higher value of void ratio in the unconstrained sample due to swelling during saturation.

3.6.3.2 Testing on conventional volumetric pressure plate extractor

a) Saturate the base plate made up of a high air-entry value ceramic disk by de-aerated water before tests start. Cover the whole plate with a certain amount of de-aired water at the same time.

b) Assemble all the components of the equipment and apply a small air pressure in the airtight chamber to force water going through the plate in order to saturate the ceramic disk. The time needed is about 30 min to get full saturation.

c) After the saturation completed, remove the excessive water on the plate carefully, meanwhile, make sure the ceramic fully saturated.

d) Load the prepared specimens onto the base plate. Ensure the good contact between the bottom surface of the specimen and the ceramic disk, in order to have an air pressure differential in the top surface and the base surface of the sample. So that the water inside the sample can drain out under a certain positive air pressure.

e) Add a little amount of water into the burette and the air trap to make the water circulation device filled with de-aerated water.

f) Run a steel roller over the connecting rubber tube to force out air bubbles which accumulate in the water compartment underneath the ceramic disk. Air bubbles are collected in the air trap after rolling. Sometimes, some unexpected air bubbles are seen in the bottom of the burette and can be removed by applying a small vacuum at the top of the burette.
g) Properly assemble all the components of the device and make sure the chamber is airtight. Run the roller again to ensure the whole circulation without trapped air bubbles.

h) Running a test for hysteresis studies, install the heater block on the top of the cap of the chamber to prevent warm moisture accumulated on the inside of the relatively cool chamber wall.

i) Open the stopcock of the burette and let a little amount of water go down to the ballast tube. Close the stopcock.

j) Open the stopcock of the air trap and elevate the ballast tube to force the water to fill the air trap up to the mark. Close the stopcock.

k) Open the burette stopcock again and let the water to fulfill the ballast tube. Until the level mark is reached, close the stopcock.

l) Record the water level according to the reading on the burette. The water level is the reference for the calculation of the whole test.

m) Apply a small vacuum at the top of the burette and open the stopcock at the bottom. Let the water in the ballast tube rise to the burette. Close the stopcock when a little amount of water is left in the ballast.

n) Open the air pressure valve carefully to maintain a steady but small pressure on the specimen.

o) Equilibrium is assumed when the length difference of water inside ballast is less than 2 mm between every 24 hr. It is less than 0.02% change in moisture content of the soil sample. As the equilibrium is achieved, adjust the water inside the system to the level marks, both the air trap’s and the ballast’s and remove diffused air bubbles through rolling or flushing the tubes. Make sure the whole system is free of air bubble and then record the water level in the burette.
p) Repeat step (l)-(n) until the desired maximum suction is reached. Normally, the maximum suction is the same with the air-entry value of the underlying ceramic disc.

q) Then start the wetting path (sorption path) by decreasing air pressure applied. Repeat operation steps that are same with (l)-(n) until zero or very close to zero air-pressure. In the tests of this thesis, the minimum value of air-pressure is 1 kPa, due to the pressure existing in the whole system.

r) Terminate one wet-dry cycle. Remove the soil sample from the pressure plate. If necessary, keep the sample properly in an air-tight plastic bag for the future SEM or MIP tests, otherwise, oven-dry the sample at 105°C for 24 hr and calculate the moisture content at the end of the wetting path.

s) Back calculate the complete SWCC and compare the result with the calculation from the initial state.

3.6.3.3 Testing on 1D volumetric pressure plate extractor

All steps are the same with the conventional pressure plate extractor except in step (f), when the top cap is loaded, the semisphere-shaped tip of the loading ram is also connected to the loading cap of oedometer which is on the top of the sample. The dial gage is installed on the top of the cap, so the settlement of the soil can be read from the gage. The volume change of the sample can be calculated from the settlement, assuming the oedometer ring is rigid without any lateral deformation. The load was applied by a loading frame with a number of weights. Through the load cell, a desired value of pressure is adjusted and applied onto the soil and the soil starts to consolidate immediately. Matric suction is not applied until the consolidation process is completed. Taking a reading when equilibrium is achieved, at the same time, record
the reading of the dial gage. For details of the process and setup of 1D volumetric
pressure plate tests on SWCCs, please refer to Pang (1999).

3.6.3.4 Triaxial Apparatus for measuring SWCCs under isotropic condition.

Saturation of the ceramic disk:
(a) Saturate the high air-entry value ceramic disk installed on the base by de-aerated
water before tests start. Fix the base onto the base plate of the cell.
(b) Set up the cell and make all the tubes connected correctly, including all accesso-
ries. Place the tube connected from the air pressure valve to the top cap onto the
inner side of the wall of the cell with a sticker in a high enough position.
(c) Make sure the cell pressure valve, the air pressure valve and two water outlets are
all closed at the beginning.
(d) Open the cell pressure valve and then de-aerated water from the water source
flows into the cell through tubes passing through air-water interface and volume
change until the whole ceramic disk is immersed by water. Close the cell pressure
valve.
(e) Modulate the regulator to attain a certain value of air pressure. Open the air pres-
sure valve and so that a positive air pressure is applied on the water surface in the
cell. Open one of the water outlets.
(f) Water inside the cell starts to flow through the disk to the water compartment and
then flow to the opened water outlet.
(g) After some time, open another water outlet and roll the tube connecting the two
outlets to drive accumulated air bubbles out.
(h) At least 2 hours are required for saturating the disk, since the permeability of the high air-entry value ceramic disk is low. After saturation is completed, open the cell pressure and increase the air pressure, so that the water left in the cell can flow out. Close the air pressure valve and release the air pressure inside the cell. Remove the excessive water on the plate carefully before mounting the sample.

Sample mounting

(i) Remove the oedometer ring from the prepared specimen carefully without much disturbance and soil mass loss and then load the sample onto the ceramic base. Ensure the good contact between the bottom surface of the specimen and the ceramic disk.

(j) Add the metal cap onto the sample.

(k) With the help of a vacuum and a cylinder, a membrane is loaded to wrap the base, the sample in the middle and the cap on the top. Use 4 O-rings to keep the sample inside the membrane watertight.

(l) Set up the cell and close all the valves and outlets. Keep the small hole on the top plate of the cell open, so that later water in a container at a certain height can flow into the cell under the gravity.

(m) Set the volume change in the bypass state. Open the valve of cell pressure and other related valves connecting with the water source. Water starts to flow into the cell.

(n) Until the water overflows from the hole in the top cap of the cell, the cell is entirely filled with water, close the cell pressure valve and the hole on the top plate. In this process, make sure there are not many air bubbles sticking to the cell wall and the top plate. It is important for the accurate measurement of volume change
in the later stage. To reduce the total volume of the air-bubbles accumulated at the
top, slow down the water flow rate to fill the cell or shake/move the cell slightly to
make the air bubbles flushed out by overflowing water.

(o) Adjust the dial gage of the volume change into a certain clear and simple reading
and set the valve in the flow-up state.

(p) Open the stopcock of the burette and let a little water flow down to the ballast tube.
Close the stopcock of the burette. Open the stopcock of the air trap and elevate the
ballast tube to force the water to fill the air trap up to the mark. Close the stopcock.
Open the burette stopcock again and let the water to fulfill the ballast tube. Until
the level mark is reached, close the stopcock. Record the water level according to
the reading on the burette. Apply a small vacuum at the top of the burette and
open the stopcock at the bottom and the water in the ballast tube rises to the bu-
rette. Close the stopcock when a little amount water left in the ballast (This step is
the same as the steps (i)-(m) described for testing on a conventional volumetric
pressure plate extractor).

Consolidation:

(q) Close all the valves and all the water outlets. Adjust the regulator controlling cell
pressure on the panel board to the desired net-normal stress. Then open the cell
pressure valve so that the sample starts to consolidate under certain stress. Open
the water outlet connected with the air-trap.

(r) Terminate the consolidation stage when no more change of the readings of the dial
gauge and the water volume in the ballast. Normally, 2 days are adequate for
completing the consolidation process. Open the other water outlet to remove dif-
fused air bubbles through rolling or flushing the tubes. As the equilibrium is
achieved, adjusted the water inside the system to the level marks, both the air
trap’s and the ballast’s. Make sure the whole system is free of air bubbles and then
record the water level in the burette. Record the reading in the dial gage.

(s) Apply a small vacuum from the top of the burette and open the stopcock on the
burette, in order to reduce the water amount in the ballast. Leave only a small
amount of water in the ballast. Then close the stopcock of the burette. Now the
soil is ready for applying matric suction.

Desorption/sorption

(t) Close all the valves and water outlets in the cell. Adjust regulators of cell pressure
and air pressure. When the new desired pressures, both cell pressure and air pres-
sure, are reached, open the cell and air pressure valves.

(u) When the length difference of the water inside the ballast is less than 2 mm be-
tween every 24 hr, the equilibrium state for the present suction value is reached. It
is equal to a change less than 0.01% in moisture content of the soil sample. As the
equilibrium is achieved, adjust the water inside the system to the level marks, both
the air-trap’s and the ballast’s. Remove diffused air bubbles through rolling or
flushing the tubes. Make sure the whole system is free of air bubble and then re-
cord the water level in the burette and the reading in the dial gage.

(v) Repeat step (t)-(u) for a new matric suction value, until the maximum suction is
reached. Normally, the maximum suction is the same with the air-entry value of
the underlying ceramic disk.

(w) Then start the wetting path (sorption path) by decreasing air pressure applied. Re-
peat operation steps are same with (t)-(u) until zero or very close to zero air-
pressure. As stated before, the minimum value is 1 kPa in this study.
(x) Terminate the test of one wet-dry cycle (desorption-sorption curves).

(y) Remove the sample from the pressure plate extractor and weigh the sample mass. Dry a part of the wet sample in a 105°C oven and keep the rest in an air-tight plastic bag for the following SEM and MIP tests, if necessary. (The dry weight can be used to back-calculate the curve to check with the results calculated from the assumed initial saturated state.)

Calculations:

According to the recorded changing values in the burette, the volume of drained water can be computed and in turn the change of water content in the soil sample can be obtained. One common way to calculate to SWCC curve is to weigh the soil sample after the test termination and to do a back calculation. Another way is to calculate the degree of saturation or volumetric water content is then obtained assuming the soil specimen in the initial state is fully saturated. These two methods can be used concurrently to check the reliability of the test results.

3.6.3.5 Pressure membrane extractor (15-Bar) for measuring SWCCs

(a) Take one piece of membrane and immerse it flatly under de-aerated water. According to the instruction given by the manufacturer, half an hour is enough to saturate the membrane.

(b) Take out the membrane and spread it on the base plate. Use fingers to drive air bubbles trapped under the membrane carefully. Then lay the cylinder o-ring seal on the cellulose membrane disc and set the extractor cylinder on the top of the o-ring.
(c) Place all saturated samples onto the membrane.

(d) Place the second o-ring seal in the top groove of the cylinder. Close the extractor and set the top plate on the extractor cylinder so that the top and bottom bolt slots line up. Insert the clamping bolts and tighten.

(e) Place a small glass container with a small opening right below the outflow tube. Weigh the bottle with water periodically. The reading ceases increasing when the soil samples approach the equilibrium state for the current extraction pressure.

(f) Release the pressure applied. Open the pressure regulators slowly and adjust the pressure to be zero.

(g) Take out one sample and weigh the sample mass with the containing ring. Record the reading. Normally, 5-8 days is adequate to reach the equilibrium under a certain value of matric suction.

(h) Dry the sample in an oven at 105°C to get the dry weight and then dry density and the water content.

(i) Close the extractor again according to the instruction of step (d) and then repeat (f)-(h) until the sample corresponding to the maximum pressure is measured.

3.6.3.6 Pressure plate extractor (5-Bar)

(a) Saturate the high air entry value disk by adding some water on the surface of the disk. Close the extractor by inserting and tightening all the bolts. Carefully adjust the pressure regulator and apply a certain positive pressure in the extractor to saturate the ceramic disk until no more air bubbles flushed out from the water compartment.

(b) Open the extractor and remove excessive water from the surface of the disk. Place all the saturated samples in their respective containing rings on the ceramic disk.
(c) Set the top plate on the cell and close the extractor. Open the pressure regulator and adjust the air pressure to the desired extraction pressure. Water starts to flow out from the outlet.

(d) Connect a small opening glass bottle is connected to the water outlet. Weigh the bottle regularly, say, once every 4 hr or once a day. When there is no more change in the weight of the bottle containing water, assume that the equilibrium state is reached.

(e) When an equilibrium state is reached, release the pressure.

(f) Open the extractor and take out one sample to be weighed and then dried in the oven at 105°C. The water content is thus directly measured from the oven-dried sample.

(g) Repeat (c)-(f) until the sample for the maximum pressure is measured. As stated before, the maximum value of the applied pressure in this thesis is the air-entry value of the ceramic disk.

Calculation for 5-Bar pressure plate extractor and 15-Bar pressure membrane extractor

Water retention curve can be calculated either from the strictly controlled initial state or back-calculated from the measured water content from oven-dried samples. In this study, presented data were calculated from the initial water content, either because the sample number was limited or because some soil samples were damaged or broken under the suction, especially those natural samples. The water contents of some measured samples were compared with the values calculated from the initially controlled water content and it was found that the difference was not significant and could therefore be ignored. Hence, in this thesis, most data presented were calculated
from the controlled water content at the beginning of the sample preparation. The probable error of a SWCC is most likely to be a shift in position but not the shape of the measured curve.

3.6.4 Measured soil-water characteristic curves of LZ and ZY samples

Expansive soils are sensitive to volume change due to any change in water content, thus it was suggested by Chao et al. (1998) that SWCC of expansive soils expressed in gravimetric water content (GWC) was more fundamental. And also because GWC for all samples tested in different apparatus are truly measured and can therefore be compared without any assumption, SWCCs expressed with GWC are presented firstly. However, since the measured SWCCs of this thesis are applied into transient seepage analysis, the prediction of water permeability from SWCCs requires that SWCCs be expressed with volumetric water content (VWC) in later study (Fredlund et al., 1994). In this thesis, SWCCs expressed with GWC is shown first. Then SWCCs in terms of VWC calculated based on measured GWC and volume change are also shown. For those tests conducted in conventional volumetric pressure plate, volume change is assumed to be zero, although it is certain that there must be some volume change due to the change of water content in soil. Variables associated with SWCC, such as air-entry value and desorption rate (desaturation rate) are estimated roughly by visual observation according to Fredlund and Xing (1994). Residual water content cannot be seen because the maximum suction applied is not high enough for these expansive soils of high plasticity.

Measured SWCCs were input into SoilVision (SoilVision, 1997) to get SWCCs in the full range of matric suction from 0.01 to 1000, 000 kPa. Best-fit curves as well as predictions based on grain size distributions are presented. Therefore, the air-entry value, desorption/sorption rate and the residual water content can be seen more easily.
All the soil samples tested in the thesis are listed in Table 3.4. As for the IDs of all the samples, ‘ZY’ and ‘LZ’ are the names for the soil origins and the letters of ‘N’ and ‘R’ represent natural and recompacted samples, respectively. The initial water contents, dry densities and testing apparatus are all shown in the table. The void ratio after the process of saturation is assumed to be the same as that before saturation, because all the samples are restricted from swelling by the steel plates with clamping rods. ID or ISO in the brackets represents the stress condition, either 1-dimensional in the vertical direction or isotropic stress. The value with the unit of ‘kPa’ stands for the stress condition of the sample during the test. For instance, 0 kPa means the result is from one conventional pressure plate, while 1D-50 kPa shows the sample is tested under 1D (vertical) stress of 50 kPa.

3.6.4.1 General description on soil-water characteristics for Nanyang expansive soils

The results of drying path for four series of natural ZY samples obtained from different apparatus (refer to Table 3.4) are shown in Fig. 3.11. The water retention curves in Fig. 3.11 (a) and (b) are expressed in term of GWC and VWC, respectively. VWC is obtained by multiplying gravimetric water content with dry density, assuming zero volume change, since no volume change can be measured in those conventional apparatus for SWCCs. The different initial water contents shown in the figure may be due to different compaction methods, corresponding to different sample dimensions (refer to 3.6.3.1). Generally, these apparatus surely have reliable performance and all samples are reasonably identical, since all the series of data look as a smooth curve as a whole. It can be roughly estimated that the air-entry value for natu-
eral sample is about 40 kPa. The residual water content cannot be seen. By using Soil-Vision, a fitted curve can be obtained based on some empirical equation for SWCC. In this study, Fredlund & Xing’s equation was adopted (Fredlund and Xing, 1994). The fitted curve and the predicted SWCC based on the grain size distribution are shown in Fig. 3.11 (c), along with the measured soil water characteristic data (all the input and output parameters are summarized in Appendix III). It should be noted that the predicted SWCC is the same for all the ZY samples in this thesis, since it is predicted from the grain size distribution. The prediction gives an air-entry value of about 100 kPa and the residual water content is less than 6 % with a corresponding matric suction of 120000 kPa. From the fitted curve, it can be seen that the air-entry value is also 40 kPa. Moreover, the residual water content and its corresponding suction can be approximately estimated from the visual inspection of the figure and found to be about 4 % and 300000 kPa. It is clearly shown that the fitted curve agrees well with the measured data and these two curves are close to each other in the whole suction range. Therefore, it can be stated that the four series of experimental data do reflect the soil-water characteristics of natural expansive soil ZY. The prediction from the grain size distribution gives a good illustration of the workability of the method, at least for this particular clay.

Similarly, drying curves for several recompacted ZY samples are shown in Fig. 3.12. From Fig. 3.12(a)&(b), the air-entry value is estimated to be 60 kPa. As stated before, the difference between Fig. 3.12(a) and (b) is the scale that is equal to the dry density. Again, the point of the residual water content cannot be observed. The fitted curve and the prediction from the grain sizes are also shown in Fig. 3.12 (c). They are very close to each other. It can be estimated from the fitted curve that the air-entry
value and the matric suction corresponding to the residual water content are 60 kPa and 200000 kPa, respectively, and the residual water content is about 4%. 

The soil-water characteristics of LZ-R1 from 15-Bar pressure membrane extractor are shown in Fig. 3.13. The fitted curve as well as the prediction from the grain size distribution of LZ is also shown. The curves agree well with the test. The estimated air-entry value from the fitted curve is about 150 kPa, whereas the prediction shows a value of 350 kPa. The value of LZ sample is obviously greater than that of ZY. Considering the grain size distributions of these two samples, LZ has a slightly greater content of clayey particles than ZY, although they can still be viewed as very similar soils. It is known that a soil with a high percentage of fine particles usually has a high air-entry value for its SWCC. Moreover, the ZY samples were compacted to 1370 kg/m³ whereas the LZ sample had a dry density as high as 1600 kg/m³. Referring to Pang (1999), it can be found that the air-entry value increases with increasing soil dry densities for recompacted soil samples. Therefore, the clear difference of air-entry values can be mainly attributed to the significant difference in dry density of these two samples after recompaction, apart from the slight difference of the percentages of fine particles for the two samples. In Fig. 3.13, based on the fitted and predicted curves, the residual water content and the suction of the state are about 5% and 200000 kPa. The desorption rate of LZ is similar to that of ZY, as seen from the above fitted curves, which may imply that they generally have a similar pore size distribution in the matric suction ranging from 200 to 1000 kPa.

The drying path of SWCC in terms of GWC of a completely decomposed volcanic (CDV) at Peak Road, Hong Kong is also shown in Fig. 3.14(a) for comparison. It should be noted that it is a recompacted sample with a very low dry density of 1078 kg/m³. As shown in Fig. 3.14(a), this loose material starts to desaturate significantly
from the very beginning and the desorption rate is much higher under matric suction of 1-50 kPa than that of either natural or recompacted ZY sample. Thus, it possesses an air-entry of nearly zero, estimated from the measured data point. It resulted from the loose and open structure of the soil sample. Both the fitted curve and the prediction along with the measured soil-water characteristics in terms of VWC are also shown in Fig. 3.14(b). In this figure, the prediction from the grain size distribution does not agree well with the fitted curve. It can be seen that the air-entry value is about 2 kPa and 20 kPa in the fitted curve and the predicted curve, respectively. Therefore, it can be said that the air-entry value of the Peakroad CDV is quite low, compared with the values presented above for expansive soil. Moreover, the air-entry value of another CDV was about 2-20 kPa (Pang, 1999). Because most expansive soils are clays of high plasticity, high air-entry value and low desorption rate are both major characteristics of expansive soils (Fredlund et al., 1995). As reported by Chao et al. (1998), the air-entry value of the tested expansive soil samples was higher than 100 kPa. Therefore, it can be seen that the air-entry value of expansive soils is usually high and the desorption rate is relatively low, compared with non-expansive soils.

For the fitted curve, the residual water content is about 10 % and the matric suction of the state is only 1000 kPa, which is much lower than that of each expansive soil sample. For the predicted curve, the residual water content is also 10 %, but the corresponding matric suction is 20000 kPa, higher than that of the fitted curve. The relatively significant difference between the fitted and predicted curves. The predicted curve has a much higher air-entry value of 20 kPa, which leads to a generally higher water retention curve (Fig. 3.14(b)). This can be attributed to the fact that the prediction does not take the low density well into account and therefore overestimates the water retention ability of the loose soil sample.
It should be pointed out that no volume change has been taken into account in all the curves shown above. It can be seen that in terms of gravimetric water content SWCCs have a similar shape when expressed in volumetric water content. The difference between these two is the scaling factor, i.e. dry density. Therefore, only SWCCs are plotted in gravimetric water content hereinafter, when volume change is not available. In fact, there is no significant shrinkage occurring in expansive soil samples until the matric suction exceeds 700 kPa. Hence, only the results of 15-Bar pressure membrane extractor involve some errors in the suction range of 700-1500 kPa, which is only a small portion of a complete curve. As for the Peakroad CDV, there is some reduction in volume after saturation when the matric suction is low, it is not due to shrinkage but due to some change in the soil fabric. Because the density is too low and the CDV is of low plasticity, the loose particle arrangement of the soil tends to collapse when it is saturated. Since the volume decreases, there is more water expelled out when the suction is low, which results in a low (nearly zero) air-entry value in the measured SWCC. That can explain the clear difference between the fitted and predicted curves, because the prediction is only based on the particle size distribution but does not take into account the reduction of the large void ratio.

3.6.4.2 Difference between natural and recompacted samples

ZY-N1 and ZY-R1 were tested in a 5-Bar pressure plate extractor and a number of duplicated samples were used to obtain a complete curve to represent the SWCC of a particular soil sample and the results are shown in Fig. 3.15. They possessed a same initial water content and density, referring to Table 3.4. It can be seen that the two samples show different characteristics regarding water retention ability. First of all, the natural sample, ZY-N1, starts to desaturate as soon as the air pressure is ap-
plied on the soil whereas the recompressed sample sucks in a little water before water starts to be expelled out significantly from the matric suction of 35 kPa. It is probably due to the initial unsaturated condition of the sample. The air-entry values for natural and recompressed samples appear to be about 50 and 100 kPa, respectively. The desorption rate of the natural sample is greater than that of the recompressed sample from 0 to 35 kPa. This can be attributed to the fact that the natural sample has some natural fissures and relatively larger open pores whereas the recompressed sample does not have these large and open drainage paths. Since the samples tested may not be fully saturated, it is thus understandable that ZY-R1 can even suck in water from the ceramic disk when the suction is very low, e.g. 1 kPa. Starting from 35 kPa, two samples desaturate in the same rate because the two curves are nearly parallel from 35 to 500 kPa in Fig. 3.15. It should be noted that there is no visible shrinkage in the test occurring upon drying of both samples up to the maximum matric suction of 500 kPa, based on the author’s visual inspection and measurement with a caliper.

From the discussion above, it can be seen that compared with a recompressed sample, an intact expansive soil sample usually possesses a lower air-entry value and a higher desorption rate for the matric suction lower than its air-entry value (40 kPa). The air-entry value for the recompressed sample is estimated to be about 60 kPa. This can probably be due to existing natural fissures and large pores, although the dry densities are the same for the two types of sample. Two samples can have different pore size distributions but the same dry density. However, these two types of samples desaturate in the same desorption rate for the matric suction beyond their air-entry values. Therefore, a recompressed sample generally has a higher water retention ability.
than a natural sample. Compared with results reported by Pang (1999) and Ng and Pang (2000b), the above conclusions are consistent with the laboratory measurements on both natural and recompacted CDV samples. Reported by Pang (1999), the air-entry value for the recompacted CDV sample is about 5 kPa, while the natural sample has an air-entry value of 1 kPa. The desaturation rate is nevertheless the same under the matric suction beyond 5 kPa.

3.6.4.3 Hysteresis

Natural soils experience continuous wet-dry cycles in field. One natural ZY sample, ZY-N3, was tested in a 2-Bar volumetric pressure plate extractor. Thus, no volume change could be measured and constant volume was assumed. Three wet-dry cycles of a ZY intact sample are shown in Fig. 3.16. All three cycles exhibit different relationships of water content vs. matric suction. It can be seen that the loop of the first cycle is the greatest in size among the three, which looks to be the envelope for the subsequent loops. At the end of the second cycle, the water content goes above the starting point of the second drying path, which is rarely reported. According to the existing literature, the water content at saturation of a drying curve is greater than that of a wetting curve for a wet-dry cycle (Fredlund and Rahardjo, 1993). It should be noted that surprisingly, the third wetting path lies above the third drying curve, which appears to be a ‘reversed’ hysteresis loop, which has rarely been seen before (Top and Miller, 1966; Hillel, 1998; Ng et al, 2000; Pang, 1999). The result from Pang (1999) is shown in Fig. 3.17. The first cycle of the CDV looks the greatest in size and the desorption curve of the first cycle are the highest in position in this figure, which is consistent with the result shown in Fig. 3.16.
According to Hillel (1998), hysteresis can be mainly attributed to ink-bottle effect, contact angle effect, entrapped air and swelling, shrinking or aging phenomena. The first hysteresis loop can be mainly attributed to the ink-bottle effect. Starting from the second cycle, a smaller loop may be caused by the previous wet-dry cycle, which may cause some change in soil structure. After experiencing two wet-dry cycles, the structure of the soil sample may be changed. According to Lin and Benson (2000) studying on effect of wet-dry cycling on some geosynthetic clay liner (GCL), the permeability of samples permeated with a solution of CaCl$_2$ increases significantly because of cracks formed during desiccation. This may also be applicable for ZY-N3, since the water used for preparation of the samples in the thesis is de-aerated only but not deionized. Thus, it is very likely that the ion of Ca$^{2+}$ exists in the water used, the solution of which (water in fact) can be attracted by the soil. The volume of soil sample therefore increases significantly. Moreover, the water may contain an amount of Mg$^{2+}$, which is easily attracted by the mineral of montmorillonite in the soil (Mitchell, 1993). As the ions enter the soil pore spaces, the soil sucks in a large amount of water and increases in its volume. This is the most common mechanism for swelling in expansive soils.

As introduced in the literature review in Chapter 2, the minerals of the montmorillonite group have a structure consisting of three-layer arrangements in which the middle octahedral layer is mainly gibbsite but with substitution of Al by Mg. As a result of this weak linkage, water molecules are easily admitted between sheets, resulting in a high shrinking/swelling potential (Whitlow, 1995). ZY expansive soil can be classified as a soil with medium to high swelling potential, as stated in detail before in this chapter. Therefore, the physicochemical interactions between the minerals and
water probably play an important role in the presented hysteretic behavior of this sample.

On the other hand, the change in soil fabric may also be influential in hysteresis. The process of structural change may be postulated and illustrated as Fig. 3.18, according to Gens and Alonso (1992) and Alonso et al. (1995). The fabric of this expansive soil is postulated to be double-structured (see Fig. 2.1-3). Macropores and micropores are interparticle (inter-aggregate) and intraparticle (intra-aggregate) pores, respectively. The original state is shown in Fig. 3.18(a) and it can be seen that there are some large void spaces, which are interparticle pores. Some small cracks along intraparticle pores may form upon drying (Fig. 3.18(b)) and soil particles around big pores are disintegrated down in rehydration and fall into the pores when the large pores are intruded by water during wetting. In the subsequent drying path, a lot of smaller pores are formed after particle rearrangement (Fig. 3.18(c)). The soil sample then possesses a pore size distribution with more small pores, compared with its original condition. Therefore, it is understandable that the soil may have higher water retention ability under a given suction in later wet-dry cycles. The postulation can be used to explain the third reversed hysteresis loop in Fig. 3.16. The first loop looks like the envelope of the three cycles, except the part of the third loop corresponding to matric suction from 70 to 200 kPa. This part with a higher water retention ability can also be explained by the postulation stated above, i.e. the wetness of a soil sample is higher with generally smaller pores under a given suction.

As seen from Fig. 3.19, the SWCC is expressed with degree of saturation (DOS) and the maximum difference in DOS between drying and wetting paths is in the first cycle and about 5% only, which is smaller than 15% compared with the hysteresis
observed in CDV (Ng et al., 2000). The relative small hysteresis loop also agrees with Hillel (1998) that hysteresis in swelling soils is not so significant as non-swelling soils.

The natural expansive soil sample exhibits hysteretic phenomenon upon wet-dry cycles but the hysteresis may look strange, since a 'reversed' hysteresis loop may appear after several wet-dry cycles. The higher water retention ability of a wetting path may indicate that there is some structural change occurring during wet-dry cycles. Some more small pores are formed. This may reflect the combined effect of the special mineralogical composition of the expansive soil and the proposed double-structure for expansive clay.

3.6.4.4 Stress effects on SWCCs of ZY recompacted samples

A conventional volumetric pressure plate extractor (2-Bar) was modified into a new apparatus that could apply 1-dimensional vertical load on the soil sample while measuring SWCC and unignorable stress effects on SWCCs of CDV were reported (Pang, 1999; Ng and Pang, 2000).

So far, all the SWCCs presented in this chapter were measured in a traditional way without any external stress applied. In the thesis, 1D pressure plate extractor (2-Bar) and newly modified triaxial apparatus (5-Bar) are used to investigate the influence of different stress conditions on SWCCs of expansive soils. As introduced previously, in both 1D and ISO apparatus, volume change can be measured. Therefore, apart from being plotted in GWC, SWCCs in terms of VWC are shown, which are calculated from GWC and measured volume change. Stress dependent SWCCs expressed with volumetric water content are used to predict hydraulic conductivity for transient seepage analysis later.
Totally, three soil samples were tested under non-zero stress condition, ZY-R5, R6 and R7. ZY-R5 was tested in the modified 1D volumetric pressure plate extractor and the vertical load was controlled to be 50 kPa. ZY-R6 and ZY-R7 were tested in two newly modified triaxial cells as introduced before, applied with isotropic stresses of 50 and 100 kPa, respectively. It should be noted that each sample experienced one dry-wet cycle under an ideally constant stress. As stated before, in triaxial apparatus the applied load (σ₃) and matric suction (uₑ-uₘₑ) were controlled independently and simultaneously, in order to attain a constant net normal stress (σ₃- uₑ). For the 1D apparatus, the weights applied are adjusted according to a given air pressure. The net normal stress is read from the load cell in the chamber. It should be noted that the swelling pressure of ZY samples of 1370 kg/m³ with 30 % gravimetric water content is around 25 kPa, when measured in conventional oedometer. Therefore, the applied external stresses are all greater than the swelling pressure. The soil cannot swell during a wetting path.

The stress state of each sample is illustrated in Fig. 3.20. Because ZY-R5 is under 1D load, with the lateral stress unknown and assumed to be K₀ condition with reference to Pang (1999). According to Chen (1988), the lateral swelling pressure eventually was the same with the vertical swelling pressure, when the sample volume remained constant. It gave a K₀ value equal to unity. The test condition is not exactly the same with the 1D stress condition in the thesis, however, Chen’s result might be also applicable for the study in the thesis. Since the swelling pressure is exceeded by the applied 50 kPa vertical stress, the soil cannot swell during wetting and the oedometer ring is without deformation. It may be assumed that in the thesis the K₀ value, i.e. the ratio of the lateral stress and the vertical stress also reaches unity when the equilibrium state is achieved. SWCC tests are time-consuming and mostly it takes 3-6
months to measure a complete curve, it is therefore reasonable to assume that the equilibrium of the $K_0$ value is already reached. ZY-R6 and R7 are both under isotropic stresses.

3.6.4.4.1 SWCCs in terms of GWC

Fig. 3.21 shows the SWCCs in terms of GWC for the four samples. ZY-R3 is the only sample that was measured in a conventional volumetric pressure plate extractor without any stress applied. Before the water inside samples is expelled by applied air pressure, ZY-R5, R6 and R7 first undergo a consolidation stage. In this stage, the samples remain saturated and the volumes decrease as water drained out. As seen from Fig. 3.21(a), after consolidation, the water content of ZY-R6 is reduced more than that of ZY-R5 and the water content of ZY-R7 in Fig. 3.21(b) is even lower than that of ZY-R6. This may imply that the higher the stress is, the lower the water content after consolidation would be due to a lower void ratio. Those large pores in the soil must have been compressed under stress.

As matric suction is applied, water starts to be expelled out in a large amount from all soil samples except ZY-R3. ZY-R3 does not lose water significantly until the matric suction reaches 10 kPa. The air-entry value can be estimated to be about 20 kPa, a little lower than the one estimated for Fig. 12, where the results of a number of samples are used. For the other three samples, air entry value cannot be defined, but will be discussed later using SoilVision when they are plotted in terms of VWC. The difference can be mainly attributed to the reduced volume of large pores in the soil samples under non-zero stresses. The relationship between ZY-R3, R5 and R6 is not clear under the matric suction from 1 to 10 kPa in Fig. 3.21 (a), so it is not discussed.
in detail. Starting from 10 kPa, pore water is expelled from ZY-R3 in a constant desorption rate in the plot. ZY-R5 and R6 desaturate in a same rate along the drying paths and the two parallel drying curves are very close to each other in Fig. 3.21 (a). It can be used to verify that the $K_0$ value is almost unity in the test. 1D of 50 kPa and ISO stress of 50 kPa are therefore essentially of no difference, except for the consolidation stage, in which they may be quite different (the consolidation stage is not long, only about 1-2 days). After consolidation, the water content of ZY-R7 is clearly lower than that of ZY-R6, which implies a lower void ratio induced in this sample by a higher external stress. It is because the reduction in water content and void ratio is due to the stress applied. The difference between ZY-R6 and R7 becomes less significant starting from the matric suction of 20 kPa. ZY-R5, R6 and R7 desaturate in a nearly constant rate from the very beginning. All the curves look nearly parallel from 10 to 200 kPa in Fig. 3.21 (a) and (b). It means the applied external stress conditions do not affect the desorption rate of the samples. In fact, it may be inferred that the sizes of smaller pores in soil have not been changed by the applied stresses. At the end of the drying path, the difference between each sample under stress is relatively small in GWC. All the drying curves including that of ZY-R3 tend to converge at 500 kPa. It implies that this type of soil samples probably have a same water content (GWC) in the given high suction regardless the stress conditions applied.

As matric suction is reduced gradually from 500 kPa, each sample starts to suck in water, but hysteresis cannot be clearly seen for these samples except for ZY-R3. Since the sorption rate of ZY-R3 is much lower than its desorption rate, the hysteresis looks quite significant. However, for ZY-R5 and R6 and R7, the sorption rates are more or less the same with their respective desorption rates, hence, hysteresis in this plot is not clear. This can be attributed to the applied stresses being able to reduce the
volume of large pores in soil, thus the pore size distributions become more uniform and the neck-bottle effect is less significant in ZY-R5, R6 and R7 tested under non-zero stresses, according to the main causes stated by Hillel (1998). At the end of sorption, each sample has a lower water content compared with the initial state. The final water content at 1 kPa of ZY-R5 is the highest, whereas the water content at 1 kPa of ZY-R7 is the lowest. Once again, it means that the saturated GWC is closely related to void ratio, since it is conceivable that ZY-R7 has a smallest void ratio due to the highest applied stress.

So far, it can be seen that the stresses acting on the soil during the measurement of SWCCs causes some reduction in those large pore volumes of the soil, therefore, the void ratio is reduced. The reduction in water content is related to the applied stress. After consolidation, the water content is lower than the initial state. However, those small pores are not affected so that the desorption rate under the suction from 20 to 500 remain more or less the same with that under zero-stress conditions. The 1D and ISO stress condition do not exhibit significant difference. It may be due to the $K_0$ value being equal to unity so that the stress conditions are indeed the same. By comparing the different ISO stresses, it is found that the soil-water characteristics under low suction are more significant, since the difference between different ISO stresses decreases with increasing matric suction. Hysteresis is clear for the sample under zero stress, whereas the loop looks very small and even not clear for those samples tested under stresses. This could be attributed to the more uniform pore size distributions caused by applied stresses. In summary, the major difference is mainly seen in the matric suction ranging from 1 to 20 kPa, which can be explained by the hypothesis that small pore spaces are left unchanged until large pores are compressed by applied stresses (Delage and Lefebvre, 1984). As a result, these samples have very close water
retention ability under matric suction ranging from 20 to 500 kPa, regardless their stress conditions.

3.6.4.4.2 SWCCs in terms of VWC

In order to calculate VWC, volume change has to be measured. The developments of volume change of ZY-R5, R6 and R7 have been recorded during the tests. The variation of the volumes of soil samples is shown in Fig. 3.22. For ZY-R5 under 1D external stress, the sample shrinks with increasing matric suction in the drying path and swells in the wetting path, though there is a little hysteresis that means the volume change is not purely elastic. However, the feature of the volume change of ZY-R6 and R7 is significantly different. It can be seen from Fig. 3.22 that the volumes of ZY-R6 and R7 decrease with increasing matric suction. After the drying path is completed and matric suction is reduced, ZY-R6 swells a little from 500 to 100 kPa and after that, the volume decreases sharply as matric suction further decreases. The volume of ZY-R7 does not increase after matric suction is reduced but keeps unchanged and decreases rapidly from 200 kPa. The sharp decrease in volume under matric suction from 50 to 0 kPa implies that the volume of expansive soil can also decrease upon wetting but not swell, under certain stress conditions. Because the swelling pressure, 25 kPa, is lower than the applied stress, either 50 or 100 kPa. Referring to Alonso et al. (1995), the soil also swelled a little and then kept constant in volume over the wetting path. Therefore, the observed decreasing volume of the soil sample can be possible. In this thesis, it is still possible that the volume decreases upon wetting, considering the low dry density of 1370 kg/m³ of the soil samples used in the tests. The loose soil fabric is easily affected by applied stresses, because those large pores (inter-aggregate pores) can be compressed significantly.
Therefore, SWCCs can also be expressed with VWC, calculated from the measured GWCS and volume changes. SWCCs of ZY-R3, R5 and R6 are shown in Fig. 3.23(a). From consolidation to the end of drying paths, the trend of these water retention curves is very similar to that described before for Fig. 3. 21(a). So this is not repeated here. Once the matric suction is released and the soils are allowed to suck in water, there is some difference between what was observed in Fig. 3. 21 (a). Since the volume change of ZY-R3 cannot be considered, the wet-dry cycle still looks normal, i.e. the water retention curve of the wetting path is lower than that of the drying path. As stated before, without volume change considered, SWCCs in terms of VWC are essentially the same with those in terms of GWC except a different scale (i.e. dry density × GWC = VWC). However, the hysteresis becomes reversed for other two samples expressed with VWC. This is because the volume reduction along wetting path increased the volumetric water contents and the measured very small hysteresis loop. From the matric suction of 10 to 500 kPa, the water retention curve of ZY-R6 is slightly higher than that of ZY-R5, either along wetting or drying. Although there is no clear relationship that can be found when SWCCs are plotted with GWC, different stress conditions do have different effects on SWCCs when volume change is taken into account. The higher water retention curve for ZY-R6 is due to the greater reduction in volume caused for the sample under that stress. The loops of ZY-R5 and R6 are nevertheless much less significant than that of ZY-R3, therefore, the wet-dry paths of ZY-R3 look like an envelope of other two samples.

When the wet-dry cycles of ZY-R3, R6 and R7 are placed in the same plot of Fig. 3.23 (b), it can be seen that the difference of SWCCs ZY-R6 and R7 is even smaller than that in Fig. 3. 21 (b). The hysteresis for ZY-R7 is reversed, like that of ZY-R6. The major difference is in the suction range of 1 to 10 kPa, however, the trend
is not very clear. It can still be inferred that some large pores get compressed in different extent under different stresses. However, from 10 to 500 kPa, these two loops look like they are overlapping. This is also possible. The only difference between these two cases is the stress level, however, they both are greater than the swelling pressure. The soil may behave very similarly under any stress condition that is greater than its swelling pressure, with the intra-aggregate pores not changed by the external stresses. Again, the loop of ZY-R3 looks to be the envelope of the other two, except for one point of ZY-R7 under 4 kPa. Comparing with Fig. 3.23(a) and (b), it may be seen that the difference brought about by different stress conditions is greater than that caused by difference stress levels of the same stress.

So far, it can be concluded that the different stresses mainly cause some difference in soil-water characteristics in terms of VWC in low suction, from 1 to 10 kPa. The difference is not significant under higher suctions, from 10 to 500 kPa, possibly due to the intra-aggregate pores not being compressed by external stresses. The difference of SWCCs indeed reflects the difference in soil fabrics. It comes to the same conclusion as in the discussion on SWCCs in terms of GWC, that the applied stresses affects those large pores in the soil but those small pores cannot be compressed. It should also be pointed out that the difference of SWCCs between 1D and ISO stresses with the same magnitude is greater than those of SWCCs between different ISO stress conditions, namely, 0, 50 and 100 kPa. This is probably due to the difference between 1D and ISO stresses before the lateral and vertical stresses come to the equilibrium state, as reported by Chen (1988). SWCCs expressed in terms of VWC are clearer than those expressed with GWC in physical meaning, which can reflect the behavior and characteristics of unsaturated soils (Fredlund and Rahardjo, 1993). Therefore, the conclusion drawn from the discussion on SWCCs of GWC may be overwritten by the
conclusion based on the discussion on SWCCs of VWC. It can be stated that different stress conditions, e.g., 1D and ISO stresses, have different influences on the SWCCs, which is indeed attributed to the different stresses acting on soil. Moreover, as for the different ISO stresses, it should also be noted that the difference in the magnitude of the same stress condition does not have a significant effect on SWCCs when the stresses are greater than the soil swelling pressure. Moreover, another stress effect is the size of the hysteresis loop being reduced while the hysteresis of expansive soil tested under the zero-stress condition is significant. The loop of the sample tested under the zero-stress condition looks like the envelope of the other samples, regardless the stress conditions.

3.6.4.4.3 Analysis of measured stress dependent SWCCs with SoilVision

Curve-fitting and predictions have also been carried out by means of SoilVision for ZY-R3, ZY-R5, R6 and R7. Since they are of the same soil type, the prediction is the same for all four samples based on the same grain size distribution of ZY. The SWCC equation is Fredlund & Xing's (Fredlund and Xing, 1994). The fitted curves and the predicted SWCCs are shown along with measured data. However, in this part, the discussion is focused on the fitted curves and prediction.

The SWCCs of ZY-R3 are shown in Fig. 3.24. As stated before, the prediction gives an air-entry value of around 100 kPa and the residual water content and its corresponding matric suction is about 6 % and 200000 kPa. For the fitted curves, the air entry value of the fitted desorption curve is about 20 kPa. The desorption and sorption rates are different up to matric suction of 200 kPa, but do not have much difference when the matric suction goes higher. The desorption and sorption curves are very close to each other under matric suction from 1000 to 1000000 kPa, therefore the hys-
teresis is not clear. The curves are straight lines when matric suction approaches 1000000 kPa. Generally, the three curves from SoilVision agree well with each other. The SWCC predicted from the grain size distribution agrees well with the desorption curve under conventional stress condition, i.e. zero stress condition. It seems that the prediction can be better applied for the desorption curve. The parameters of the curves are given in Appendix III.

The SWCCs of ZY-R5 are shown in Fig. 3.25. The air-entry value can be estimated at about 20 kPa, quite close to that of ZY-R3. There is no much difference between the desorption and sorption curves, but the difference between the fitted curves and the predicted curve is quite significant in the matric suction of 0.01-1000 kPa. It can be attributed to the fact that the prediction based on grain size distribution does not take into account the stress effect on the measured SWCCs, while the prediction agrees very well with the desorption curve of ZY-R3, which is tested under zero-stress condition. As the matric suction goes beyond 1000 kPa, the difference of the three curves become ignorable. Hysteresis is reversed in the suction up to 100 kPa, but it is a normal hysteresis loop from 100 kPa matric suction. It is due to the measured data. No residual water content can be defined clearly, but can be roughly estimated at about 100000 kPa.

Fig. 3.26 shows the SWCCs of ZY-R6. The fitted curves of desorption and sorption are close to each other and also close to the prediction based on the grain size distribution. The air entry value is about 50 kPa, for the desorption curve. The residual water content cannot be identified clearly but can be estimated to be about 150000 kPa. Again, the stress effect on ZY-R6 make the fitted curves different under the matric suction from 0.01 to 2000 kPa, which is also due to the stress effect that makes the prediction only based on grain size distribution different from the measure data. Com-
pared with the fitted curves of ZY-R5, it can be seen that ZY-R6 has a little higher air-entry value under ISO stress of 50 kPa.

In Fig. 3.27, the SWCCs of ZY-R7 are illustrated. The air-entry value for the desorption and sorption curves is about 90 kPa, and the residual water content is 6%. Therefore, comparing the desorption curve ZY-R3, R6 and R7, it is found that air-entry value increases applied external stresses. This is understandable because a higher confining stress can bring about a higher dry density and the air-entry value of SWCC generally increases with dry density. When all the fitted curves are plotted in a same plot, it can be found that the difference between each sample can only be seen up to 1000-2000 kPa. The fitted curves of all the samples nearly overlap when the suction is higher than 2000 kPa.

Several conclusions can be drawn from the analysis on measured SWCCs by means of SoilVision. The prediction based on the grain sizes fit the measured desorption curves of the sample under zero stress better than the sorption curve. The prediction can no longer fit either desorption or sorption curves of the samples tested under certain stress conditions. Based on the fitted curves, it is found that one of the stress effects is that the air-entry value is increased, compared with the curves for the sample with zero stress. The major difference of fitted curves and the predicted curve is with the range of 0.01 to 2000 kPa. As matric suction goes higher, the soil-water characteristics of either fitted curves or the prediction look very similar.

3.7 SUMMARY

The experimental SWCC results can be summarized as follows:

1) The soil-water characteristics of the expansive soil samples tested in the study show that air-entry value of expansive soils is high, mostly higher than 50-100 kPa, and the desorption rate is low, because the expansive soils are clays of high
plasticity. Compared with non-swelling soils – CDV, expansive soils have a higher air-entry value and lower desorption rate.

2) Based on the comparison of soil-water characteristics of natural and recompressed samples, it can be concluded that intact expansive soil samples desaturate more quickly than recompressed samples in the suction range of 1 to 10 kPa. So that natural samples seem to have a lower air-entry value. The difference may be attributed to the existing natural fissures and large pores in natural samples. The re-compacted samples are more uniform without those large drainage paths.

3) The hysteresis is generally not significant in expansive soils compared with one CDV. The first hysteresis loop is relatively large in size compared with subsequent cycles. At the end of the second wetting path, the wetting curve goes higher than the second drying path. The most interesting point is in the third cycle, where the wetting curve is higher than the third drying path. Therefore, it can be seen as a 'reversed' hysteresis loop, which has rarely been reported in the literature. There may be some change gradually developed in soil during continual wet-dry cycles, due to those special minerals, and some particular chemicals in water used and the proposed double-structure of the highly expansive soil.

4) Based on the experimental results, the stress effects on SWCCs of ZY expansive soil are found to be significant. Applied external stresses reduce the volume of large pore spaces (inter-aggregate pores) in soil during the consolidation, so that the desorption rate is significantly higher for the samples tested under stresses in the matric suction range of 1 to 10 kPa. Beyond 10 kPa, the desorption rates are the same for all the samples, regardless the stress conditions. It can be inferred that the microfabric of soil remains unchanged under stress. 1D and ISO stress conditions have different influence on soil-water characteristics, when SWCCs are
expressed in terms of VWC, but look very similar when they are expressed in terms of GWC. It is due to the transient difference of the stress conditions. It seems that ISO stresses of different values do not have a different effect on SWCCs. This may be due to the fact that the values of stresses being applied in this study, 50 and 100 kPa, are greater than the swelling pressure of the soil, 25 kPa. Hysteresis becomes reversed for the samples under stress. The loops are nevertheless small in size. The hysteresis loop of the soil sample tested under conventional zero stress condition looks like an envelope of SWCCs of the soil samples tested under various stress conditions.

5) SoilVision helps analyze the measured data and extend the data to the complete range of matric suction. It is observed that the predicted curves can fit the SWCC of a drying path of a sample under the conventional zero stress condition very well. But the fitted and measured curves are different for the samples tested under non-zero stress conditions in the matric suction range of 0.01 to 2000 kPa. Almost no difference can be seen from the fitted curves and the prediction when the matric suction is higher than 2000 kPa. Comparing the samples tested under the three different ISO stresses, the air-entry value is found to be increased by stress applied on the soil. The higher the stress is, the higher the air-entry value is. This can indeed be attributed to the increase in soil dry density since the applied stress can reduce void ratio and increase soil density.

3.8 NOTES FOR THE TESTS

1) The geotechnical lab is well air-conditioned and the temperature normally does not vary significantly, within the range of ± 0.5°C. Thus, it is appropriate to assume the temperature remains constant.
2) Evaporation is assumed to be negligible because one rubber lid is plugged in the opening of the burette to prevent potential evaporation.

3) The soils are sedimentary expansive soil and naturally unsaturated. Oven drying of 100°C to 110°C may destroy the fabric characteristics of such samples. So only the oven with the temperature ranging from 40°C to 50°C is used to dry the soil samples except for measuring water contents.

4) In the 105°C oven, 24 hr is required to dry a wet sample removed from extractors after SWCC tests.

5) The mercury differential regulator of the membrane pressure extractor is not used because of the potential leakage from the container. It is assumed that soil samples still have a good contact with the cellulose membrane under the major range of applied matric suctions, without the inner top membrane pressing the samples down to the membrane at the bottom.

3.9 REFERENCES


Table 3.1  Some natural properties of LZ and ZY samples

<table>
<thead>
<tr>
<th></th>
<th>Depth of sampling (m)</th>
<th>Dry density (kg/m$^3$)</th>
<th>Natural density (kg/m$^3$)</th>
<th>Natural water content (%)</th>
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<tr>
<td>LZ</td>
<td>1.4-3.6</td>
<td>1560-1650</td>
<td>1970-2040</td>
<td>23.4-26.4</td>
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<tr>
<td>ZY</td>
<td>1.5-3.0</td>
<td>1370-1510</td>
<td>1810-1910</td>
<td>26.8-30.3</td>
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Table 3.2  Grain size distributions of LZ, ZY and PK CDV (British Standard)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Percentage (%)</th>
<th>Soil type</th>
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<tr>
<td></td>
<td>&lt;0.002 mm</td>
<td>0.002 ~ 0.06 mm</td>
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<td>Silt</td>
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<td>LZ</td>
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<td>43</td>
</tr>
<tr>
<td>ZY</td>
<td>49</td>
<td>44</td>
</tr>
<tr>
<td>CDV (from Peak Road)</td>
<td>15</td>
<td>67</td>
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</table>
Table 3.3  Summary of plasticity indices

<table>
<thead>
<tr>
<th>Sample</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>SL (%)</th>
<th>Activity</th>
<th>Classification (USCS)</th>
<th>Degree of expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>LZ</td>
<td>63.5</td>
<td>27.3</td>
<td>36.2</td>
<td>12.0</td>
<td>0.66</td>
<td>CH</td>
<td>Medium to High</td>
</tr>
<tr>
<td>ZY</td>
<td>61.5</td>
<td>28.5</td>
<td>33.0</td>
<td>N/A</td>
<td>0.67</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CDV (after Chiu, 2000)</td>
<td>48.0</td>
<td>35.0</td>
<td>13.0</td>
<td>N/A</td>
<td>2.33</td>
<td>ML</td>
<td>N/A</td>
</tr>
<tr>
<td>Sample identity</td>
<td>Initial dry density (kg/m³)</td>
<td>Initial compaction water content (%)</td>
<td>Initial void ratio</td>
<td>Initial degree of saturation (%)</td>
<td>Degree of saturation after saturation</td>
<td>Sample diameter /height (cm/cm)</td>
<td>Void ratio after consolidation</td>
</tr>
<tr>
<td>-----------------</td>
<td>---------------------------</td>
<td>-------------------------------------</td>
<td>-------------------</td>
<td>-------------------------------</td>
<td>----------------------------------</td>
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<td>---------------------------------</td>
</tr>
<tr>
<td>LZ-R1</td>
<td>1596</td>
<td>25.1</td>
<td>0.71</td>
<td>96.8</td>
<td>99.6</td>
<td>5.0/1.0</td>
<td>N/A</td>
</tr>
<tr>
<td>CDV-R1</td>
<td>1078</td>
<td>20.0</td>
<td>1.47</td>
<td>36.1</td>
<td>100</td>
<td>5.0/1.0</td>
<td>N/A</td>
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<tr>
<td>ZY-N1</td>
<td>1386</td>
<td>29.3</td>
<td>0.97</td>
<td>82.6</td>
<td>95.1</td>
<td>5.0/1.0</td>
<td>N/A</td>
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<tr>
<td>ZY-N2</td>
<td>1376</td>
<td>29.3</td>
<td>0.98</td>
<td>81.3</td>
<td>93.2</td>
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<tr>
<td>ZY-N3</td>
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<td>0.98</td>
<td>84.3</td>
<td>91.2</td>
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<td>ZY-R1</td>
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<td>31.0</td>
<td>0.99</td>
<td>85.4</td>
<td>93.9</td>
<td>5.0/1.0</td>
<td>N/A</td>
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<td>ZY-R2</td>
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<td>30.3</td>
<td>0.99</td>
<td>83.7</td>
<td>100</td>
<td>7.0/2.0</td>
<td>N/A</td>
</tr>
<tr>
<td>ZY-R3</td>
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<td>30.5</td>
<td>0.99</td>
<td>84.1</td>
<td>100</td>
<td>7.0/2.0</td>
<td>N/A</td>
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<td>ZY-R4</td>
<td>1369</td>
<td>31.0</td>
<td>0.99</td>
<td>85.2</td>
<td>96.1</td>
<td>5.0/1.0</td>
<td>N/A</td>
</tr>
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<td>ZY-R5</td>
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<td>1.00</td>
<td>82.0</td>
<td>100</td>
<td>7.0/2.0</td>
<td>0.99</td>
</tr>
<tr>
<td>ZY-R6</td>
<td>1368</td>
<td>30.3</td>
<td>1.00</td>
<td>83.1</td>
<td>100</td>
<td>7.0/2.0</td>
<td>0.99</td>
</tr>
<tr>
<td>ZY-R7</td>
<td>1371</td>
<td>30.5</td>
<td>0.99</td>
<td>83.6</td>
<td>100</td>
<td>7.0/2.0</td>
<td>0.97</td>
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</table>
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CHAPTER 4  MICROFABRIC ANALYSES OF SEM AND MIP

4.1 INTRODUCTION

4.1.1 Definitions of fabric and double-structure

Lambe (1958) extended the meaning of the term of structure, which was initially limited to the arrangement of soil particles, by including electrical forces as a component of structure. In ‘Microstructure of Fine-Grained Sediments’ (Bennett et al., 1990), the authors also referred to ‘microstructure’ as the combination of fabric and physicochemistry. In the book of ‘Fundamentals of soil behavior’ (Mitchell, 1993), the term of ‘fabric’ was used for soil particle arrangement, particle groups, and pore spaces in a soil. It was also stated in the book that ‘structure’ is sometimes used interchangeably with ‘fabric’ but structure was more preferably used to refer to the combined effects of fabric, composition and interparticle forces, with both fabric and its stability taken into account. Hillel (1998) used structure only, implicitly including bonding and forces. Therefore, to be more precise, the author will only use fabric and microfabric instead of structure and microstructure in this chapter, according to Mitchell (1993), since the terms herein are referred to particle arrangement and pore orientation only, without interparticle forces and compositions considered.

The framework of the behavior of expansive soils was presented by Alonso et al., 1987). The possible fabrics of double-structure expansive soils were illustrated as Fig. 4.1 (Gens and Alonso, 1992). The fabric types are shown in Fig. 4.1(a) and (b)
and the elementary particle is illustrated as Fig. 4.1(c). In Fig. 4.1(a), the particle assemblages are formed by arrays of elementary particle arrangements and they are described as matrices. The pores are made up by intramatrix pores existing between elementary particle arrangements. In Fig 4.1(b), the elementary particle arrangements join together to make aggregates resulting in a three dimensional structure of a granular type. Both inter- and intra-aggregate pore spaces exist. There are also inraelement pores separating the clay platelets in an elementary element. The concept of double-structure, which is made up of microstructure and macrostructure, was therefore proposed. The inter-aggregate pores and the aggregate arrangements can be grouped in the macrostructure, whereas the intra-aggregate reactions and pore spaces are in the scale of microstructure (Alonso, 1998). The behavior of unsaturated expansive soils can be attributed to these two structure levels. The microstructure is where the basic swelling of the active minerals takes place and the macrostructure is responsible for major structural rearrangements. For a highly expansive double-structure clay, the fabric was postulated as shown in Fig. 4.2 (Alonso et al., 1995). Both intra matrix and intra-element pores exist in this type of soil fabric, which are equivalent to inter-aggregate and intra-aggregate pores. This figure illustrates the bimodal fabric, which is corresponding to the double structure concept. In this thesis, the soil fabric can be postulated as either the fabric in Fig. 4.1(b) or the fabric in Fig. 4.2. With the MIP results reported by Atabek et al. (1991), the bimodal distribution of pore sizes was verified, as shown in Fig. 4.3. There are a lot of pore spaces with diameter from about 3 to 150 μm, which can be viewed as inter-aggregate pores. It can also be seen that pores with diameter from 0.03 to 0.003 μm, which are intra-aggregate pores, also compose a major part of the soil void spaces. In this thesis, to be more consistent the author only use the term of ‘inter-aggregate pores’ when referring to those large pores and use ‘in-
tra-aggregate pores’ to refer to the small pores within an aggregate. The inter-aggregate pores are mostly in the range of 7.5-150 μm and the intra-aggregate pores are from 0.06-7.5 μm in diameter.

The knowledge of soil fabric plays an important role as an aid for understanding the mechanical response of soil under various circumstances. SEM and MIP are widely used in soil science and soil mechanics worldwide (Prapaharan et al., 1985; Juang and Holtz, 1986; Craig et al., 1990). In this chapter, the characteristics of the microfabrics of the expansive soils tested in the thesis are addressed. The fabric of partially saturated soils are discussed using two sources of evidence: visual examination through Scanning Electron Microscopy (SEM) and porosimetry data from Mercury Intrusion Porosimetry (MIP) tests. The pore size distribution (PSD) of a soil can be illustrated by MIP test results. The results presented in this chapter can provide useful proofs for the interpretation of SWCC results.

4.1.2 Principle of SEM

The principle of SEM is shown in Fig. 4.4 (Reimer, 1985). When scanned by the microprobe, the specimen displays various interaction effects: electron scattering and diffraction; secondary electron and Auger electron emission; photon emission; electron absorption and energy loss; phonon and plasmon excitation; generation of electric and magnetic fields, etc. Each of these effects may be used for imaging, provided that a suitable measuring device is available for converting the object effect into an electrical signal for processing.

The amplified output of the selected detector serves to control the intensity of the electron beam of a CRT with synchronised scanning. Illumination at a given point of the screen is thus directly related to the intensity of the selected interaction effect at
the corresponding point of the specimen. After at least one scan period, the recorded
data result in an image (Eberhart, 1991). The method helps visualize real particle ar-
rangements of fine-grained soils, especially for clay.

XRD tests were also conducted to identify mineralogical compositions of the
expansive soils. The test results show that the major portions of ZY and LZ are smec-
tite and quartz. The minerals in the smectite group, e.g. Montmorillonite, are mostly
of high degree of expansion. Therefore, XRD results indicate that the swell potential
of these expansive soils is not negligible. However, the current results cannot quantify
the percentages of these minerals. More XRD tests should be carried out in the future.
The plots of XRD results are attached in Appendix IV.

4.1.3 Principle of MIP

Mercury intrusion porosimetry (MIP) is one of the techniques used to measure
PSDs of dry soil samples. The use of mercury intrusion porosimetry is to study the
influence of pore structure in the engineering behavior of soils. In this study, the ob-
jective of conducting MIP tests is to investigate the influence of PSDs on SWCCs of
the expansive soil samples. Better interpretation of SWCC test results may be sought
out by this means in a quantitative manner. In fact, some researchers have shown that
MIP can be a prompt way to determine SWCCs and good agreement has been proved
between MIP results and measured SWCCs (Prapahan et al., 1985; Kong and Tan,
2000). SEM is a method to analyze pore and particle arrangements in soils in a quali-
tative manner.

The method involves measurement of the pressure required to force mercury
into the pores of a dry sample and the volume of intruded mercury at each pressure
(Lawrence, 1978). The principle of the method is that a nonwetting fluid (fluid-to-
solid contact angle > 90°) will not enter the pores without application of pressure.
The soil moisture characteristic curve, i.e. soil-water characteristic curve, is usually obtained by using water as the pore fluid where surface forces draw water into the soil pores. SWCC is indeed a reflection of the PSD of a soil. However, compared with time-consuming SWCC tests, rapid mercury intrusion procedures have some advantages over SWCC tests and MIP therefore becomes another method to characterize the PSD of a soil. The use of mercury as the fluid will require the use of external pressure to intrude the pores. This procedure can be measured and is related to the characteristics of the pore being intruded by the equation (Prapaharan et al., 1985):

\[ P_m = -\frac{4T_m \cos \theta_m}{d} \]  

(4.1)

in which \( T_m \) = surface tension of mercury; \( P_m \) = intrusion pressure applied to the mercury; \( \theta_m \) = contact angle between mercury and soil; and \( d \) = diameter of pore being intruded.

For water, the same relationship applies:

\[ P_w = \frac{4T_w \cos \theta_w}{d} \]  

(4.2)

in which \( P_w \), \( T_w \) and \( \theta_w \) = the parameters for water. Thus, from Eqs. 4.1 and 4.2, the following equation is obtained

\[ P_w = \frac{T_w \cos \theta_w}{T_m \cos \theta_m} P_m \]  

(4.3)

so that capillary pressures for water can be predicted from measurements using the mercury. In this chapter, several MIP curves are selected to predict SWCC and the results are presented. The pore-water pressure and the pressure in mercury intrusion test with respect to pore size can be illustrated as Fig. 4.5. It can be seen that for a given pore size the water pressure is smaller than the mercury pressure. However, it should be noted that, as illustrated by Delage and Lefebvre (1984), the measured PSD
does not give a measurement of pore sizes, but of pore entrance sizes. As shown by
the comparison of the first intrusion and re-intrusion, they succeeded to demonstrate
that there were a significant amount of restricted pore spaces in the soil, where the
mercury were entrapped after first extrusion. In this thesis, the extrusion curves of
MIP are not shown in this chapter, but the data are included in Appendix IV. Studying
the MIP results, it is found that the amount of entrapped mercury is about 75.5 % of
the total intrusion. Therefore, the discussion of MIP results is focused on the first in-
trusion, which represents the whole porosity, according to Delage and Lefebvre

As for water, $T_w$ and $\theta_w$ are equal to 72.3 mN/m at 23°C and zero, respectively
(Fredlund and Rahardjo, 1993; Hillel, 1998). Therefore, corresponding to 1 and 500
kPa matric suctions, the pore radiuses are 145 and 0.3 μm, respectively. Since pore
diameter is more commonly used for pore size, hereinafter only all the quantities for
pore sizes will be expressed with pore diameter. Hence, the SWCCs measured in this
thesis can reflect the PSDs from 0.6 to 290 μm. As for the mercury, the values of the
surface tension of 484 mN/m and contact angle of 141° are adopted (Lawrence, 1978;
Mitchell, 1993). The intrusion pressure is from 10 to 25100 kPa, corresponding to the
range of pore sizes from 0.06 to 150 μm. Those pores greater than 150 μm in diameter
cannot be covered by the MIP results. From calculation, it can be seen that only the
 corresponding parts of SWCCs measured under 1-2 kPa is not covered by the range of
MIP tests, whereas the soil-water characteristics in the rest range can be interpreted
with the assistance of the measured MIP results.

According to Mitchell (1993), microfabric is in the scale of tens of micrometers,
minifabric is a few hundred micrometers in size and macrofabric may contain cracks,
fissures, root holes, laminations etc. Therefore, the MIP results presented in this chap-
ter are indications of soil microfabrics. However, in the concept of bi-modal PSD, pores are divided into two categories, one in macrostructure and the other in microstructure (Alonso et al., 1987; Gens and Alonso, 1992). The pores in macrostructure are about several to hundreds of micrometers, whereas the pores in microstructure are 2-3 orders smaller in size, according to the measurement of MIP tests from Atabek et al. (1991), which was quoted by Gens and Alonso (1992) and Alonso (1998).

However, it should be noted that there are some limitations of MIP (Mitchell, 1993). They are:

1. Pores must be dry initially.
2. Isolated pores are not measured.
3. Pores accessible only through smaller pores will not be measured until the smaller pore is penetrated.

Despite of these limitations, MIP can still be useful for measuring PSDs.

4.2 SAMPLE PREPARATION OF SEM AND MIP

Proper sample preparation is of much importance in order to get reliable final results. Significant disturbance should be avoided and any potential disturbance that can change the soil fabrics during this process should be reduced to as a low level as possible. Extreme care should be taken when handling the soil samples. However, for the samples with confining pressure during the measurement of SWCCs, some rebound surely occurred after unloading.

**Drying:** One requirement of both SEM and MIP is that the moisture must be removed from pore spaces prior to testing. Delage and Lefebvre (1984) concluded that oven-drying should be avoided because it results in some structure change after drying. It
was suggested that either critical-point drying or freeze-drying technique be used to reduce changes in the soil structure resulted from the removal of pore-water (Lutenegger and Saber, 1987). In this study, the freeze-drying technique was adopted to dry samples. The procedure is described as follows.

(1) Being removed from the apparatus for measuring SWCCs, i.e. conventional volumetric pressure plate extractor, or pressure plate/membrane extractor, or the modified 1D volumetric pressure plate extractor, or the modified triaxial apparatus for measuring SWCC, each sample was immediately sealed into an air-tight plastic bag to maintain the moisture content unchanged.

(2) When it was time for drying, all the samples were taken out from the sealed bags, separated by filter paper and then put into several glass bottles.

(3) Then all samples contained in the glass bottles were firstly frozen in a refrigerator at \(-20^\circ\text{C}\) for 0.5-1 hr. It should be noted that there is some unavoidable structural change due to the volume increase of the soil moisture when water is frozen into ice.

(4) Glass bottles containing soil samples were immediately transferred to a freeze-dryer (Labconco, the brand name of the freeze-dryer). Soil samples were held in a sublimation unit to remove moisture from the soil for 48 hr.

48 hr is assumed to be enough to dry these clay samples. Normally 24-hr is adequate for drying soils. Due to the special structure of expansive soils, soil samples were dried for 48 hr to ensure complete dryness. After 48 hr, each of soil samples was again separately sealed in plastics bags for future SEM and MIP tests. At that time, some small cracks of an opening about 0.1-0.3 mm (100-300 μm) can be observed in the sample by visual inspection, which means even freeze-drying cannot completely
prevent structure change. The cracks forming during the drying process, however, do bring some error into the measured MIP data, because the range of the pore sizes measured in the tests is from 0.06-150 μm. Some error may be involved in the pore sizes of tens of micrometers to 200 μm. As for SEM, these cracks do not cause error, since the whole area that can be seen under microscope are mostly about the same size of the width of a crack or even smaller.

Cutting and coating: only a small volume of samples is used for SEM, about 2×2×2 mm³. Therefore, a small particle was cut from each whole sample by a sharp knife. These small particles were coated with gold under the vacuum of argon gas and the rest of samples were used for MIP tests.

The equipment used for taking SEM photos was KYKY Model 1000B, manufactured in China. For the photos taken with the microscope, the author chose the values of magnification ranging from 170 to 2500. Most of soil pores can be viewed in this range of magnification. Photos taken in the SEM test can assist interpretation of SWCC test results presented in Chapter 3. MIP tests were conducted in the department of Geology at Peking University. Therefore, the detailed test procedures are not introduced in this thesis. Only the results are presented and discussed.
4.3 SOIL FABRICS OBSERVED IN SEM TESTS

4.3.1 SEM photos of different soil samples

The images of LZ-R2, ZY-R1 and Peak Road CDV-R1 are illustrated as Fig. 4.6-8 (760×, 0011, 0008, 0002), representing three different soil samples used in the thesis (in the bracket, the value before ‘×’ is the magnification and the others are the numbers shown on the bottom of the film).

First of all, it can be noticed by comparing the three pictures that there is a clear difference of soil fabrics between expansive soils (LZ and ZY) with CDV (Peakroad). According to Hillel (1998), the fabric of CDV is defined as ‘single grained’ (Fig. 4.8). The particles are of the sizes of 5-30 μm, classified as ‘silt’ (0.002-0.06 mm). Clay platelets coated on silt particles cannot be observed from this figure. This is probably due the low fraction of clay particles in the CDV. However, for expansive soils, either LZ or ZY, the fabric is ‘aggregated’, containing only clay platelets. The aggregates are about 5-10 μm in size and seem to be arranged in a flocculated manner, referring to Fig. 4.7. No silt or sand particle can be seen from Fig. 4.6 and 4.7. According to Mitchell (1993), typical fabric units are up to a few of micrometers across. Therefore, the aggregates can be referred to as ‘microaggregates’ in these figures (Hillel, 1998).

LZ-R2 has a high dry density of 1.6 g/cm³. It can be seen from Fig. 4.6(760×, 0011) that it truly has the densest fabric amongst the three. Its pores have some orientation generally. ZY-R1 is only of 1.37 g/cm³ thus it has some kind of loose but aggregated fabric, as seen from Fig. 4.7 (760×, 0008). Referring to Fig. 4.8 (760×, 0002), CDV is more uniform regarding pore spaces than the two expansive soils. Pore sizes of CDV (in terms of equivalent diameter) are mostly in the range of 1-10 μm, corresponding to the matric suction from 20 to 270 kPa in the measured SWCC. It can
explain that the desorption rate in its SWCC is high under this suction range, as shown in Fig. 3. 14. There are two categories of pores in expansive soils, large and small pores (Fig. 4.6 and 7). Large pores seem to be 20-50 μm in diameter, which can be viewed as inter-aggregate pores, corresponding to the matric suction from 1 to 10 kPa. The desorption rate of these two expansive soils is nevertheless smaller than that of Peakroad CDV, referring to Fig. 3.11-14. This can be attributed either to the fact that these large pores are occluded in expansive soils due to compaction, or to the significant reduction in soil volume for CDC under low matric suction, or to the combined effects of these two causes. Small pores range from infinitely small to about several micrometers in diameter, which can be classified as intra-aggregate pores. The physicochemical forces may also be different for expansive soils and CDV, which may cause different SWCCs for these soils.

Moreover, when comparing Fig. 4.6 and 4.7, it can be seen that the large pores are larger in size for ZY-R1 than those pores in LZ-R1. This can explain the higher entry value of the SWCC for LZ than that for ZY. By visual inspection, the small pores (several micrometers) in the two samples appear to be quite similar in amount. The similar desorption rates of these two soils can also be explained by the

Lambe (1958) studied different orientations of particles of recompacted clay according to their different water contents of compaction. It was suggested that the soil particles are orientated in the flocculated pattern when compacted at the water content of dry of optimum and in a dispersed matter when soil is compacted wet of optimum water content. From the compaction curve shown in Chapter 3, it can be seen that the optimum water content is about 22 %, while the water content for compaction adopted in this study is 30 %, which is wet of optimum, so that the resulted structure should be dispersed. With reference to Mitchell (1993), a soil sample with a dispersed structure
looks different from different directions. Two SEM images may indicate that the clay
particles of recompacted soil samples in this study look to be dispersed so that the
views from two different directions are different in the photos, parallel with and per-
pendicular to the plane of clay platelets, shown as Fig. 4.9 and Fig. 4.10 (1250×, 0023
& 0025). Fig. 4.9 appears to in the parallel configuration and Fig. 4.10 seems to be of
the perpendicular configuration. Benson and Daniel (1990) also showed some photos
for recompacted clays with different water contents, which are the water contents of
optimum and wet of optimum. It was seen that the clay platelets were in a parallel ar-
rangement, while looking from the perpendicular configuration.

4.3.2 Different fabrics for intact and recompacted samples

As discussed in Chapter 3, natural and recompacted expansive soils have differ-
ent soil-water characteristics. The air entry value is generally smaller for intact sam-
plies than for recompacted samples. Two photos for natural and recompacted ZY sam-
plies are shown to illustrate the difference between their microfabrics. The SEM
photos taken for an intact ZY sample, ZY-N1 and a recompacted sample ZY-R1 are
shown in Fig. 4.11 and Fig. 4.12, respectively. It can be seen that the fabrics of these
two samples are clearly different. The pore sizes of ZY-N1 are relatively less uniform,
in which both small and large voids exist. Clay particles are platelets and arranged in
a dispersed pattern (Lambe, 1958; Benson and Daniel, 1990). The pore sizes of ZY-
R1 are more uniform since it can be seen that the voids are mostly of similar size,
which is about the same as those small voids in the intact sample. It seems that those
large voids that originally existed in the natural sample have been eliminated by re-
compaction. Therefore, it is understandable that the air-entry value of a natural sample
is smaller. And the desorption rate from 1 to 35 kPa is greater than that of a recompacted sample due to the large pores inside natural sample. The desorption rate of the matric suction from 35 to 500 kPa is nearly the same for either intact or recompacted sample because most small voids are of almost same sizes for both samples. It can be seen from Fig. 4.12 that the orientation of particles is also dispersive, but particles in the recompacted sample look more packed and the particle edges are generally rounded, compared with that in the natural one. It means that compaction may have changed the size and orientation of pores in the sample but not particle orientation.

4.3.3 Change in soil fabric after wet-dry cycles

The picture of a natural ZY sample, ZY-N3 after three wet-dry cycles is shown in Fig. 4.13 (382×, 0024). After several wet-dry cycles, the fabric seems denser since pores are mostly of the same size. But the reduction of soil void spaces cannot be clearly observed in this figure. However, it can still be postulated due to wetting, some small particles are disintegrated and filled into those big voids. Consequently, the soil has more small pores and the PSD becomes more uniform. The total volume might have no much change but the PSD is different after several cycles, and the water retention capacity may be increased. Therefore, it can explain the SWCC of three cycles in Fig. 3.17, in which the water content of the third wetting path is higher than that of the third drying path. However, without a photo with the same scale with Fig. 4.11, this postulation of the fabric change due to repeated wet-dry cycles cannot be substantiated. Further detailed work should be done in the future, in order to verify the workability of this postulation.
4.3.4 Influence of different stress conditions on soil fabrics

The SWCCs of ZY-R1 and ZY-R7 were measured under zero-stress and isotropic stress (100 kPa), respectively. The SEM photo on ZY-R7 is shown as Fig. 4.14. Comparing with Fig. 4.7, it can be observed that the fabric seems looser in Fig. 4.7 than that in Fig. 4.14. The void spaces seems to be larger in ZY-R1 in Fig. 4.6 than in ZY-R7 in Fig. 4.14. This means the volume of voids has probably been reduced by applied external stress. Therefore, in Fig. 3.22, the reduction in volume and decrease in water content are greater for ZY-R7, which is tested under non-zero-stress condition. (760×, 0008, 0029/0030). Therefore, it may be able to explain that the greater desorption rate of ZY-R7 in the suction ranging from 1 to 10 kPa than that of ZY-R3, which is a sample tested under zero stress condition.

The two images of ZY-R5 and ZY-R6 are shown in Fig. 4.15 and 4.16 (380×, 0016, 0018). These are two samples tested under 1D stress of 50 kPa and ISO stress of 50 kPa, respectively. It can be seen that 1D and ISO stresses may bring about different soil fabrics due to different stress conditions. The fabric of ZY-R5 looks relatively anisotropic and the aggregate arrangement has certain orientations, compared with those of ZY-R7, whose pores look more uniform and isotropic. It means that different stress conditions bring about different fabric changes in. According to Chen (1988), the measured lateral swelling pressure is the higher than the vertical one. Both the vertical and the lateral sides were assumed rigid with any deformation during the test. As the equilibrium state was reached, it was found that the ratio of the vertical to the lateral pressure would reach unity. Therefore, it may be postulated that the different fabrics brought about by different stress conditions was mostly caused by the transient different effects of the stresses, when soil fabrics were relatively loose and thus easy to be affected. The 1D stress might cause anisotropic pore and particle arrange-
ment, while the soil fabric seems to be more isotropic under the ISO stress. Moreover, with SEM photos, Delage and Lefebvre (1984) also showed that 1D consolidation brought about anisotropy in soil structure.

However, it should be noted that it is hard to identify clear difference between ZY-R6 and R7. SEM is a method that is able to investigate soil fabrics only in a qualitative manner. The external stresses on ZY-R6 and R7 are both isotropic but only differ in the magnitude. So to quantitatively examine the difference in their fabric seems not possible by this means. According to Delage and Lefebvre (1984), large inter-aggregate pores get compressed first and then progressively smaller pores are compressed. Intra-aggregate pores are not compressed until all macro pores have been compressed.

It should be noted that the change in soil fabrics visualized by SEM in this chapter is due to the compression of inter-aggregate pores under stresses. It should be noted that external stresses are assumed not to be able to change intra-aggregate pores (Alonso, 1998). Two pictures of ZY-N1 and ZY-R7 with the same magnification (2500×, 0005, 0022) are shown in Fig. 4.17 and Fig. 4.18, respectively. It can be seen that the fabrics look similar in these two figures, under such a large magnification scale. They both possess some flat areas with a few leaves curled on the surface. The light-colored ‘leaves’ like cabbage are one major feature for smectite minerals (Mitchell, 1993), since ZY contains montmorillonite, a subgroup of the smectite group. These smooth and flat areas must be composed by a number of platelets. In Fig. 4.18, some parallel clay particles can be seen. It can be reasonably inferred that the layers for ZY-N1 are also arranged in a similar manner, though the edges of layers cannot be seen from Fig. 4.17. For both pictures, particles are platelet-like and associated in a parallel manner. It seems that the intra-aggregate arrangement of these two
sample tested under two distinct stress condition have not been changed. More importantly, it can be noticed that the intact and recompacted samples appear similar under a high magnification. Thus, it seems that recompaction may not affect the microfabric characteristics of the soil, at least the arrangement of clay platelets within an aggregate may remain the same after recompaction.

4.3.5 Summary on SEM tests

Soil microfabrics can be visualized through SEM. The findings from SEM tests are interesting. The microfabrics of different soils are different. The fabric of the volcanic soil (CDV), silt of low plasticity, is single-grained, whereas the fabrics of expansive soils, either intact or recompacted, are aggregated. The particles are arranged in a parallel way for both intact expansive soil samples and recompacted samples which are compacted at a water content of wet of optimum.

Soil fabric can be altered by applied stresses. Inter-aggregate pores seem to be compressed in a visible scale by external stress. Generally, a sample looks denser if it is under non-zero stress condition. Assuming the initial fabrics are exactly identical for all recompacted samples, 1D and ISO stresses may cause different soil fabrics due to some transient difference between these two stress conditions during the beginning part of the tests. However, SEM seems not able to identify the difference of two samples under a same stress condition but two different stress values. Therefore, SEM cannot be used to quantitively describe soil fabrics but used as a qualitative method to help visualize the mysterious world of soil microfabrics.
4.4 QUANTITATIVE MICROFABRIC ANALYSIS WITH MIP

PSD data can be expressed with accumulated volume of intruded mercury. Because the pore size magnitude usually cover several orders, it is common to plot the pore size axis in a logarithmic scale. On the other hand, PSD can be expressed as the so-called density function (Juang and Holtz, 1986), which is a derivative of the pore size distribution. The unit for the pore density function are cm$^3$/g/chosen diameter interval, but the diameter interval is dimensionless on a logarithmic scale. In this section, both of them will be used to present MIP data to investigate the different PSDs of different soil samples and their possible implication on SWCCs.

4.4.1 Comparison of natural and recompacted samples

Two MIP curves of ZY-R1 and ZY-N1 are shown in Fig. 4.19. In Fig. 4.19 (a), the coordination is accumulative intruded volume of mercury in a unit weight of soil sample. The figure shows the process of mercury intrusion of the particular samples. Mercury first intrudes to those big pores, as the pressure increases, those small pores can also be occupied by mercury. It can be seen from Fig. 4.19 (a) that the amount of mercury related to the pore diameters of 30-150 μm intruded into ZY-R1 is quite significant, when the pressure is quite low, about 10-50 kPa. In that range, however, there is no mercury intruded into ZY-N1 at all. However, as seen from Fig. 3.14, it was postulated that there may be more large pores for ZY-N1, the natural sample than for ZY-R1, the recompacted sample, because the desorption rate for ZY-N1 is greater than that for ZY-R1 from 1 to 35 kPa. The reason might be that the data from 0-10 kPa in MIP is not recorded and there is some disturbance leading to the disappearance of large pores and natural fissures whose openings are in the range of 7.5-150 μm, in handling the natural sample during the test. Some other results of ZY sample pre-
sented later will show that the amount of intruded mercury in the large pore range (50-150 μm) is significant, so it seems not possible for ZY-N1 to have no large pores at all. If this part is skipped, the test result looks reasonable. Therefore, the MIP curve of ZY-N1 can be postulated as shown in the dashed line in Fig. 4.19 (a), assuming that there are some pores with the size from 150 to 300 μm in the natural sample.

As seen from Fig. 4.19(a), the increase rate of intruded mercury related to the pores of 0.3-30 μm are the same for both the natural and recompacted samples. It can explained the reason for the same desorption rate of ZY-R1 and ZY-N1 from 35 to 500 kPa in Fig. 3.14, corresponding to pore diameters from 0.3 to 20 μm. As pressure increases further, more mercury intruded into both samples, and the intrusion rate of ZY-N1 is greater than that of ZY-R1, which implies that there are more pores in this range of diameter in ZY-N1 than in ZY-R1. Since the maximum matric suction of SWCC tests of ZY-R1 and N1 is 500 kPa, pore sizes smaller than 0.3 μm are no longer relevant for the discussion of SWCCs.

At the end of the tests, the total intrusion is about 0.26 and 0.18 cm³/g. From calculation, ideally, the total intrusion should be about 0.365 cm³/g. Therefore, it seems that a few closed pores and some pores out of the pore size range covered by the tests may exist in the soil samples, although most of the pores are interconnected in the samples (Lutenegger and Saber, 1987; Mitchell, 1993). Moreover, it can be seen that there is a big difference in the total amount of intruded mercury volumes of the two samples.

Since the two samples are of the same initial density, the difference cannot be that significant. It implies that there might be some big openings in the intact sample, however, they are larger than 150 μm, so their existence cannot be detected in the results shown here, or these large openings vanished due to some disturbance in han-
dling. The author therefore still believe that the desorption rate of ZY-N1 is greater than that of ZY-R1 under the matric suction from 1-10 kPa, because of some large pores and natural fissures in the natural sample.

Incremental intruded volume is used in Fig. 4.19 (b), which is able to reflect the density function of the PSD. The higher the curve goes, the more pores corresponding to the given size exist in the sample. It can be seen that the volume of pores of 30-150 μm of ZY-R1 is much greater than that of ZY-N1. According to the postulation, as shown by the dashed line, the pore density function is also high in this pore size range. From 0.3-30 μm, the PSDs of this size range are similar for both samples, but only a few pores are in this range. From 0.3-0.06 μm, the number of the pores in ZY-N1 is significantly greater than that in ZY-R1. It indicates that some of the intra-aggregate pores might have been destroyed, when the natural sample were pulverized and then compacted into a recompacted sample. However, it should be noted that the pores in this small pore size range are intra-aggregate pores, which cannot be expelled by matric suction. Thus, its implication on SWCCs cannot be addressed here.

The major difference between natural and recompacted is in the range of large pore sizes, 30-150 μm. The recompacted sample has many large pores in this range, whereas the natural sample does not possess a large amount of pores in this pore size range. Considering the difference of the total intruded mercury volumes, it can be inferred that there may be some large pores and natural fissures in the natural sample whose opening should be larger than 150 μm, which cannot be detected by the MIP tests in the study. From 0.3-30 μm, the PSDs are more or less the same for both natural and recompacted samples. This can explain the similar desorption rate of ZY-R1 and ZY-N1 in their SWCCs presented in Chapter 3. Moreover, the pores of 0.3-0.06 μm are less in amount for the recompacted sample than for the natural sample. That
may implies that the process of preparation for recompacted samples may bring some reduction into intra-aggregate pores of soil. The PSDs agree well with the measured SWCCs, i.e. the more pores exist, the higher the desorption rate the soil has.

4.4.2 Comparison of the PSDs of different soils

When the results of ZY-R1 and LZ-R1 are plotted in the same figure, it can be seen that the small pores (intra-aggregate pores) are the major part of pores in LZ-R1 (Fig. 4. 20(a)). The volume of intruded mercury did not increase significantly until the pressure can make mercury intrude to pores smaller than 0.7 μm. The rate of increase in the volume of mercury intrusion of LZ sample is greater than that of ZY in the pores ranging 0.06-0.7 μm. However, the final total intrusion volumes are about the same for the two samples. Since the void ratio of LZ-R1 is about 0.7, smaller than the void ratio for ZY-R1, which is about 1, it may be inferred that the pores in the sample of LZ-R1 are more interconnected than those in ZY-R1.

In the pore density function shown in Fig. 4.20 (b), it can be observed that only a few pores distribute in the range of 0.7-150 μm. Most pores are smaller than 0.7 μm in size. It can be attributed to the high density of LZ-R1 after recompaction, 1600 kg/m³, which is higher than 1370 kg/m³ for ZY recompacted samples. Large pores have been compacted into smaller pores, whereas the same size large pores can exist in the loose sample of ZY-R1. Due to the deficiency of those relatively large pores, the LZ sample has a higher air-entry value, greater than 100 kPa in its SWCC (Fig. 3. 13).
In the pore sizes from 30 to 150 μm, the distinction between the two samples can be attributed to their different densities (void ratio), while the difference in the range of 0.06-0.7 μm can be attributed to their different soil origins. They can be called intrinsic differences. The soils are from two different sampling sites and they may have experienced different geological cycles, with different mineral contents, and so on. As results presented later on, it can be observed that no other recompacted ZY samples will show any peak in this pore size range. Therefore, it can be concluded that different soil samples of different soils will have a distinct difference in those relatively small pores, 0.06-1 μm, whereas, different soil densities or other extrinsic effects can only have some influence on those relatively larger pores, 30-150 μm in the study.

4.4.3 Stress effects on soil fabric

SWCCs of ZY-R5, R6 and R7 were measured under non-zero stress conditions. ZY-R5 was under 1D vertical stress of 50 kPa, ZY-R6 and R7 were tested under ISO stresses of 50 and 100 kPa, respectively. From the measurement of volume change, it is noticed that the samples were compressed to some extent under stress conditions. However, after releasing the stresses acting on the samples, it can be expected that the samples would increase in volume. Therefore, the PSDs may have changed and be different from the PSDs during the measurement of SWCCs. However, the deformation cannot be recovered completely, i.e. the deformations that the soil samples have undergone cannot be purely elastic. Therefore, some information about soil fabric changes were contained in the samples even though the confining pressures were released.
Plotted together with ZY-R1, it can be seen that the PSDs of ZY-R5, R6 and R7 have been changed and have a clear difference from that of ZY-R1 (Fig. 4.21(a)). In the pore size ranging from 7.5 to 150 μm, the amount of intruded mercury of ZY-R7 is greater than any other samples, followed by ZY-R6 and R5 in order. The increased amount of pores in this size range, 7.5-150 μm, can be attributed to the sizes of some inter-aggregate pores having been reduced, as the soil samples consolidated and underwent wet-dry cycles. It can be used to explain the greater desorption rates of ZY-R5, R6 and R7 in the suction of 1-20 kPa than the sample of ZY-R3, which is also tested under zero stress. From 0.3-7.5 μm, the rates of mercury intrusion of stress affected samples are close to each other and only a little higher than the intrusion rate of ZY-R1, which implies there must be some slight fabric change due to applied external stresses. The most likely change is that more pores evolved from pores of larger sizes, due to compression, or disintegration of large aggregates and rearrangement of aggregates. In this range, the reaction is mostly physical. As to smaller pore sizes, 0.3-0.06 μm, it can be seen that the curves of ZY-R5, R6 and R7 again are close to each other but have a clear gap from that of ZY-R1, whose increasing rate is clearly higher than the other three samples. This is hard to explain, since the pores of those small sizes, intra-aggregate pores, should not be affected by external factors. The probable cause may be some complicated chemical interactions or reactions, e.g. some cation exchange, between the minerals in the soil and the water used in the SWCC tests under stress conditions. From the discussion above, the close desorption rates of ZY-R3, R5 – R7 of the matric suction range of 20-500 kPa can be explained by their close pore distributions in the range of pore sizes. At the end of intrusion, the total amounts of intruded mercury are quite close for the samples, with only some small differences, since all these samples were compacted at the same initial density.
Fig. 4.21 (b) shows the pore density functions of these samples. As for the large pores, i.e. inter-aggregate pores, from 60 to 150 μm, ZY-R7 contains the largest amount of the pores of the sizes. ZY-R1 seems to have more small pores of diameter smaller than 0.3 μm. As for the middle part of the curves, it is hard to distinguish the differences between samples from this plot. It may be a proof for the similar desorption rates of the samples of ZY-R3 and R5-R7 under the matric suction of 20-500 kPa. It can be seen from Fig. 4.21 (b) that the difference in the PSDs is mainly due to those relatively large pores, i.e. inter-aggregate pores. Starting from the size of 7.5 μm, the PSDs do not differ very much for all the soil samples shown in this figure. This may explain the clear difference of the SWCCs of these samples tested under zero/non-zero stress conditions under the low suction range (1-20 kPa) but the similar soil-water characteristics from 20 to 500 kPa of matric suction.

However, more specific differences between ZY-R5, R6 and R7 cannot be clearly identified through MIP. It may due to the limited accuracy of the MIP technique at the present. On the other hand, this may be also helpful to explain the very similar looking of the SWCCs discussed in Chapter 3. It was stated in that chapter that except for the matric suction of 1 to 10 kPa, the desorption rate is more or less the same for all the soil samples, regardless of with or without stress applied.

4.4.4 Implied bi-model distribution

It can be noticed that the PSDs of ZY-R1 in Fig. 4.19 fits the bi-modal PSD as compared with Fig. 4.3. (Alonso et al., 1987; Atabet, 1991; Alonso, 1998). The amount of pores is large either in the range of 30-150 μm or smaller than 0.3 μm, but
the pores of 30-0.3 μm are limited. Therefore, for recompacted ZY samples, the pores of 150-30 μm and those smaller than 0.3 μm may be viewed as inter-aggregate and intra-aggregate pores, respectively. The proposed bi-modal distribution of soil pore spaces contributes to the double-structure framework for expansive soils later on (Alonso et al., 1995; Alonso, 1998). If the postulation of the natural sample, ZY-N1, is valid, the bi-modal can also be observed (Fig. 4.19(b)). Moreover, from the MIP results shown later, the bi-modal distribution can also be observed from other soil samples. Nonexpansive soils usually does not exhibit bi-modal distributions (Juang and Holtz, 1986; Lutenegger and Saber, 1987).

To have an overview of the MIP results presented, the curves expressed incrementally for all the samples are plotted together (Fig. 4.22). Except for ZY-N1 and LZ-R1, all the other sample have a considerable amount of large pores, from 15 to 150 μm. For LZ-R1, probably there is no such large pores in the sample due to its high dry density, while the large pores in ZY-N1 may not be detected by the MIP tests in this thesis. It can be inferred from that the total volume of intruded mercury is about the same for all the soil samples except for ZY-N1 (Fig. 4.22 (a)). All ZY samples, regardless of being intact or recompacted, the dry density is the same for each sample and the void ratio is also identical for every sample, therefore, the total intrusion should be about the same for samples with the same dry density.

From Fig. 4.22(b), it can be seen that the differences of the PSDs for different samples are mainly in the pore size ranges of 7.5-150 and 0.06-0.75 μm. It seems that the volumes of pores distributed in the middle size range are relatively limited. The bi-model distribution is clear for ZY-R1, with the right and left peaks referring to inter- and intra-aggregate pores, respectively. However, the intra-aggregate pores of the rest recompacted ZY samples, ZY-R5-R7, seem to be affected by external stresses, so
that the bi-modal distribution is not as clear as ZY-R1. ZY-N1 and LZ-R1 only shows a single mode, i.e. there is only one peak in the pore density in the range of intra-aggregate pores. The single mode in the range of large pores, 50-300 μm, was observed in some pure sand, reported by Juang and Holtz (1986). However, the single mode appearing in this study is in the range of intra-aggregate pores, 0.06-0.3 μm. The explanations are the same as stated in the above paragraph. Generally, most of pore spaces in expansive soils are inter-aggregate and intra-aggregate pores. For a same soil type, extrinsic factors, such as void ratio and soil density, can influence those large pores (7.5-150 μm), which can be viewed as inter-aggregate pores. Different soil types and stress affected soil samples will exhibit some clear difference in smaller pores (0.06-0.3 μm), which are intra-aggregate pores.

4.4.5 Prediction of SWCCs using MIP

MIP is sometimes recommended to be an alternative method for the direct measurement of SWCC. Because of the long period needed for completing a SWCC test, the more efficient method of the prediction from MIP become appealing. Some predictions made by Prapaharan and Altschaeffl (1985) showed that this method worked well for clay soils. The most latest work reported by Kong and Tan (2000) also proposed some empirical formula for prediction of SWCCs using MIP results and some laboratory test results on some expansive soils verified the validity of their proposed formula. The duration for completing one SWCC test is especially long for expansive soils, therefore, the research on the validity of the method is of much importance.
Therefore, the MIP results of several samples are selected and used to predict the SWCCs of these samples. The predicted and measured SWCCs are compared and the explanation for their difference is tentatively proposed. The original MIP data were expressed with the ratio of void space volume to soil sample volume, therefore, the water content can be calculated from the saturated volumetric water content subtracted by the intruded volume of MIP expressed with the ratio of the void volume to the total soil volume.

The SWCCs of LZ-R1, both predicted and measured, are shown in Fig. 4.23. The two curves have good agreements with some slight difference only. Generally the prediction gives a lower water retention ability of the soil than the measured data. LZ-R1 was tested under zero stress condition in a 5-Bar pressure plate extractor. The prediction is quite satisfactory, considering the advantages of MIP over SWCC tests. ZY-R7 was a sample which was tested under ISO stress of 100 kPa in the newly modified triaxial apparatus. The predicted and measured SWCCs of this sample are shown in Fig. 4.24. The desorption rate of the SWCCs under the matric suction from 10 to 500 kPa is almost the same, since the two curves look nearly parallel. But the difference from 1 to 10 kPa is significant. As mentioned before, there are a few cracks that formed after freeze-drying. These cracks may bring some errors to the large pores detected through MIP. This may be one of the possible causes for the discrepancy between the predicted and measured SWCCs shown in Fig. 4.24. Another cause might be the structure change due to stress. The measured SWCC was obtained under ISO stress condition during the SWCC test, while the predicted SWCC was from the MIP test, during which the stress was no longer applied on the sample. It may be postulated that some inter-aggregate pores may increase in volume after the stress is released. Therefore, the measured PSD may not be exactly the same with that during the SWCC
test and it is understandable that there is some difference between these two curves. The intra-aggregate pores may not vary due to the release of the external stress, so that the SWCCs under higher suction look similar for both the predicted and measured curves. From the discussion above, it seems that the prediction of SWCC from MIP does work in same cases, while the possible error caused by some extrinsic factors may sometimes be significant.

The comparison of the predicted and measured curves of ZY-R7 is shown in Fig. 4.24. They do not agree well for the low suction range, 1-10 kPa, but have similar characteristics under the matric suction beyond 10 kPa, since the two curves are parallel with each other. The samples of ZY-R1, R5 and R6 are also predicted and compared with their measured SWCCs.

The predicted SWCCs of ZY-R1, R5, R6 and R7 are shown in Fig. 4.25 along with the measured SWCCs. As for the pair of ZY-R1, the predicted and measured curves are different from 1-2 kPa suction, which can be caused by the cracks forming during the drying process, and become parallel in the suction range of 2-40 kPa. They have different desorption rates when the suction is higher than 40 kPa. Therefore, the prediction generally is not good.

As for the rest three samples, the main discrepancy is also in the low suction range, from 1 to 20 kPa, which again may be attributed to the drying cracks during the sample preparation. These cracks may bring some errors to the large pores detected through MIP. This may be one of the possible causes for the discrepancy between the predicted and measured SWCCs shown in Fig. 4.25. On the other hand, a part of the reason might be the structure change due to stress. The measured SWCC was obtained under ISO stress condition during the SWCC test, while the predicted SWCC was from the MIP test, during which the stress was no longer applied on the sample. It
may be postulated that some inter-aggregate pores may increase in volume after the
stress is released. Therefore, the measured PSD may not be exactly the same with that
during the SWCC test and it is understandable that there is some difference between
the two curves of a given sample in the low suction range (1-20 kPa).

For a given suction higher than 20 kPa, the desorption curves (one predicted and
the other measured) for a given sample are nearly parallel, which indicate two similar
PSDs that are measured from two different methods, namely, SWCC and MIP. The
intra-aggregate pores may not vary due to the release of the external stress and not
involve the potential error from drying cracks, so that the SWCCs under higher suc-
tion (from 20 to 500 kPa) look similar for both the predicted and measured curves.
From the discussion above, it seems that the prediction of SWCC from MIP does
work in same cases, while the possible error caused by some extrinsic factors may
sometimes be significant, which has an influence on the large pores (inter-aggregate
pores/ cracks).

4.4.6 Summary on MIP test results

    MIP is one of the techniques that quantify the PSDs of porous materials. In this
study, 6 MIP tests were carried out on 6 expansive soil samples whose SWCCs were
measured before. PSD can provide useful information on factors influencing soil fab-
ric and fabric-property relationships.

    Intact samples and recompacted samples do not have much different PSDs in
the range of 0.3-30 μm, which may explain the same desorption rate of both natural
and recompacted samples from about 35 to 500 kPa in measured SWCCs. From the
results, it can be inferred that there might be some large pores and natural fissure in a
natural sample, whose openings are larger than 150 μm and some void spaces in the range of 7.5-150 μm. However, due to the limitation of the equipment and probably due to some error involved in the test, they are not visible in the data presented, therefore the different desorption rates under the matric suction of 1-35 kPa of two natural and recompacted samples cannot be substantiated.

Different soils have different PSDs due to some intrinsic factors, such as soil origins, and extrinsic factors such as soil dry densities. The inter-aggregate pores are mostly governed by those extrinsic factors whereas the intra-aggregate pores only depend on the intrinsic factors, such as mineral compositions and geological history.

Stresses acting on soil samples during the measurement of SWCCs have influence on PSDs. The soil samples tested under stresses have more pores from 7.5 to 150 μm, which might evolve mostly due to physical reactions, from the compressed larger pores (greater than 150 μm), disintegration of large aggregates and rearrangement of particles and inter-aggregate pores. It can explain that the higher desorption rates of those samples measured under certain stress conditions. The slight difference in these PSDs from 0.3 to 7.5 μm can also explain that all the samples desaturate in the same rate in measured SWCCs, regardless of stress conditions, i.e. 1D and different ISO stresses. Compared with the PSD of the sample tested under zero-stress condition, external stresses cannot only affect the PSDs in the relative large pore size range, 7.5-150 μm, i.e. inter-aggregate pores but also in the small pore size range, i.e. intra-aggregate pores, 0.06-0.3 μm by some complicated chemical interactions and reactions between ions and clay minerals/particles. The difference of the effects on the PSDs in the intra-aggregate pore size range between different stress conditions is not clear.
The bi-modal distribution is also observed from the presented MIP results. Relative satisfactory predictions of SWCCs using MIP data are demonstrated by two selected samples. It is shown that without much error resulting from the relative large pores (most likely from cracks forming during sample drying), good prediction of SWCC can be made from the PSD information from MIP tests.

4.5 Notes for MIP and SEM tests

It should be noted that, as described by Delage and Lefebvre (1984), the sample should be first frozen and then freeze-fractured in order to obtain a smooth and flat surface for SEM tests. Even a little variation on the surface will be magnified greatly under a microscope and therefore, images cannot be focused accurately. However, in this study, this technique was not adopted. The special technique for obtaining a smooth and undistorted surface after cutting by a razor was also described by Mitchell (1993). This was again not used in the thesis. The results in later sections do not show much adverse effects on the image quality, by ignoring the special concerns. However, the author would like to suggest that these techniques be used for better test results of SEM and MIP tests.

The author would like to give several suggestions that extreme care should be taken during the freeze-drying process, in order to reduce the amount of cracks. Lower temperature and less time for freezing soil samples can reduce the potential cracks induced during the drying process. However, cracks brought about by drying can be by no means eliminated completely.

The potential error due to the release of applied stresses cannot be eliminated when dealing with those samples tested under non-zero stress conditions. It should be noted that the careful handling of samples in any step of the test procedure is crucial
to maintain a soil fabric as close to the soil fabric during the SWCC test as possible. If most of the external influences to the fabric change can be reduced to the least amount, good prediction of SWCCs may be made by MIP tests, which are more efficient than SWCC tests.

4.6 REFERENCES


Fig. 4.1 Fabric types (a) Clay matrix predominantly constituted by elementary particle arrangements of clay platelets (b) Microfabric of a clay predominantly made up of aggregations of elementary particle arrangements (c) Elementary particle arrangement in a parallel configuration (after Gens and Alonso 1987)
Fig. 4.2 (a) Fabric of the tested specimen  (b) pellet (after Alonso et al. 1995)
Fig. 4.3  Pore-size distribution in compacted FoCa clay (after Atabek et al. 1991)

ω – water content
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CHAPTER 5  3D NUMERICAL ANALYSIS OF GROUNDWATER RESPONSES IN AN UNSATURATED SLOPE SUBJECTED TO VARIOUS RAINFALL PATTERNS

5.1 INTRODUCTION

Rainfall induced landslides are one of the most common disasters and one of the major concerns in many areas of the world, including Hong Kong and Singapore (Lumb, 1962 and 1975; Brand, 1995; Rahardjo et al., 1998). An example that figures prominently in this study is a cut slope located at Lai Ping Road in Shatin, Hong Kong, which has failed several times over the past twenty years due to rainfall. The most notable instance was in July 1997. A massive slope failure occurred when several days of light intermittent rainfalls were followed by a heavy and prolonged rainstorm. A detailed 3-dimensional (3D) numerical investigation of the groundwater conditions of the slope was conducted by Tung et al. (1999) after the fact, since the subsurface groundwater flow was expected to have had a significant effect on the stability of the slope (Tung et al., 1999). The program used was the model of FEMWATER (Lin et al., 1997). FEMWATER is a 3D finite element computer model for simulating density-dependent flow and transport in variably saturated media. It has now been integrated into the US department of Defense Groundwater Modeling System (GMS) along with other groundwater related models. The governing equation for groundwater flow is given in Appendix V. The back-analysis was to explore the possible
mechanisms of the slope failure under rainwater infiltration. In the resulting report, groundwater responses under various rainfall conditions were presented with the discussion of the possible mechanisms for the landslide involved.

In general, with regard to infiltration problems and hence slope stability in unsaturated soils, permeability is one of the most influential factors on groundwater flow in unsaturated soils. A theoretical basis for modeling infiltration and groundwater flow came early on with the 1-dimensional (1D) Green-Ampt Model (Green and Ampt, 1911). Subsequent 2-dimensional (2D) analyses (Lumb, 1962; Pradel and Raad, 1993; Fourie, 1996; Sun et al., 1998; Ng and Pang, 1998) and a field observation (Lim et al., 1996) built on this early model. According to the Green-Ampt model, there will be a wetting front that continuously moves downward from the ground surface if the flux exceeds the water permeability. Apart from water permeability, rainfall parameters are also important to the study of infiltration and transient seepage analysis (Ng and Shi, 1998). In spite of Tung et al.’s report, 3-dimensional (3D) numerical analysis has nevertheless not received adequate attention up till now. Moreover, rainfall pattern has rarely been used as an important independent parameter to investigate groundwater response.

In this light, Tung’s report, which focused primarily on the landslide occurred in July 1997, seemed to call for and offer a starting point for a more general and systematic parametric study on rainfall infiltration and groundwater flow. It was expected that the results from a general study could be more widely applied to improve the understanding of the influence of rainfall patterns and other rainfall parameters on groundwater flow. Hence, a parametric study following up on Tung’s investigation was conducted and is presented in this chapter. The Lai Ping Road cut slope and FEMWATER were again used, with the mesh and all the inputs, including the initial
groundwater condition, being the same with the previous back analysis in Tung’s report. The use of this field-based information makes the results presented in this chapter more realistic and reliable.

Distinct rainfall patterns were the major concern of our 3D parametric study. The 3 most typical rainfall patterns were chosen from the total of 6 available for Hong Kong (Ting, 1998). Their influence on groundwater flow was then examined in detail. The influences of rainfall duration, intensity and return period were also investigated. The rainfall patterns were employed for both short intense 24-hr rainstorms and prolonged 168-hr rainfalls. 10-, 100- and 1000-yr were the three return periods used to investigate the influence of rainfall intensity.

The mesh was 3D, however, it should be noted that most results of groundwater response will be shown in a 2D cross section in this chapter to facilitate the illustration and interpretation of groundwater responses. Hence, pore-water distributions and hydraulic head distributions along two 1D sections, namely, the crest and the toe of the cut slope, of the 2D cross-section will be shown. Details on the cross-section selected will be addressed later.

5.2 3D FINE ELEMENT MESH AND BOUNDARY CONDITIONS

The finite element mesh and the geological conditions employed in this study are borrowed from the original 1999 analysis of the Lai Ping Road landslide (Tung et al., 1999). The finite element mesh was constructed based on the field geometrical conditions gathered from bore-hole information (Fig. 5.1). The dimensions of the site are also shown in Fig. 5.1. The site consists 3 different layers of soil. The first layer is colluvium and is 20 m thick on average and the second and third layers are rocks of 30 m
thick on average each layer, in which the third layer is more impervious.

After a process of discretization, a total number of 4005 finite elements with 2500 nodes were generated. The bottom of the mesh was assumed to be impermeable, whereas the ground surface is a flux boundary receiving rainwater infiltration. The boundary along the perimeters AB and BC in Fig. 5.1 was set as a zero-flux boundary, whereas the perimeter AC was set as a fixed-head boundary governed by the water level in a stream along the perimeter.

In the present study, isotropic and heterogeneous water permeability is assumed. The soil-water retention curves and the permeability with respect to matric suction are shown in Fig. 5.2 (a) and (b), respectively. The saturated water permeability of each of the three layers from top to bottom is 10E-5, 10E-7, 2.8E-9 m/s, respectively (Tung et al., 1999). Because the soil slope has already experienced seasonal changes many times, no hysteresis is assumed between the drying and wetting paths in hydraulic properties (Topp and Miller, 1966; Ng et al., 2000). Moreover, the program is based on another important assumption that there is no deformation caused by the variations of pore-water pressure.

5.3 RAINFALL AND INITIAL STEADY STATE CONDITIONS

As seen from Fig. 5.3(a), totally there are 6 rainfall patterns available, i.e. A1, A2, C, D1, D2, U. Dimensionless rainfall intensity is used instead of real rainfall intensity in the plots. Real rainfall intensity can be obtained by multiplying the dimensionless rainfall intensity by the total rain depth. Each rainfall pattern is a generalized and idealized pattern based on the observation record in Hong Kong for the past one hundred years and represents for a particular temporal distribution of rainwater (Ting, 1998).
Patterns A1 and A2 can be classified as advanced types. Pattern C is the central type while Patterns D1 and D2 are two delayed types. The distribution of Pattern A2 is relatively uniform with respect to time. Only Pattern A2, C and D1 will be examined in detail in this chapter for reasons that will be given later.

In this chapter, firstly, the 24-hr rainfalls were used to illustrate the effects of rainfall patterns on the groundwater response in the slope. The amount of rain from a 24-hr rainfall is 358, 597 and 805 mm, corresponding to return periods of 1-in-10-, 100- and 1000-yr, respectively (Ting, 1998). Prolonged 168-hr rainfalls of the three rainfall patterns, namely, Patterns A2, C and D1, were adopted to investigate the effect of different rainfall durations (Fig. 5.3(b)). The selected return period and its corresponding rain depth are 100 yr and 892 mm, respectively (Ting, 1998). It can be seen that the intensity is a constant within a given interval, i.e. 1/12 of the rainfall duration. Therefore, the interval is 2 and 14 hr for 24- and 168-hr rainfalls, respectively (Ting, 1998). It should be noted that prolonged rainfalls are less intense than 24-hr rainfalls for a given return period, although the total rain depth is higher for prolonged rainfalls. With a lower intensity and longer duration, a prolonged rainfall results in a more even distribution of rainwater.

It should be noted that the flux applied on the boundary in the model is not the exact value of rainfall intensity. Real infiltration in the field is very complicated. Its rate is a function of time under constant rainfall intensity (Mein and Larson, 1973; Wilson, 1997; Hillel, 1998) and it decreases with time, finally approaching saturated permeability. However, it is not feasible to simulate these field phenomena using numerical analysis since a number of factors are involved, e.g. surface runoff and evaporation. Instead, an average value of 60% of the rainfall intensity was adopted for the net surface flux applied to the slope (Tung et al., 1999).
24-hr rainfalls were used to study the effect of rainfall and return periods, while 168-hr rainfalls were compared with 24-hr rainfalls with the same return period to study the effect of rainfall duration. The inputted rainfall conditions is listed in Table 1.

All the simulations were conducted from a same initial groundwater condition. This was a steady state of groundwater condition that was obtained by applying a very small flux of 10 mm/day to the flux surface for about 100 days. The groundwater table is on average 20 m in the upstream and 2 m deep in the downstream of the slope. Groundwater flows in a direction nearly parallel to the inclination angle of the slope surface. Details about the initial groundwater condition will be provided later.

5.4 RESULTS AND DISCUSSIONS

5.4.1 Preliminary study

As stated above, the initial groundwater condition was a steady state obtained by applying a small flux on the ground surface for a long time. The entire mesh is shown in Fig. 5.4, with OO’ being the cross section selected for presenting computed results later. The flow vectors throughout the mesh for the initial state are shown in Fig. 5.4. It should be noted that the vectors in the high flow rate area in Fig. 5.4 (a), where the landslide occurred, are generally in the direction with the major displacement of the landslide. As seen from Fig. 5.4(b), the direction of the vectors is also generally parallel to the slope inclination angle and the low permeability of the bottom two rock layers makes flow in these rock layers almost negligible, as indicated by the very small vectors in these two layers.
The contours of pore-water pressure head along the cross section OO' corresponding to the initial state are shown in Fig. 5.5. It can be seen that the heads range from about −5 to 70 m across the depth of the section. XX’ and YY’ are 2 cross sections located at the crest and the toe of the cut, having ground surface elevations 136.6 mPD and 100.4 mPD respectively. Contours are generally parallel to each other and nearly parallel to the slope inclination angle from section YY’ to downstream. This implies that the groundwater condition is nearly hydrostatic in this area. However, for the area from section XX’ to upstream, the contours near the ground surface are not parallel to the slope inclination angle and there is a high suction zone in upstream, which implies the groundwater condition is not hydrostatic. The initial locations of the groundwater tables for the XX’ and YY’ sections are at the depths of 19.45 m (117.15 mPD) and 1.7 m (98.7 mPD), respectively. These two sections are selected to present simulation results because they can be used to illustrate different groundwater responses due to an initial deep groundwater table or shallow groundwater table. Their respective initial suctions at the ground surface on are 53 kPa and 15 kPa, which are within the common range of measured surficial matric suction in slopes in Hong Kong. The validity of the computed initial condition was verified by imposing field monitoring data (Tung et al., 1999). As shown in Fig. 5.5, the measured groundwater table at bore-hole TT2 which was near to Section XX’ was 19.5 m in May, 1998. This was very close to the computed groundwater table of 19.45 m for Section XX’.

A series of preliminary simulations to investigate the effects of different rainfall patterns were carried out. 24-hr rainfalls with a 10-yr return period were adopted for shortening the simulation time. From output files, only the pore-water pressure heads \( p \) at sections XX’ and YY’ can be retrieved, while the hydraulic heads \( h \) will also be used later. Hydraulic head \( h \) is the summation of pore-water pressure head \( p \) and ele-
vation head $z$. The variation of the hydraulic heads actually reflects changes in pore-water pressure because elevation heads are assumed to be constant. Because the trends of the variation of hydraulic head under the ground are similar to those at the ground surface at both sections, only the hydraulic head at the ground surface with time is plotted (Fig. 5.6(a) & (b)). Patterns A2 and D1 seem to be the two extreme cases because their plots seem to envelop the other patterns. Their pressure changes are paralleled closely by their respective rainfall patterns. The soil is wetted most rapidly under Pattern A2 and most slowly under the rainfall of Pattern D1, as illustrated by the fact that the peak intensity occurs at the very beginning of Pattern A2 whereas most of rainwater is distributed towards the end of Pattern D1. Consequently, these two extreme patterns were selected. Pattern C was also selected because it behaved like a median case. Thus the rainfall patterns used represent advanced, central and delayed patterns.

A total of 12 simulations were carried out (Table 1). They can be grouped into 4 series, shown as 4 columns in Table 1. In each series, totally there are 3 cases, namely, Patterns A2, C and D1. For each series, the rain depth is the same for a given return period, regardless of rainfall pattern. The first three series are short (24-hr) and intense rainfalls. The return periods are 10, 100, 1000-yr return periods, respectively. The higher the return period is, the greater the total rain depth will be. These three series of tests are aimed at investigating the effects of rainfall patterns and return periods on groundwater responses. The fourth series is composed of long (168-hr) rainfalls with a return period of 100-yr. The prolonged rainfalls have a greater rain depth than 24-hr rainfalls, but they are less intense than the short rainfalls. By comparing the second series and the fourth one, the influence of different rainfall duration can be studied.
5.4.2 Typical groundwater response to rainwater infiltration

Pore-water pressure distributions along the depth of sections XX' and YY' are shown in Fig. 5.7 for the Pattern C rainfall of a 24-hr duration with a 100-yr return period, in other words a median rainfall pattern with a median return period. The figure illustrates a typical groundwater response in an unsaturated slope subjected to rainwater infiltration.

At section XX', the initial matric suction around 50 kPa is nearly constant in the top 6 m. The initial groundwater table is located at the depth of 19.45 m. As the rain continues for 12 hr, the matric suction of the top 6 m is reduced. The matric suction at the ground surface decreases to 10 kPa and the soil is wetted to a depth of about 10 m at the end of the 24-hr rainfall. It can be seen that rainwater infiltration reduces the matric suction gradually, but a clear continuously intruding wetting front is not observable. 168 hours after the cessation of rainfall the suction up to 6 m below the ground surface has recovered. The matric suction in the top 1 m of soil even exceeds the initial value at the ground surface. The pore-water pressure continues to increase in deeper soils from 6 to 20 m underground during this period of no rainwater infiltration. At section XX', no rise in the groundwater table can be seen during the rainfall. It rises by about 0.5 m to about 19 m 168 hr after the rainfall ends as water continues to flow down into the deeper soils.

From Fig. 5.7, it can also be seen that the initial groundwater table is 1.7 m below the ground surface at section YY', the toe of the cut slope. The pore-water pressure distribution along depth is nearly linear, which is quite different from that at section XX. At 12 hr after the rain, the matric suction in 0-4 m is obviously wetted. Matric suction at the ground surface decreases to zero. In other words, the groundwater table rises to the ground surface. At the end of the rainfall, there is a general rise in pore-
water pressure along the whole depth. At 168 hr after the cessation of the rainfall, the pore-water pressure generally decreases but does not return to its original condition. The groundwater table is about 1.1 m deep. In general, the magnitude of variations in pore-water pressure at section XX' decreases with depth, whereas the magnitude of increase in pore-water pressure for section YY' is almost constant regardless of the depth, seen from the fact that the plotted lines are parallel. This is because that unlike section XX', rainwater infiltration only affects matric suction but not the depth of the groundwater table, infiltration at section YY' brings about a clear rise in the main groundwater table. Correspondingly, there is an obvious increase in pore-water pressure along the whole of section YY' can be seen from the figure. Again, the advance of a wetting front cannot be observed in this section either. It may be said that the model of 'wetting front', which was originally proposed for 1D problems and later on widely adopted in 2D analyses, may not be applicable for the 3D analysis.

The water permeability at section XX' is lower than that at section YY', since the matric suction at section XX' is much higher than that at section YY'. When the soil near ground at section XX' is wetted by the infiltration, the suction in the deeper soil is still high, so the permeability at section XX' decreases with depth. This results in a less significant water flow downwards at section XX', whereas a considerable amount of water in the soil near the ground surface flows towards section YY' and downstream. Apart from the water flowing from upstream, the soil at section YY' also receives a large amount of rainwater from the ground surface, because of the high water permeability. As a result, the groundwater table at section YY' rises obviously whereas there is only a slight rise of the groundwater table at section XX'.

The above discussion generally shows the groundwater table does not rise greatly, though we are dealing with a 1-in-100-yr heavy rainfall. Nonetheless, the reduction of
matric suction in shallow soils is obvious. As stated by Fourie (1996), many observed slope failures were not caused by an evident rise in the groundwater table but rather could be attributed to the migration of a wetting front into the slope. Infiltration of rainfall may be sufficient to reduce the matric suction in the surficial soil substantially enough to trigger a shallow failure (Fourie et al., 1999). Therefore, a 24-hr rainfall of Pattern C with a 100-yr return period may trigger a landslide if the reduction of matric suction could reduce the FOS of the soil slope to be less than 1.0.

5.4.3 Effects of various rainfall patterns on groundwater response

For the rainfall patterns A2, C and D1, 24-hr rainfalls with 3 different return periods were used, but because the general trends are similar for all three return periods, the rainfalls of the median value, 100-yr, were used to illustrate the effects of the rainfall patterns on the groundwater flow in the slope. The total rain depth for each rainfall is the same, i.e. 597 mm.

5.4.3.1 Groundwater responses to different rainfall patterns at sections XX' and YY'

Because there is almost no significant change in groundwater flow in the deep rock layers, only three shallow depths, 0, 6.1 and 18.3 m, were selected to show the variation of hydraulic heads. An increase in the hydraulic head implies an increase of pore-water pressure head. Thus later we will sometimes mention pore-water pressure directly instead of hydraulic head.

As shown in Fig. 5.8(a), the initial hydraulic heads at the three depths are less than the elevation heads, which implies that the pore-water pressures above the depth of 18.3 m are initially negative before it starts to rain. The pore-water pressure at the ground surface rises up immediately after the rainfall starts. It rises most rapidly and
most significantly in response to the advanced Pattern A2 and soon reaches its peak, followed by responses to the central type Pattern C and then the delayed type Pattern D1, which parallels to the sequence of peak rainfall intensity for these patterns. Thus the hydraulic head in the first 12 hr is the highest for Pattern A2 whereas at the end of the rainfall, it is the highest for pattern D1. At the depth of 6.1 m, the pore-water pressure also rises most significantly for Pattern A2 than other two patterns. The difference between Pattern A2, which has a pore-water pressure head about 2 m above the other two patterns at the end of the rainfall. No increase in pore-water pressure can be seen at the depth of 18.3 m at any time during rainfall.

After the cessation of the rain, the pore-water pressure at the ground surface (0 m) drops for each pattern, with the highest pore-water pressure being for Pattern D1 and the lowest for Pattern A2. But underground, the pressure continues to rise. For the depth of 6.1 m, it can be seen from Fig. 5.8(a) that the pore-water pressures of all three patterns reach their peaks 42 hr after the cessation of rainfall. The difference between the patterns tends to be minimal thereafter and 168 hr (1 week), later, nearly no difference in hydraulic head can be detected. However, at the greater depth of 18.3 m, the pore-water pressure increases for all patterns as they approach the 168hr mark after the cessation of rain, and in particular for Pattern A2. The pore-water pressure head for Pattern A2 rises about 2 m, which is twice the increase for Pattern C or D1. It should be noted that the initial groundwater table is 19.45 m, which is very close to the depth of 18.3 m. An increase of 2 m in pore-water pressure head at the depth of 18.3 m actually reflects a rise of 2 m in the groundwater table compared with the initial state. Therefore, the groundwater table of the slope rises up about 2 m when subjected to the Pattern A2 rainfall with a 100-yr return period, and about 1 m when subjected to Patterns C and D1.
Fig. 5.8(b) illustrates the groundwater response at Section YY’. Again, the variation of the hydraulic head is plotted over time. It should be noted that Fig. 5.8(b) is plotted in the same scale as Fig. 5.8(a) for ease of interpretation. The variation of hydraulic head, reflecting pore-water pressure head actually is generally much smaller at Section YY’ compared with Section XX’. The hydraulic head at the ground surface reaches 100.4 m 6 hr after the Pattern A2 rainfall starts. The hydraulic heads at the ground surface of the Pattern C and D1 rainfalls reach 100.4 m at 12 and 24 hr, respectively. It should be noted that the elevation of the ground surface of section YY’ is 100.4 m. When the hydraulic head at the ground surface reaches 100.4 m, it means that the groundwater table at section YY’ has risen to the ground surface. In other words, a seepage surface forms around the section YY’ at that time. At the depth of 7.3 m, hydraulic heads also rise most quickly for rainfall Pattern A2, with a difference of up to 2 m between the initial condition and the condition after 18 hr. It should be pointed out that from 10 to 24 hr, the hydraulic head at 7.3 m for Pattern A2 exceeds the hydraulic head at the ground surface. It necessarily implies an upward flow during that period, from underground to the ground surface at the toe of the slope. At the depth of 23.4 m, the curve representing Pattern A2 has the greatest increase among the three patterns during rainfall. The reason for this phenomenon may be that more water infiltrates the upstream for pattern A2, as we can see that the highest pore-water pressure is at section XX’, and consequently a greater amount of groundwater flows from the upstream to the toe of the slope for Pattern A2 than for other two patterns. As a result, the pore-water pressure along the depth is also the highest for Pattern A2. This postulation is easily validated by inspecting the velocity vector field of the cross-section of OO’. It can be found that the majority of flow vectors are downward in the same direction of the inclination angle of the slope.
After the rainfall ends, the hydraulic heads decrease at the ground surface for Patterns C and D1, but not for Pattern A2. For Pattern A2, the hydraulic head at the ground surface remains at 100.4 m, even after one week, which means the groundwater table remains at the ground surface for this long period of time. This can be attributed to the great amount of groundwater that flows downstream from upstream after the rainfall, as stated before. At 192-hr, the difference of pore-water pressure head at the ground surface between Pattern A2 and other two patterns, which are really identical, is about 1.5 m. A similar trend can be seen underground, with Pattern A2 consistently inducing the greatest change in pore-water pressure among the three.

Only looking at the pore-water pressure changes in the slope subjected to the 24-hr rainfalls, Pattern A2 seems to be more adverse on slope stability, since it induces higher pore-water pressure underground than the other two patterns, not only at the crest but also at the toe of the slope. But it should be noted that this is only applicable for the short (24-hr) rainfalls and may not also be true for the prolonged rainfalls.

5.4.3.2 Role of hydraulic gradient

For a 1D problem, the rainwater infiltration rate is governed by the water permeability of surficial soil and hydraulic gradient, by applying Darcie’s Law,

\[ v = k_w \cdot i = k_w \cdot \frac{\partial h}{\partial z} \]

\( v \) is the flow rate (at the ground surface, it is infiltration rate). \( k_w \) is the water permeability for a given matric suction, which will become \( k_{sat} \) when the soil is fully saturated. \( h \) is the hydraulic head (total head). \( z \) is the elevation (elevation head) and \( i \) is the hydraulic gradient along the z-direction, since the hydraulic gradient is defined as \( \frac{\partial h}{\partial z} \). The elevation head gradient is always equal to +1 and the pore-water
pressure head gradient is equal to -1 under a hydrostatic condition. Under such a condition, the hydraulic gradient is equal to 0, which indicates a hydrostatic condition, so there is no groundwater flow in the vertical direction. The hydraulic head distribution in a vertical cross section will simply be a vertical line. If pore-water pressure above the groundwater table is greater than the hydrostatic condition, the hydraulic gradient is positive and the hydraulic head increases as the depth decreases. Under that condition, there is a downward flow from the ground surface to the soil underground. The gradient is negative when groundwater flows upwards. Hydraulic gradient is one of the most important indices for groundwater flow.

The rainwater infiltration rate is not a constant under a rainfall of constant intensity. According to the model proposed by Mein and Larson (1973), the rainwater infiltration rate is the highest at the beginning of a rainfall and then decreases as the rainfall continues. The highest infiltration rate is caused by the high hydraulic gradient at the beginning of a rainfall, when the soil is partially saturated, i.e. when the surficial matric suction is relatively high (Wilson, 1997). As the rainfall starts and continues, the soil near the ground surface is wetted gradually, i.e. the matric suction is reduced by infiltration, and the water permeability of the soil approaches saturated permeability. As a result, the infiltration rate finally approaches a steady state and is equal to saturated permeability and the hydraulic gradient near the ground will approach 1. When rainfall intensity is lower than the infiltration rate, all rainwater can infiltrate into soil. When rainfall intensity is higher than the infiltration rate, only some of the rainwater infiltrate and excessive rainwater will become surface runoff.

Hydraulic heads along the depth at both sections XX’ and YY’ at 6- and 24-hr are shown in Fig. 5.9(a) and (b) respectively. It can be seen from Fig. 5.9 (a) that the ini-
tial hydraulic gradient \( \frac{\partial h}{\partial z} \) along the vertical direction above 12 m underground at sec-
tion XX' is greater than zero but nearly equal to zero below 12 m. 6 hr after the influ-
ence of the rainwater infiltration of Pattern A2 has reached as far as 8 m deep, twice
as deep as other two patterns. The hydraulic gradient is clearly higher for Pattern A2.
This must be attributed to the high rainfall intensity at the beginning of rainfall Pattern
A2. The resulting higher gradient helps the infiltrated water flow at a higher rate,
which in turn saturates the soil more quickly than the other two patterns. The resulting
higher water permeability leads to higher flow rate and a higher pore-water pressure
along the depth at section XX'.

The line of the initial hydraulic head at section YY' is nearly vertical, which im-
plies a nearly hydrostatic groundwater condition. The hydraulic gradient is almost
zero at this section. At Section YY', the groundwater table is initially at a depth of 1.7
m and the matric suction near the ground surface is low, while the water permeability
is high. Although the gradient is small, the water permeability is relatively high, there-
fore, there still can be significant infiltration at section YY'. Moreover, the groundwa-
ter flow from upstream to downstream is significant (see Fig. 5.4(b)). 6-hr into the
rainfall, the hydraulic head along depth is increased significantly by rainfall Pattern
A2, whereas the hydraulic heads of other two patterns are nearly the same as the initial
state. This can be ascribed to not only the higher infiltration rate at this section but
also higher water flow from upstream for Pattern A2. Ultimately, it is attributed to the
higher rainfall intensity of Pattern A2 during the first several hours of rainfall.

Fig. 5.9(b) shows the hydraulic heads along depth 24-hr into the rainfall, i.e. at the
end of the rainfall. At section XX', the influence of rainwater infiltration for Pattern
A2 has extended to a depth of about 12 m, but only to around 7 m for Patterns C and
D1. Since the hydraulic head at section XX’ of Pattern A2 is the highest, which means the pore-water pressure is the highest, the permeability should be the highest for Pattern A2 in the three patterns from a depth of 1 to 12 m. Within the top 4 m, the hydraulic gradient of Pattern A2 is slightly lower than Patterns C and D1, mainly the rainfall intensity of Pattern A2 is lower than other two patterns. Below 4 m, the hydraulic head and gradient are both higher for Pattern A2, because of the greater water flow for Pattern A2 in the first several hours. The matric suction at the ground surface reflects the rainfall intensity simultaneously, whereas there is a time delay between the variation of pore-water pressure underground and the flux applied on the boundary. That is why the hydraulic head is the highest for pattern D1 at the ground surface at the end of the rainfall while it is still the highest for Pattern A2 underground. As a result, the flow rate in the soil may still be higher for Pattern A2 rather than Pattern D1.

At section YY’, the hydraulic heads along depth for all three patterns are higher at the end of the rainfall than 6 hr into rainfall. Compared with the initial line, the three lines move further to the right. This implies a general rise in the groundwater table at section YY’ due to rainwater infiltration for each pattern. A clear difference in hydraulic head distribution can be seen between the patterns. A general increase of about 2 m in the hydraulic head along depth occurs for Pattern A2, whereas the increases are less than 1 m for other two patterns and the hydraulic head for Pattern D1 is the lowest.

With the top 4 m, the line for Pattern A2 has a slight negative slope angle, which implies an upward flow. Therefore, there must be an upward flow at section YY’ subjected to rainfall Pattern A2. This is consistent with Fig. 5.8(b) where from 10- to 24-hr, the hydraulic head at 7.3 m underground is higher than that at the ground surface. Meanwhile, it can be noticed that the slope angles of the lines for Pattern D1 and C
are clearly positive and the angle for Pattern D1 is slightly greater than that for Pattern C. Therefore, for these two cases, water flows downwards at section YY'.

The higher hydraulic head and the upward flow for Pattern A2 are due to the higher infiltration rate during the first several hours, which can be attributed in particular to the extraordinarily high intensity in the first several hours of this rainfall pattern. The source of the upward flow must be the water flowing from upstream. Because a certain amount of time is needed for water to flow from one place to another, the upward flow for Pattern A2 cannot be observed 6 hr into rainfall (Fig. 5.9 (a)). The difference in rainfall patterns at the beginning of the rainfall patterns also affects the groundwater condition later on. For Pattern D1, a significant amount of rainwater infiltration occurs at the end of the rainfall, while the pore-water pressure underground is low due to its low intensity before, which results in a relatively greater hydraulic gradient. Thus, a relatively significant downward flow occurs at section YY' at the end of the Pattern D1 rainfall. Pattern C is a median case, so its pore-water pressure distribution is right in the middle of the pore-water pressure distributions of Patterns A2 and D1.

5.4.3.3 Role of q/k_w

The importance of q/k_w, the ratio of applied boundary flux to water permeability, can be seen from Equation 6 in Appendix V. Though q/k_w is not the only factor governing the variations of pore-water pressure in the equation, it can still be used to examine the influence of rainfall patterns on groundwater responses. Variations of the calculated instantaneous q/k_w ratio during a 24-hr rainfall for each pattern are shown in Fig. 5.10, which represents a 100-yr return period rainfall. In this chapter, the net
applied flux $q$ at the surface boundary is assumed to be 60% of the total rainfall intensity. $k_w$ is retrieved from the input permeability function as shown in Fig. 5.2(b) with calculated pore-water pressure from the program. When the ratio is high, it is difficult for all the water to infiltrate the soil.

From Fig. 5.10, it can be seen that the starting point of Pattern A2 at section XX' is about 200, high above that of other two patterns, which is only 30. This is because of the high surface flux for Pattern A2, caused by its high initial rainfall intensity. As it starts to rain, the $q/k_w$ of Pattern A2 drops sharply, which reflects the rapid wetting of the surficial soil by the rainwater infiltration wets. The ratio becomes lower than unity after 5 hr of rainfall. For Patterns C or D1, $q/k_w$ of section XX' also decreases but more slowly, which means that the rainfall also saturates the soil but more slowly. As the rainfall continues, $q/k_w$ of Pattern C drops down in the middle of the rainfall duration and becomes smaller than unity after about 15 hr of rainfall, when the intensity of rainfall is at its peak. For Pattern D, $q/k_w$ does not decrease significantly and is still higher than unity at the end of the rainfall, when rainfall intensity is at its peak. Therefore, it can be inferred that the decrease in this ratio is mostly governed by rainfall intensity. When the rainfall intensity is high, the ratio decreases sharply. It can be seen that high rainfall intensity brings about the reduction of the ratio of $q/k_w$, because the soil is wetted gradually and water permeability is increased by rainwater infiltration. But the pace of wetting varies with rainfall intensity, the higher the intensity is, the more rapidly the water permeability is increased. Therefore, the ratio of Pattern A2 drops most quickly, followed by Pattern C and D1 in turn. Since $q/k_w$ of Pattern A2 is mostly lower than unity and that of Pattern D1 is above unity throughout the rainfall, it is possible that the total infiltration is the highest for Pattern A2 and the lowest for Pattern D1, with C in the middle.
At section YY', the starting point of q/k for Pattern A2 is much lower than at section XX', since the initial water permeability is much higher at section YY' than at section XX'. It is also obviously reduced by infiltration in the first 4 hr but remains nearly constant after that due to nearly the constant rainfall intensity and the complete saturation of the ground after 4 hrs. The water permeability at section YY' cannot increase as significantly as that at section XX' due to its initial high permeability. Since the variation of permeability at section YY' is not very significant, rainfall intensity becomes the dominant factor. Thus, as seen from Fig. 5.10, the dashed lines follow the trend of their respective rainfall patterns very closely, i.e. their temporal variation of rainfall intensity. In addition, the difference between patterns is not as great as the difference at section XX', the crest of the slope. It can be postulated that this clear difference in pore-water pressure head is mostly caused by downward flow from the crest, where the differences in infiltration rate and in turn permeability among the patterns are more significant.

Generally, the ratio for section XX' is above unity, whereas it is mainly around unity for section YY'. It is verified that the infiltration rate will approach saturated permeability when the soil is wetted. Differences in the behavior of q/kw can explain why the groundwater responses at the two sections are considerably different. It is mainly because of their initial surficial water permeability, since initially the groundwater table is high at section YY' but very deeply seated at section XX'. The importance of q/k, has already been assessed by Kasim et al. (1998), where q is the rainfall flux and k, is the saturated coefficient of permeability. q/kw must be important in groundwater flow, which has also been discussed by Ng and Shi (1998). However, the variation of q/kw in the field is very complicated, even under constant rainfall intensity. In this study, too many variables, such as constantly changing rainfall intensity
and unsaturated water permeability, are included at the same time, making the problem more difficult to interpret. Therefore, the above discussion can only help to visualize the variation of this crucial ratio during rainfall but not to understand its role fully.

5.4.4 Effect of different return periods

A higher total rain depth corresponds to a higher return period. The rainfall intensity is thus higher for a higher return period rainfall. In this study, the rainfall depths are 398, 597 and 805 mm for the 24-hr rainfalls of 10-, 100- and 1000-yr turn periods, respectively.

Pore-water pressure distribution along depth at section XX’ for the Pattern A2 rainfall with return periods of 10-, 100- and 1000-yr are shown in Fig. 5.11. Since rainfall intensity is at its peak in the first several hours of rainfall in this pattern, the 6-hr mark of the rainfall is chosen to show the groundwater responses during the rainfall, while the 192-hr mark is used to present the groundwater condition long after rainfall. At 6-hr into the rainfall, the top 10 m soil is wetted. It can be seen that the matric suction at the ground surface under the 1-in-10-yr rainfall is about 29 kPa, reduced by 24 kPa, whereas the 1-in-100- and 1000-yr rainfalls reduce it to zero at the ground surface. The curves of the two rainfalls with higher return periods are exactly overlapped. At 192-hr, one week after the cessation of rainfall, the difference again cannot be observed between the two rainfalls with 100- and 1000-yr return periods. That means when the return period goes beyond 100-yr, no more increase in pore-water pressure underground can be caused for this pattern. It is also valid for the response at section YY’ subjected to a rainfall of Pattern A2, which is therefore not shown. As for Pattern C, there is some difference between the pore-water pressure distributions at section XX’ at a given time between 1-in-100- and 1000-yr rainfalls, but this difference is
much smaller than that between 1-in-10- and 100-yr rainfalls. For Pattern D1, the trend lies between the above two patterns, but closer to that of Pattern A2, i.e. the difference between 1-in-10- and 100-yr rainfalls is much more significant than that between the 1-in-100- and 1000-yr.

There may be some limiting value of a rainfall return period, for this study. Exceeding that value, an increase in rainfall return period, actually an increase in rainfall intensity, can no longer induce a significant increase in pore-water pressure underground. For example, the value is 100 yr in this study. The reason for this is that increasing pore-water pressure is due to infiltration and the infiltration rate is in turn determined by water permeability and hydraulic gradient. Water permeability has its own limit, i.e. saturated permeability. Although rainfall intensity is higher for a rainfall with a higher return period, not all of the water can infiltrate the soil, so no greater difference in groundwater response can be induced. The variation of $q/k_w$ is shown in Fig. 5.12. For section XX’, the crest of the cut, the ratios of $q/k_w$ are initially proportional to their respective return periods. The ratios of $q/k_w$ of the 1-in-100- and 1000-yr rainfalls drop more rapidly than that of the 1-in-10-yr rainfall, because higher intensity reduces matric suction and increases water permeability more quickly. Since water permeability under 1-in-10-yr rainfall is much lower than the other two cases, $q/k_w$ is in a higher position in the figure. Under 1-in-100- and 1000-yr rainfalls, the ground surface is completely saturated, so there is no difference in the surficial permeability for the two cases. Thus, the ratio for the 1-in-1000-yr rainfall is in a higher position than that for 1-in-100-yr rainfall only because of its higher rainfall intensity. As a result, the ratio goes below unity for both 1-in-100- and 1000-yr rainfall at about 5 hr into the rainfall, whereas the ratio for the 1-in-10-yr case is always higher than unity. As seen from the plot, the two curves of 1-in-100- and 1000-yr rainfalls are very close
to each other, which may result in a smaller difference in groundwater response. The ratios at section YY' closely follow their respective return periods.

5.4.5 Effect of rainfall duration on rainfall patterns

In this section, the comparison between 24-hr and 168-hr rainfalls with a 100-yr return period will be presented for the three different rainfall patterns. It should be noted that the rain depth of 168-hr rainfalls for a 100-yr return period is 892 mm, 1.5 times that of a 24-hr rainfall with the same return period, 597 mm. The intensity of the prolonged rainfall at a given time is therefore about 1/5 of the intensity of the short rainfall. This implies that rainwater is relatively more uniformly distributed in prolonged rainfalls than in short and intense ones.

The variations of hydraulic head at three different depths are plotted in Fig. 5.13(a)-(c) separately. In Fig. 5.13 (a), the general trend of the variations of hydraulic heads at the ground surface is similar to that for 24-hr rainfalls. The pore-water pressure of Pattern A2 reaches its peak at 42-hr and the pore water pressures of Patterns C and D1 reach their respective peak pressures at 84-hr and 168-hr. The peak of the pore-water pressure head at the ground surface coincides with the peak intensity of each rainfall. Compared with 24-hr rainfalls, the magnitude of variation for pore-water pressure in the ground surface during rainfall is generally slightly higher for 168-hr rainfalls, which means a higher intensity can cause a greater increase in pore-water pressure at the ground surface.

For the 24-hr rainfalls, the peak pore-water pressure at the ground surface of Pattern A2 is clearly higher than those of the other two patterns, whereas the difference in the magnitude of peak pore-water pressure between various patterns is relatively smaller for the 168-hr rainfalls in Fig. 5.13. It seems that there is only a time-lag be-
tween the variations of groundwater responses induced by different patterns. For instance, in Fig. 5.13 (b), the peak intensity of the Pattern A2 rainfall is at 42 hr into rainfall and the peak pore-water pressure at the depth of 6.1 m occurs at 84 hr into rainfall, while the peak intensity of the Pattern C rainfall is at 84 hr into rainfall and the peak pore-water pressure occurs at 126 hr into rainfall. So there is a time lag between the peak intensities of the two rainfalls, which is 42 hr and is exactly equal to the time lag between the peak pore-water pressures at the depth of 6.1 m for the two rainfalls. The peak value of pore-water pressure for each pattern is the same but occurs at a different time. It can be clearly seen that the pore-water pressure for the 24-hr Pattern A2 rainfall is the highest amongst the three 24-hr rainfalls. It means that rainfall pattern has clear influence for short duration rainfalls, but has relatively less influence for prolonged rainfalls, except causing some difference temporally. Moreover, the peak pore-water pressures for the 168-hr rainfalls is the same with the peak for the 24-hr Pattern A2 rainfall, but obviously higher than those for other two patterns with 24-hr duration, which means that the prolonged rainfalls can generally induce a greater increase in pore-water pressure underground.

In Fig. 5.13 (c), at the depth of 18.3 m, the pore-water pressures starts rising at different times thus at the end of the rainfall, i.e. 168-hr, the Pattern A2 rainfall causes the greatest pore-water pressure change. This also implies the groundwater table under this pattern of rainfall rises most significantly, according to the previous discussion for 24-hr rainfalls. After the rainfall terminates, the pore-water pressure from the ground surface to the depth of 6.1 m decreases while it keeps on rising at the depth of 18 m. In Fig. 5.13(a)-(c), it can be noticed that the pore-water pressures for all patterns tend to converge as time elapses beyond the moment at which infiltration occurs. At the depth of 18.3 m, pore-water pressure heads rise with the same rate, since it can be
seen that the three curves are nearly parallel to each other from 168- to 300-hr. At the 192-hr mark, the difference in the pore-water pressure heads of two neighboring patterns is only about 0.5 m, which is smaller than the 1 m difference for 24-hr rainfalls. Their difference becomes smaller at 336-hr (one week after the cessation of rainfall) and they converge at the end of the simulation, i.e. 408-hr, which means that different rainfall patterns cause no difference in a deeply-seated groundwater table 10 days after the cessation of rainfalls. As shown in Fig. 5.13(c), for the 24-hr rainfalls, the difference between patterns at the depth of 18.3 m is significant at 192-hr (one week after the rainfall terminations) and would seem not to converge soon since all the curves are all rising. However, for the 168-hr rainfalls, the total increase of every pattern in the groundwater table is about 3 m, higher than the greatest rise of about 1.5 m caused by the 24-hr rainfall of Pattern A2 in Fig. 5.13(c). This can be attributed to the greater total rain depth of prolonged rainfalls, about 1.5 times that of 24-hr rainfalls; and the intensity is lower, which allows less surface runoff during the rainfall. It is thus understandable that more water infiltrates the slope, causing a greater rise in the groundwater table. This means that the rise in the groundwater table is mainly influenced by the total rain depth, whereas the variation in pore-water pressure near the ground surface is mostly affected by rainfall intensity.

The variation of hydraulic heads at section YY’ is not shown, since the features at the toe are the same as those at the crest, such as the clear time-lag but lack of difference in the peak value of pore-water pressure at a given depth among patterns. The groundwater table also rises to the ground surface at a certain time for a given rainfall, but the upward flow occurring in the Pattern A2 rainfall cannot be seen. This means that the upward flow at the toe can only be induced by rain of a sufficiently high in-
tensity. In a prolonged rainfall, the intensity is too low to cause significant groundwater flow from upstream to the toe of the slope.

The temporal variation of \( q/k_w \) for 168-hr rainfalls is plotted in Fig. 5.14. It is clearly seen that the ratio is generally much smaller than that of the 24-hr rainfalls in Fig. 5.10. The starting point for Pattern A2 is only about 20, while it is about 200 in Fig. 5.10. It is because the rainfall intensity is much lower for prolonged rainfalls. After about 30 hr, the ratio for each pattern drops below unity, which means that the applied flux is smaller than the water permeability of the soil at that time. Soil is therefore able to receive more rainwater of the prolonged 168-hr rainfalls including at section YY' since the ratio there is below unity from the beginning of the rainfall, compared with the total infiltration for the short 24-hr rainfalls. This explains the greater rise in the deep groundwater table for prolonged rainfalls. The difference among the ratios of the patterns is less significant than that in Fig. 5.10 for either section. This may lead to a very similar total infiltration for all three patterns and consequently an equivalent increase in pore-water pressure and rise in the groundwater table. That could explain why there is no much difference in the pore-water pressure change for each pattern, save some difference temporally.

5.5 SUMMARY AND CONCLUSIONS

In this chapter, various groundwater responses in a typical unsaturated slope subjected to different rainfall conditions were investigated numerically by the 3D groundwater modeling program, FEMWATER. Three typical rainfall patterns in Hong Kong, namely, U, C and D1, with three different return periods, 10-, 100- and 1000-yr were adopted. Temporal variations of hydraulic head at different depths were plotted for
for two sections at the crest and toe of the cut in the middle part of the slope.

Pattern A2, C and D1 rainfalls of a 24-hr duration with 10-, 100- and 1000-yr return periods were used for studying the effects induced by rainfall patterns. Remarkable differences in groundwater responses caused by different patterns in both sections can be seen not only during rainfall but also long after the cessation of rainfall. The difference resulting from different rainfall patterns mainly derives from the first several hours of rainfall, when there is a more significant difference in the infiltration rates of the patterns. Different infiltration rates lead to different groundwater flows under the ground surface, and hence different pore-water pressure distributions.

The advanced type, Pattern A2, seems to be the most critical pattern among the three with a given return period for slope stability, because it can induce the greatest increase in pore-water pressure along depth in both the crest and the toe of the slope. The reason is the high hydraulic gradient caused by Pattern A2 during the first several hours of rainfall, which results in a higher infiltration rate and in turn total rainwater infiltration than other two patterns. Hence, Pattern A2 is recommended for the slope stability analysis of a rainfall induced landslide, which will make the analysis results more conservative. It should be noted that this conclusion can only be applicable for duration rainfalls.

The groundwater responses are different at the crest and the toe of the cut, which implies that the initial groundwater condition has a significant influence on groundwater response. At the crest, where the initial groundwater table is deeply seated, the matric suction is reduced by the rainwater infiltration but the groundwater table does not rise significantly. However, the soil is fully saturated at the crest because the shallow groundwater table rises to the ground surface. In addition to the different initial groundwater conditions involved, namely, groundwater table depth and hence differ-
ent pore-water pressure distribution, the groundwater response at the lower section YY' is unavoidably influenced by groundwater flow from section XX' higher up the slope. Moreover, a clear wetting front cannot be observed in either section XX' or YY', so it may be concluded that the model of the ‘wetting front’ is not suitable for the 3D analysis.

The variation of $q/k_w$ with time which differs from pattern to pattern, can enhance the understanding of the infiltration mechanism. As rainfall continues, the soil is wetted and the permeability increases gradually. The increase in water permeability can be very significant at the crest where the initial matric suction is high and the initial permeability is low, therefore, $q/k_w$ generally decreases as rainfall continues. However, the variation of this ratio is different at the toe where $k_w$ is nearly the saturated permeability $k_{sat}$. Therefore, $k_w$ cannot increase greatly so $q/k_w$ is mainly governed by $q$, so it closely follows its corresponding rainfall pattern.

The influence of return period can be concluded that generally the higher the return period is, the greater the increase in pore-water pressure will be, but the relationship between the two is not proportional. In this study, when the return period exceeds 100-yr, the rate of increase in pore-water pressure caused by rainwater infiltration decreases. For the extreme case, the advance type Pattern A2, there is clearly not any difference between the 100- and 1000-yr cases. The reason is that surficial permeability mainly governs infiltration during intense rainfalls and thus rainwater has to become surface runoff when rainfall intensity far outstrips surficial water permeability. That means a rainfall with higher intensity does not necessarily affect the slope stability more adversely because the infiltration is dependent on surficial permeability. It might therefore be recommended that for numerical analysis, a return period of 100-yr may be sufficient to study the problems of slope instability induced by rainfall.
It can be noticed that the variation in pore-water pressure in shallow soils is mostly influenced by rainfall intensity. The higher the rainfall intensity, the greater the variation of pore-water pressure near the ground surface. However, prolonged rainfall will cause a greater increase of pore-water pressure in deep soils (about 19 m underground in this chapter) due to lower rainfall intensity and greater rainfall depth, and hence greater total infiltration, which means groundwater response in deep soils is mainly governed by total rainfall depth. Therefore, it might be concluded that prolonged rainfall is more critical for deep slope failure whereas a short rainstorm is more likely to induce a shallow slope failure. Moreover, prolonged rainfalls with a 168-hr duration cause less difference in the magnitude of variation of pore-water pressure between patterns, except for a temporal difference, which can be attributed to different temporal rainwater distributions for these rainfall patterns. This is mainly because the rainfall intensity is generally much lower and rainwater is more uniformly distributed with time. It is suggested that more attention be paid to rainfall patterns of short intense rainstorms rather than to prolonged rainfalls.

5.6 REFERENCES


Ting, K. Y. (1998). Storm statistical properties in Hong Kong. Final Year Project of Dept. of Civil Engineering, Hong Kong Univ. of Sci. and Tech.


<table>
<thead>
<tr>
<th></th>
<th>Pattern A2, C, D1</th>
<th>Pattern A2, C, D1</th>
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<td>597</td>
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Fig. 5.1 Three-dimensional finite element mesh of the investigation site (after Tung et al., 1999)
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Unit: m

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Fig. 5.14  Variations of hydraulic head (a) at ground surface (b) at depth of 6.1 m (c) at depth of 18.3 m at section XX’ for 168-hr rainfall with 100-yr return period
CHAPTER 6 INFLUENCE OF STRESS-DEPENDENT HYDRAULIC PROPERTIES ON PORE-WATER PRESSURE DISTRIBUTIONS

In this chapter, some results of transient seepage analyses computed from a finite element program will be presented. The mesh of an unsaturated expansive soil slope was constructed according to the designed slope of the MRP canal. The stress effects will be and not be considered in two parallel series of simulation tests. The groundwater responses in the slope subjected to rainfalls of 7 days were simulated by the program. The results from the comparable cases will be used to investigate the stress effects of soil properties on transient seepage analysis.

6.1 DESCRIPTION OF FEM PROGRAM

6.1.1 Introduction to coupled analysis

Deformation of unsaturated soils may be considerable due to any change of moisture content. All the soil behaviours including flow through unsaturated soil should take into account soil deformation. Huang et al. (1998) developed a coefficient of permeability function for a deformable unsaturated soil, attempting to model the problem of flow through saturated/unsaturated soils. Thomas and He (1995) developed a theoretical formulation for the analysis of coupled heat, moisture and air transfer, which is applicable to
a deformable unsaturated soil. The original objective of the study presented in this chapter is to develop a program for computation of flow-deformation coupled problems in unsaturated soils. Therefore, both stress-strain relationship and governing equations for seepage analyses are incorporated into the program. The stress, strain and water content in the soil body can be solved simultaneously. However, it should be noted that the results presented in this chapter is not a so-called coupled analysis, because deformation due to change of water content is not calculated but the hydraulic properties adopted are stress dependent.

6.1.2 Theoretical background of the program

The air pressure in the unsaturated part of a soil is assumed to be equal to the atmospheric pressure and only the water phase (single-phase approach) will be considered in the mathematical formulation. Darcian flow is assumed.

For an isothermal system, the governing equations can be written as the following conservation equation (Eq. 6.1):

For water volume:

\[
\frac{\partial (\rho n S_w)}{\partial t} + \text{div}(\rho V_w) = 0
\]  

(6.1)

Where \( n \)-porosity; \( S_w \)-Saturation; \( V_w \)-velocity of water \( \rho \)-density of water.

Momentum equation (Eq. 6.2):

\[
V_w = -K_w \text{grad}(z + \frac{u_w}{\rho g})
\]  

(6.2)

Where \( z \)-elevation
The concept of two stress-state variables together with a simple state surface approach were proved to be useful in coupled flow and deformation problems and was used in this study (Fredlund and Rahardjo, 1993). Based on this, equations can be written as follows:

Equilibrium equations (Eqs. 6.3&4):

\[
\frac{\partial (\sigma - u_x)}{\partial x} + \frac{\partial \tau}{\partial y} + \frac{\partial \tau_{xy}}{\partial y} + X = 0
\]  
(6.3)

\[
\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial (\sigma - u_y)}{\partial y} + \frac{\partial \tau_{xy}}{\partial y} + Y = 0
\]  
(6.4)

 Constitutive equations for the plane strain problem (Eq. 6.5-7):

\[
\varepsilon_x = \frac{1-\mu^2}{E} \left[ (\sigma_x - u_a) - \frac{\mu}{1-\mu} (\sigma_y - u_a) \right] + \frac{u_a - u_x}{H}
\]  
(6.5)

\[
\varepsilon_y = \frac{1-\mu^2}{E} \left[ (\sigma_y - u_a) - \frac{\mu}{1-\mu} (\sigma_x - u_a) \right] + \frac{u_a - u_y}{H}
\]  
(6.6)

\[
\gamma_{xy} = \frac{2(1+\mu)}{E} \tau_{xy}
\]  
(6.7)
Lloret and Alonso (1985) suggested an analytical expression for suction induced volumetric deformation (Eq. 6.8):

\[
e = d + a \log(\sigma - u_a) + b \log(u_a - u_b) + c \log(\sigma - u_a) \log(u_a - u_w)
\]

where \(e\) is the void ratio, \(\sigma\) is the isotropic stress, \(a, b, c, d\) are constants. It has been shown to be convenient in solving coupled flow and deformation problems.

From Equation (Eq. 6.8) the modulus of suction deformation can be obtained as follow:

\[
\frac{1}{H} = \frac{b + c \log(\sigma - u_a)}{(1 + e_0)(u_a - u_w)}
\]

(6.9)

By using the total hydraulic head \(\Psi = Z + \frac{u}{\rho_w g}\) as an unknown variable, regarding to \(\frac{\partial \sigma}{\partial t} = 0\), Equation (Eq. 6.1) can be rearranged as following:

\[
[S_h, S_v + C(h)] \frac{\partial \psi}{\partial t} = \text{div}[K_w \text{grad} \psi]
\]

(6.10)

This is the governing equation for seepage in the unsaturated zone (Eq. 6.11):

\[
S_s = \rho g [n\beta - \frac{0.4343(1 - n)^2[b + c \log(\sigma - u_a)]}{u_a - u_w}]
\]

(6.11)

Where \(\beta\) is compressibility of water \([T^2LM^{-1}]\), \(C(h)\) is water storage, equal to zero in the saturated zone, \(h\) is the pressure head. Similarly using the effective stress concept one can
obtain the following expression for the saturated part:

$$S_s = \rho g (n \beta + m_s)$$ \hspace{1cm} (6.12)

Where $m_s$ is soil compressibility factor [$T^2LM^{-1}$].

Eq. 6.10 together with Eq. 6.11 & 12 can be used for seepage prediction in saturated and unsaturated flow in soils, but here all coefficients in Eq. 6.10 are related to stress-state of soil, so this requires simultaneous solution of flow and mechanical problem.

A Galerkin weighted residual technique leads to a spatial discretization of the problem and transforms the set of differential equations into a system of first order differential equations. A 2-dimensional (2D) computer program using the state surface approach has been developed with Fortran by J. K. Liu\(^1\) to solve the formulated problem.

As stated before, it should be noted that in this study, only transient seepage analyses have been carried out. Gravity force is considered in the stage of computing steady state only but are no longer used in transient flow computation. Moreover, no deformation is taken into account, which means stress-strain is not computed in the transient seepage analyses. Those hydraulic inputs are stress dependent, derived from the laboratory test results. Therefore, more accurately speaking, it is not truly a flow-deformation coupled study but simply a preliminary study of transient seepage analyses with stress dependent hydraulic properties as inputs, in order to investigate the stress effects on pore-water pressure distributions in unsaturated soil slopes. In this thesis, since the soil samples are expansive soils, the soil slope is an unsaturated expansive soil slope.

\(^1\) Dr J. K. Liu is currently full-time faculty in Northern Jiaotong University, China.
6.2 INPUTS OF THE SIMULATION TESTS

6.2.1 Numerical mesh

The mesh was constructed according to the trunk canal design of the MRP, as shown in Fig. 6.1. The dimensions of the mesh are 227 m wide (horizontal) and 57 m high (vertical). In the figure, the right side is the aqueduct and the left side is infinite horizontal ground. The slope of the canal bank is designed to be 1:4, so the inclination angle is only 14°. An impermeable layer is assumed to be at 57 m underground from the horizontal ground surface, which is 12 m below the bottom of the aqueduct. The half width of the bottom of the aqueduct is 8 m. To simplify the problem, it is also assumed that the entire mesh is composed of one soil layer only, which is a typical expansive soil (ZY). After the process of automatic discretization, the entire mesh is comprised of quadrangles of 3×3 m² elements. For the elements touching the boundaries, the program can automatically adjust the element shape and size according to the geometry condition of the boundaries. However, the finite element mesh can not be output.

6.2.2 Boundary conditions

The vertical sides of AB and FG are both defined as zero flux boundaries, which means there is no water flow crossing the two sides. It is clear that BCD should be defined as a constant head boundary, since the water level in the aqueduct is assumed to be constant regardless of precipitation or evaporation. It should be noted that there must be some variation in the water level in the canal due to different climatic conditions. Therefore, the assumption should be valid and reasonable. BCD and FG are constant head boundaries of 20 and 27 m high respectively. It means that the groundwater table is lo-
cated at 30 m below the ground surface. It is possible because the areas where the canal cross through are arid and can have deeply seated groundwater tables in dry seasons.

DE and EF are the inclined slope surface and the horizontal ground surface respectively. They are both flux boundaries receiving rainwater during rainfall.

Finally, as mentioned before, AG is another zero-flux boundary of the mesh, since the soil/rock underlying below the mesh is assumed to have very low permeability and treated as an impermeable layer. It can be unweathered rock or considerably impermeable clay.

6.2.3 Hydraulic properties

Because the objective of the study described in this chapter is to investigate the stress dependent hydraulic properties on transient seepage analysis, two series simulations have been carried out. In each series, there are two parallel cases, one is with the stress effects considered, and another is without. For the former case, all soil properties are stress dependent, such as SWCCs and permeability, whereas for the latter one, only the soil properties of zero-stress state are adopted. Hysteresis is considered in both cases, which means that both cases use wetting and drying curves. The only difference between the two cases is whether the properties are stress dependent or not.

The slope is composed of ZY expansive soil. Those index properties were the same as introduced in Chapter 3. The relevant soil properties for the program include SWCCs expressed in VWC, permeability, soil density. If stress and deformation are calculated, elastic modulus and shear strength parameters are needed. However, they are irrelevant for this study.
The soil density for input is 1780 kg/m³, which is the natural density of the slope. The SWCCs are those measured under ISO stress conditions, including three stress levels, namely, 0, 50 and 100 kPa (Fig. 3.23 (b)). The saturated permeability was calculated but not measured, based on the Kozeny-Carman equation (Eq. 6.13) (Ahuja et al., 1989).

\[ k_s = B n_r^4 \]
\[ n_r = n - \theta_w \]  
(6.13)

where

\( k_s \) = saturated permeability (m/s),

\( B \) = constant equal to 0.002939,

\( n \) = porosity of the soil,

\( \theta_w \) = volumetric water content when a suction of 33 kPa is applied to the soil.

For the saturated permeability, the values are different for wetting and drying paths for a give stress condition, except for zero stress, whose the saturated permeability is assumed to be the same for both sorption and desorption curves. Porosity was calculated according to measured volume change and the volumetric water contents were obtained from the measured SWCCs. The saturated permeability is listed in Table 6.1. It can be seen from the table that the saturated permeability decreases with increasing stress condition. It is because the soil porosity decreases with stress significantly, while the volumetric water contents at 33 kPa matric suction are relatively of small differences. The zero stress condition corresponds to the ground surface, whereas 50 and 100 kPa can be considered to be about 3 and 6 m underground. The calculated saturated permeability is in the reasonable range referring to Liu (1997). In that book, it was reported that different expansive soils were slightly different in terms of permeability. Brown-yellow Nanyang expansive soils usually have permeability ranging from 3.4e-8 to 3.6e-10 m/s through
laboratory tests, whereas the field measurements give about 1.1e-7 to 2.2e-8 m/s. ZY is a brown-yellow Nanyang expansive soil. Moreover, one must be aware that there are a lot of cracks in the field of expansive soils. However, cracks are not considered in the study due to the deficiency of usable data. Therefore, the values shown in the table are applied for the numerical analysis.

The permeability function was calculated according to Fredlund et al. (1994). First, SWCCs of all three stress levels were fitted with the Fredlund&Xing's SWCC equation (Fredlund and Xing, 1994) in SoilVision to get these parameters of the SWCCs. The parameters are listed in Appendix III. Then the parameters of one SWCC were input to the equation to calculate the permeability at different matric suctions to obtain a curve of water permeability. Correspondingly, permeability functions are also of three stress levels, 0, 50 and 100 kPa, giving 6 different curves of wetting and drying paths. In computation, the program will automatically interpolate and extrapolate the data and adopt the right values according to a given stress. The computed curves of permeability are shown in Fig. 6.2. As seen from the figure, the permeability of a drying path is higher than that of a wetting path, for a give stress condition. And the curves look nearly parallel to each other, except for the desorption and sorption curves of zero stress state. Since the saturated permeability decreases with stress, the coefficient of water permeability for a given suction also decreases with stress. The trend is clearly shown in Fig. 6.2.

However, it should be noted that there are only three stress levels, i.e. 0, 50, 100 kPa of isotropic stress, of hydraulic properties. In computation, the program automatically interpolates the three levels, using the linear interpolation method, according to any computed stress level. For those stresses higher than 100 kPa, no extrapolation is made.
Instead, the hydraulic properties of 100 kPa are used for a soil element with a stress higher than 100 kPa.

6.2.4 Rainfall conditions

The adopted rainfall is 168-hr (7 days) long. The data were obtained from the Engineering Geology Report for the trunk canal of the Middle Route Project of the South-to-North Water Transfer. The rainfall parameters are listed in Table 6.2.

The rainfall intensity is assumed to be constant with time. Although it has been shown that rainfall patterns do have influence on groundwater responses, the seepage study in the chapter is to investigate the influence of stress effects on hydraulic properties on transient seepage. To simplify the problem and also because no rainfall pattern is available in the rainfall records for Nanyang areas, constant rainfall intensity is assumed for the period of rainfall.

To simulate the exact process of rainwater infiltration is not practical. Considering many factors involved in the infiltration coefficient, the coefficient of infiltration is decided to be 0.6, which means that 60% of rainfall intensity is adopted for the infiltration rate in the study.

6.2.5 Initial groundwater condition

6.2.5.1 Same initial groundwater condition

By specifying a boundary condition, only a hydrostatic condition can be obtained through the program, the surficial matric suction will thus be too high to have a reasonably high water permeability. Therefore, the rainwater of high intensity rainfalls can not
get into the soil in a considerable amount. It should be noted that those soil properties are not stress dependent in running the initial condition, i.e. the stress effects are not considered for computing this initial state. After the desired initial groundwater condition was obtained, then two series studies were carried out, one was the stress effects considered, the other was without considering the stress effect on the soil hydraulic properties.

The initial condition, named Initial A, used in the study was obtained by applying a very small flux on the slope surface of only 0.1 mm/day, to reduce the surficial matric suction. 1000 days after, a nearly steady state was reached, the surficial soil was wetted and the suction was reduced. The initial groundwater condition (Initial A) is the same for Cases D and E in the study.

The computed initial groundwater condition is illustrated with a number of pore-water pressure head contours in Fig. 6.3. The contours are nearly parallel in the slope except for the region near the ground surface. As seen from the figure, the pore-water pressure head is negative along the slope surface, which means that slope is initially unsaturated. The negative pore-water pressure (matric suction) is low downstream and high upstream, up to 15 m (150 kPa).

The cross section, OO' (x = 162 m, the horizontal distance with respect to the origin), is selected in the middle of the slope to demonstrate the variations of pore-water pressure distributions under rainfall. All the results will be presented in the section hereinafter. The total height of the selected section is 46 m. The initial pore-water pressure distribution along depth is shown in Fig. 6.4. The pore-water pressure is nearly constant down to the depth of 10 m, which results from the long period of small flux applied on the slope to reduce surficial matric suction. The groundwater table is located at 21 m be-
low the ground surface. Compared with the hydrostatic condition along the section, it can be seen that the reduction of matric suction generally decreases with depth, and the top 10 m of soil is wetted significantly, but there is only a slight rise in the groundwater table at the depth of 21 m.

6.2.5.2 Two different initial conditions

The initial conditions, Initial B and Initial C, were obtained from running two cases, starting from the same hydrostatic condition with Case A. Stress dependent SWCCs and permeability were used in Initial B, whereas Initial C adopted conventional properties only without considering stress effects. The permeability functions are as shown in Fig. 6.2. They are two nearly steady states by applying 0.2 mm/day infiltration for 1000 days.

The pore-water pressure distributions at section OO’ are shown in Fig. 6.5. It can be seen that the two initial conditions have different pore-water pressure distributions on the slope. There is a clear rise of 5 m of ground-water table from the original depth of 21 m in Initial C, while the groundwater table in Initial B retains at the depth of 21 m. According to the results in Chapter 5, the higher rise in the groundwater table implies a greater amount of total infiltration. This is because the permeability without considering stress effects is higher in Initial C. A greater amount of rainwater can infiltrate into deeper soil in the slope, compared with the situation in Initial B. On the other hand, the pore-water pressure at the ground surface of Initial B (about -50 kPa) is lower than that of Initial C (about -90 kPa). This can be attributed to the lower permeability in Initial B as stress increases. The water flows in a lower rate underground, which results in higher water content (lower matric suction) at the ground surface, but lower water content (higher matric suction) underground. Therefore, the pore-water pressure for Initial B above the
depth of 1m is higher than that for Initial C, whereas the pressure of Initial B below 1 m is lower than Initial C. There is no change in the depth of the groundwater table for Initial B. The result is skeptical because normally the pore-water pressure are lower when the permeability is higher.

6.3 RESULTS AND DISCUSSION

Totally four cases, Case D-G, were conducted. The applied rainfalls were all 168 hr long. The first two cases were both computed based on Initial A. One was with the stress effects on hydraulic properties considered, named ‘Case D’ and the other did not use the stress dependent hydraulic properties, named ‘Case E’. Therefore, the two cases with the same rainfall condition, Initial A, will be compared to see the stress effects on the groundwater responses in the slope. The other 2 cases, Case F and Case G were computed from Initial B and Initial C respectively. Consistent with the properties adopted in the initial state, Case F used stress dependent hydraulic properties and Case G used conventional hydraulic properties by ignoring stress effects. The relevant information of these simulations is summarized in Table 6.3.

6.3.1 Stress effects – running upon Initial A

The pore water pressure distributions of the two cases at section OO’ are shown in Fig. 6.6. Only the pore-water pressure conditions at the end of the rainfall were selected to be shown, because the variations of pore-water pressure change under rainwater infil-
tration have been presented in great detail in Chapter 5. In this study, rainfall intensities are constant, so that the matric suction in the surficial soil is reduced gradually from the ground surface. Unlike those responses seen under some particular rainfall patterns, e. g. advanced pattern, the peak hydraulic head will not appear during the rainfall but normally at the end of the rainfall under a rainfall with constant intensity.

It can be seen that after 7 days of rainwater infiltration, Case A with the stress effects considered predicts a generally higher matric suction profile. At the end of the rainfall, only 1.5 m depth of soil has been wetted for Case A, whereas pore-water pressures are generally reduced up to 4 m underground for Case B. The groundwater table is not affected in either case. It can be expected that applying longer rainfalls, the trend will simply be the same as the 7-day cases, but the depths of soil being wetted will be greater for both cases, with the stress effects considered and not considered. The results presented above can be attributed to two causes: (1) The permeability of the soil is very low, which can be attributed to the initial high matric suction, so that even after one week’s rainfall, the influenced zone is only 1.5 m for Case A and 4 m for Case B. (2) For any given non-zero stress state, the permeability of Case A is lower than that of Case B. The matric suction in the slope in Case B is more easily reduced by rainwater infiltration, since the water flows at a higher rate.

Therefore, it may be concluded that for this particular expansive soil, the seepage analysis with the stress effects considered in the input hydraulic properties predicts a higher negative pore-water pressure distribution. The higher matric suction in the surficial soil will contribute to the shear strength of the soil, and may result in a high factor of safety in the slope stability analysis. The case ignoring the stress effects comes up with a
generally higher pore-water pressure distribution in the slope, so it is more conservative to consider this case for slope stability problems.

6.3.2 Stress effects – running from Initial B and Initial C

Case F was computed based on Initial B, with stress effects considered in both stages. Case G was computed based on Initial C, without considering stress effects. Therefore, the initial groundwater conditions are different for Cases F and G and the pore-water pressure distributions are also different after 7 days of rainwater infiltration. The rainfall condition for these two cases is the same as that for Cases D and E.

The pore-water pressure distributions at section OO’ at the end of the rainfall are shown in Fig. 6.7. At the ground surface, the matric suction is only about 5 kPa for both cases, although the initial matric suction for Initial B is lower. This is because the surficial matric suction is mostly determined by rainfall intensity. As for Case F, only 1 m deep of soil is wetted by 168 hr (7 days) rainwater infiltration, whereas the wetted zone is about 4 m for Case G. It can be seen that generally Case G predicts a lower matric suction profile at this section that Case F. The phenomena described above can be attributed to the lower stress dependent permeability for Case F. For Case G, the permeability for any location underground is as low as that at the ground surface, therefore, the water can flow in a higher rate underground, compared with the groundwater flow in Case F.

It may be inferred that the pore-water pressure profile of Case F is safer, considering that the slope stability is adversely affected by higher pore-water pressure. The conventional case predicts a generally lower matric suction profile in the slope, which is more dangerous for the safety of the slope. Therefore, it may be more conservative to use conventional hydraulic properties for transient seepage and slope stability analysis.
6.3.3 Further discussion

Ng and Pang (1999) conducted an analysis in an unsaturated residual soil slope to investigate the stress effects of hydraulic properties on pore-water pressure distributions due to rainwater infiltration. It was found that the pore-water pressure for the case using stress-dependent properties was lower, compared with the case using conventional hydraulic properties. It should be noted that the two cases were both computed based on the same initial groundwater condition. Their result concords to the results of the comparison of Cases D and E, i.e. after the slope is subjected to the same rainfall condition, the matric suction profile for the case with stress effects considered is higher than that for the case using conventional properties.

However, in their subsequent analysis, when two different initial groundwater conditions were used, an opposite conclusion was drawn (Ng and Pang, 2000). That transient seepage analysis using stress dependent SWCCs predicted significantly lower pore-water pressure in an initially unsaturated slope of a residual soil. It would have adverse effects on the slope stability. So it is more conservative to carry out slope stability analyses with the input of pore-water pressures computed from a seepage analysis using stress dependent SWCCs and other hydraulic properties with considering the stress effects. The seepage analysis using stress dependent properties gave a lower suction distribution in the initial condition, the matric suction was thus more easily destroyed to a greater depth by the applied rainstorm. Therefore, the authors highly recommended that stress dependent hydraulic properties be adopted for transient seepage analysis. In this study, when the initial conditions are different and retrieved from two initial cases (Initials B and C), which means that during the period of computing the initial groundwater conditions, one case
(Case B) considers the stress effects while the other (Case C) does not, the results of the two following infiltration simulations (Cases F and G) are again opposite to the results from Ng and Pang (2000), i.e. a case using stress dependent hydraulic properties predicts higher matric suction profiles after rainfall. This is because the two initial conditions shown by Ng and Pang (2000) are different from those two presented in this study. In their study, the pore-water pressure in the case with stress dependent properties used was generally higher than that in the other case with only conventional properties as inputs. Therefore, in their result, the stress dependent inputs predicted generally lower matric suction profiles in the slope. However, according to the two initial conditions in this study, only the matric suction in the top 1 m in the simulation with stress effects considered is lower than that in the other simulation without stress effects considered, while the matric suction below the depth of 1 m for the former case is higher than that for the latter one. This is due to the sharp decrease in water permeability with increasing depth (stress) Therefore, after 7 days of rainwater infiltration, the matric suction becomes generally lower for the case using conventional hydraulic properties, which may be more adverse for the slope stability.

It should be noted that the results obtained in the study may not be applicable to the transient seepage analyses in any expansive soil slope. Because the soil density in the study is quite low, the measurements of SWCCs on a soil with another density and hence the predictions of other hydraulic properties may lead to different conclusions.
6.3.4 Implications on slope stability

Seepage analysis can predict the groundwater conditions, e.g. pore-water pressure distribution profiles, in a slope. The computed pore-water pressure profiles are used as one of the inputs for the following slope stability analysis. It is known that the matric suction in a slope contributes significantly to the factor of safety of the slope (Fredlund and Rahardjo, 1993; Fourie, 1996; Sun et al., 1998; Ng and Shi, 1998; Fourie et al., 1999). Leong et al. (1999) showed that a lower matric suction profile before rainfall resulted in higher pore-water pressures and hence a lower factor of safety of the slope after subjected to rainwater infiltration. Rainfall induced landslides are the most common natural disasters in many regions in the world. Reduction in matric suction has adverse influence on slope stability.

The results of the seepage analysis imply that for carrying out a slope stability analysis subjected to rainfall, if the initial groundwater condition is known, e.g. controlled by human, or specified by field measurements, conventional hydraulic properties may be more conservative, since a lower suction profile will be predicted.

When the initial groundwater condition before a rainfall is unknown and should be determined prior to a transient seepage analysis subjected to rainfall, no definite conclusion can be drawn. The pore-water pressure distribution after rainfall depends the hydraulic properties and the initial condition, which in turn depends on the hydraulic properties. For some stress dependent properties, it may be more conservative to run the initial state and infiltration simulation with stress dependent properties used, since a lower matric suction profile in the slope will be predicted and it is more conservative for the following slope stability analysis. However, for some other stress dependent properties, it seems
that the conventional properties without considering the stress effects are more conservative, since a lower matric suction profile is predicted by the case using conventional hydraulic properties. Hence, it seems that no conclusive statement can be made. The pore-water pressure distribution (matric suction profile) is determined by hydraulic properties. Further research is necessary. It can nevertheless be concluded that stress dependent hydraulic properties as inputs for infiltration simulations predicts different pore-water pressure distribution, compared with conventional properties ignoring stress effects. It is important to consider stress effects for a better estimation of pore-water pressure distribution and slope stability.

6.4 SUMMARY

In the first series of simulation, with the same initial groundwater condition, two cases of transient seepage analysis, one case using stress dependent hydraulic properties and the other using conventional hydraulic properties, were conducted on an initially unsaturated slope of expansive soil. A 7-day rainfall was simulated. The results of pore-water pressure distributions in the slope after rainfall are presented in the chapter. It is found that the case with stress effects considered predicts a higher matric suction profile, due to its lower permeability. The results agree with the previous study on stress effects on slope stability (Ng and Pang, 1999). Therefore, prior to a transient seepage analysis, if the initial groundwater condition in the field is known, it is better to adopt the properties without stress effects considered for this particular expansive soil with the density used similar to that in this study, in order to be more conservative.
However, in the second series of simulation, stress effects are included in the initial states, i.e. the two initial condition are with and without stress effects considered, respectively. Then two infiltration simulations start from the two different initial conditions. The case without stress effects included predicts a lower matric suction profile in the slope because the matric suction of its initial condition is much lower below the depth of 1 m. The permeability without considering stress effects is higher and thus rainwater can infiltrate into the soil more easily and flow into a greater depth. Therefore, the pore-water pressure is higher after the rainfall, which may be more critical for the safety of the slope. But it should be noted that the result is very skeptical. The results seem to be opposite to the conclusion drawn by Ng and Pang (2000). It is thus recommended that further research are necessary for this problem, since the pore-water pressure distribution highly depends on the input hydraulic properties.

For a transient seepage, it is more desirable to adopt stress dependent hydraulic properties as input parameters to make better estimation of groundwater condition and slope stability, since infiltration simulation with or without stress effects considered for hydraulic properties can be very different.

6.5 REFERENCES


<table>
<thead>
<tr>
<th>Stress condition (kPa)</th>
<th>Wet/dry path</th>
<th>$K_{sat}$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (ZY-R3)</td>
<td>Desorption</td>
<td>$2.60 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>Sorption</td>
<td></td>
</tr>
<tr>
<td>50 (ZY-R6)</td>
<td>Desorption</td>
<td>$1.84 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>Sorption</td>
<td>$7.52 \times 10^{-8}$</td>
</tr>
<tr>
<td>100 (ZY-R7)</td>
<td>Desorption</td>
<td>$1.37 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>sorption</td>
<td>$3.93 \times 10^{-9}$</td>
</tr>
</tbody>
</table>
Table 6.2 Rainfall parameters

<table>
<thead>
<tr>
<th>Duration of rainfall (hr)</th>
<th>Rain depth (mm)</th>
<th>Intensity (m/s)</th>
<th>Return period(^1) (yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>168</td>
<td>560</td>
<td>9.2594e-7</td>
<td>20</td>
</tr>
</tbody>
</table>

\(^1\) They are not explicit in the report, so the values of rainfall return periods are estimated by the author, with reference to rainfall statistics in Hong Kong.
Table 6.3  Summary of various infiltration simulations

<table>
<thead>
<tr>
<th>Simulation</th>
<th>Stress effect considered</th>
<th>Initial condition</th>
<th>Rainfall intensity × Duration × Infiltration coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial A</td>
<td>No</td>
<td>Hydrostatic</td>
<td>0.1 mm/day × 1000 day × 1</td>
</tr>
<tr>
<td>Initial B</td>
<td>Yes</td>
<td></td>
<td>0.2 mm/day × 1000 day × 1</td>
</tr>
<tr>
<td>Initial C</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Case D</td>
<td>Yes</td>
<td>Initial A</td>
<td></td>
</tr>
<tr>
<td>Case E</td>
<td>No</td>
<td></td>
<td>80 mm/day × 7 day × 0.6</td>
</tr>
<tr>
<td>Case F</td>
<td>Yes</td>
<td>Initial B</td>
<td></td>
</tr>
<tr>
<td>Case G</td>
<td>No</td>
<td>Initial C</td>
<td></td>
</tr>
</tbody>
</table>
Fig. 6.1  Geometry of the cross-section of the designed canal slope
Fig. 6.5 Stress effects on pore-water pressure distribution for the initial conditions of B and C at section OO'.
Fig. 6.6 Stress effects on pore-water pressure distribution at section QO under 168-hr rainstorms.

Initial A
Case D (168 hr)
Case E (168 hr)

Pore-water pressure (kPa)

Depth (m)
Fig. 6.7  Stress effects on pore-water distribution at section OO' after 168 hr rainfall
CHAPTER 7 CONCLUSIONS AND RECOMMENDATIONS

The research that has been completed in the thesis has resulted in a number of conclusions based on the detailed analysis addressed in the previous chapters. In this chapter, the conclusions drawn upon the research work are listed, which are the major achievements of the thesis. Furthermore, some relevant recommendations for the continuing research are given based on some of the limitations of this thesis.

7.1 CONCLUSIONS OF THE THESIS

Conclusions based on the laboratory tests of SWCC, SEM and MIP can be summarized as follows:

1. Soil microfabrics can be visualized through SEM. The microfabrics of different soils are different. The fabric of the volcanic soil (CDV), a silt of low plasticity, is single-grained, whereas the fabrics of expansive soils, regardless of being intact or recompacted, are aggregated. There are mainly two categories of pore spaces in the aggregated expansive soil fabrics, inter-aggregate (about 7.5-150 μm in diameter) and intra-aggregate pores (smaller than 7.5 μm in diameter), according to the SEM photographs and MIP data. This agrees with the bi-modal distribution of pore spaces in recompacted expansive soils.
2. Soil fabric can be altered by applied stresses. Inter-aggregate pores are compressed in a visible scale by external stress. Generally, a sample looks denser if it is under non-zero stress condition. Assuming the initial fabrics are identical for all the recompacted samples, different stresses may cause different soil fabrics in the inter-aggregate pore size range, which leads to different soil-water characteristics in the low suction range, from 1 to 10 kPa.

3. Intra-aggregate pores cannot be affected by external stresses and the pore size distributions do not differ significantly for all the samples in the pore size range of 0.3 to 30 μm. Therefore, the SWCCs tested under different stress conditions look parallel when the matric suction beyond 10 kPa.

4. The difference between the SWCCs tested under 1D and ISO stress conditions cannot be clearly identified. Comparing the SWCCs of different ISO stress conditions, the desorption rate under the matric suction lower than 5 kPa under a higher stress is higher, because more large pores (larger than 150 μm in diameter) can be compressed into inter-aggregate pores of the sizes ranging from 30 to 150 μm.

5. The pore size distributions of intact and recompacted samples are similar in the range of 30-0.3 μm, which can explain the same desorption rate of both natural and recompacted samples from about 35 to 500 kPa in measured SWCCs.

6. For the same soil type, extrinsic factors, such as dry density and external stresses, can influence those large pores (150 – 7.5 μm), which can be viewed as inter-aggregate pores, but intrinsic factors, such as different soil types, can cause some clear difference in smaller pores (0.3-0.06 μm), which are intra-aggregate pores.
7. Generally, the soil-water characteristics of the expansive soil samples tested in the study show that the air-entry value of expansive soils is high, mostly higher than 40-100 kPa, and the desorption rate is low. This is because the expansive soils tested are mainly composed by fine particles (≤ 2 μm in diameter). Compared with non-swelling soils – CDV, whose air entry values are usually lower than 20 kPa, expansive soils have a higher air-entry value and lower desorption rate.

8. The hysteresis is generally not significant in expansive soils compared with CDV. This may be due to the special mineralogical compositions of expansive soils, which lead to a stronger ability of sucking in water in the wetting path for this type of soil. The first hysteresis loop is relatively larger in size compared with subsequent cycles. At the end of the second wetting path, the wetting curve goes higher than the second drying path. In the third cycle, the wetting curve is much higher than the third drying path.

Through the analysis using SoilVision, more soil information can be obtained since the program can extend the measured data to a wider range (10^2 to 10^6 kPa) with some empirical/theoretical equations.

9. SoilVision helps analyze the measured data and extend the data to the complete range of matric suction. It is observed that the predicted curve based on grain size distribution can predict SWCC of a drying path of recompacted samples under conventional zero stress condition very well. But the prediction method cannot take into account natural fissures in intact samples, soil density or stress effects. The air-entry value is
found to be increased by the stress applied on the soil. Analyzed through SoilVision, generally, the higher the stress, the higher the air-entry value. It can indeed be attributed to the increase in soil dry density since a higher stress can reduce the void ratio more significantly.

The 3D numerical parametric study on rainfall analysis shows that:

10. Rainfall patterns, amount and duration have been found very influential for the groundwater responses in an unsaturated slope of residual soils. As for 24-hr short rainfalls, the advanced rainfall pattern seems to be the most critical rainfall type for the slope stability because it can always induce the highest pore-water pressure distributions along the slope. Generally, the higher the rainfall depth is, the higher the pore-water pressure buildup will be. However, it is found in this study that an further increase in the return period of 100 yr of a rainfall, the pore-water pressure does not necessarily increase because the higher intensity of rainwater infiltration increases the permeability of the soil and pore-water pressure will reach an limiting state and not rise any more. Rainfall patterns are more relevant for short rainfalls, since the rainwater distributes more uniformly for storms with long duration.

The 2D seepage analysis intends to investigate the influence of the stress dependent hydraulic properties of ZY expansive soil on the groundwater response in an initially unsaturated slope subjected to rainfall.
11. For a case with a known initial groundwater condition, two simulations were carried out based on the same initial condition. The slope is then subjected to a 7-day rainfall. The results show that the simulation using stress dependent hydraulic properties predicts a generally higher matric suction profile in the slope due to its lower permeability underground. The permeability in the soil for the conventional case is higher, so that water can flow in a higher rate and to a greater depth in the soil, which leads to a higher pore-water pressure distribution in the slope. The higher pore-water pressure distribution (lower matric suction profile) may have more adverse effects on the slope stability.

12. If a case does not have an initially known groundwater condition, the initial condition should be determined by the investigator before carrying out the seepage analysis simulating the groundwater responses under rainfall. Then, as expected, the initial conditions are different by adopting different hydraulic properties, with or without stress effects considered. The case with stress effects included predicts a lower matric suction profile within the top 1 m in the slope and the conventional case predicts a lower matric suction below the depth of 1 m. As a result, the permeability for the conventional case is higher below the depth of 1 m. After the 7-day rainfall, the two cases predict very close pore-water pressures at the ground surface, while the pore-water pressure distribution underground of the conventional case is generally higher, which may have important implications on slope stability. But it should be pointed out that the results are skeptical and further work is needed. However, stress dependent hydraulic properties are essential for better estimation of groundwater response in unsaturated soils under rainfall conditions.
7.2 SOME LIMITATIONS OF THE STUDY

A number of useful points can be concluded from the thesis, however, the study is still in the preliminary stage. Some limitations of the study can be summarized as follows:

1) XRD tests conducted are not able to identify the exact fractions of the clay minerals in the soil samples.

2) The explanation for the reversed hysteresis is nevertheless unsubstantiated. The result of the chemical analysis of the water used in the laboratory tests is not available. Natural samples usually exhibit insignificant hysteresis under wet-dry cycles, which may lead to unclear or unusual hysteretic behavior of the expansive soil.

3) The adopted ISO stresses all exceeded the swelling pressure of the samples, which made the stress effects somehow not clear. A stress below or equal to the swelling pressure should have been used.

4) The triaxial apparatus lacks an air-saturator to eliminate potential evaporation. Because of the special properties of expansive soils, a considerably long time is required for completing one test. Therefore, although the potential evaporation may not be significant for common nonswelling soils, it can have an unignorable influence on the results of measured SWCCs.

5) The measurement of the volume change is not of adequate accuracy. Even though the calibration was carried out, the measured volume changes could not be adequate confidence.
6) SEM and MIP tests were limited in results because of a shortage of usable soil samples after SWCC tests. Moreover, the stress effects cannot be exactly seen through SEM and MIP, since the volumes and in turn the soil fabrics of all the samples change with external stresses released after being taken out from the apparatus.

7) The seepage analyses considered the stress effects of those hydraulic properties, however, it is still far from being a flow-deformation coupled study.

7.3 RECOMMENDATIONS FOR FUTURE WORK

Following on from the limitations listed previously, some suggestions can be proposed as follows:

(1) More XRD tests to clarify the clay mineral contents of the expansive soils are needed. Other relevant tests for determining the chemical and physical properties of the expansive soil samples are recommended. They can be valuable for obtaining comprehensive information on soil properties and improving understanding of soil engineering behavior in other soil tests.

(2) The chemical analysis of the water used in the laboratory is recommended, which is of much value to assist the interpretation of the test results, since the physicochemical behavior may be dominant for these expansive soils. Moreover, distilled water is recommended for the tests.

(3) Wet-dry cycles are recommended for a recompacted soil sample. This may be more useful for investigating the hysteretic behavior of the expansive soil.

(4) More tests on the soil ZY with the same stress conditions as those in the thesis are recommended, in order to add confidence in the measured SWCCs. More-
over, the magnitudes of the stresses applied should be more carefully determined according to the swelling pressure of the soil. Some more test results of SWCCs under external stresses below/equal to the swelling pressure will be very helpful for the investigation of the stress effects on SWCCs of the expansive soils.

(5) The air-saturator is necessary for obtaining reliable wetting curves. It should be added to the newly modified ISO triaxial apparatus (and the prospective deviatoric triaxial apparatus). The position can be between the air-pressure source and the air-pressure valve.

(6) A double-wall cell will improve the performance of the volume change measurement, so it is highly recommended. Moreover, deviatoric stress will be able to simulate the stresses in field more closely. Therefore, further work should be focused on the SWCCs tested under deviatoric stress conditions.

(7) If SEM and MIP are to be carried out, the soil samples from SWCC tests should be more carefully handled in order to reduce the possible disturbance to the soil fabrics. Proper preparation of soil samples for microscopic analyses is of much importance, such as freeze-drying, freeze-fracture. However, some changes in the soil fabric of a sample are unavoidable, when the applied stress is released after the test finished. To freeze sample under an extremely low temperature can prevent the formation of ice structure by freezing the pore-water very quickly. Probably some more advanced techniques may be adopted to simultaneously observe the changing soil fabric under stress conditions and wet-dry cycles. For example, if some tiny sensor/micro-camera is imbedded in the soil
sample to make the change of soil fabric indirectly/directly visible, without bringing any disturbance to the soil or changing the stress condition.

(8) Truly flow-deformation coupled analyses would be of much value. The program should be re-developed or improved further. Therefore, more soil properties will be needed for the inputs, such as shear strength parameters. When some more new measurements are available, more transient seepage analyses will be needed, in order to provide more useful information and suggestions for the slope stability analysis of the S/N canal. Stress effects of hydraulic properties on groundwater flow need to be better understood.
Appendix I
Figure I-1  Intake of the trunk canal of S/N at Taocha

Figure I-2  Soil slope of the intake at Taocha
Figure I-3  Slope with surface drainage system at Taocha
Appendix II
Figure II-1  Cracks of ZY soil at the ground surface
Figure II-2  Field measurement of suction

Figure II-3  Exposed surface of ZY expansive soil in the field
Figure II-4  Slope with surface drainage system at Dagangpo
Figure II-5  Soil slope at Dagangpo

Figure II-6  Slope with surface drainage system at Dagangpo
Figure II-7  Soil slope at Zhanghe

Figure II-8  Sampling site at Zhanghe
Figure II-9  Soil sampling in field at Zhanghe

Figure II-10  Soil sampling at Zhanghe
Figure II-11  Compaction curve and the initial point for compaction
Appendix III
Input Volume-Mass properties for ZY expansive soil

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Input Volume-Mass properties for LZ expansive soil

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Input Volume-Mass properties for CDV from Peakroad

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For each soil, apart from the Volume-Mass relationship, the grain-size distribution and drying-wetting curves are used as input for fitting the grain-size distribution, fitting SWCCs and predicting SWCCs based on the grain size distribution.
Output of the fit curves from SoilVision

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Fredlund & Xing Equation (Fredlund and Xing, 1994):

$$\theta_w = \theta_n \left[ 1 - \frac{\ln \left( 1 + \frac{h}{h_r} \right)}{\ln \left( 1 + \frac{10^6}{h_r} \right)} \right]^{\frac{1}{\ln \left[ \exp(1) + \left( \frac{h}{a_f} \right)^{n_f} \right]^m_f}}$$

where:
\( a_t = \) A soil parameter which is primarily a function of the air entry value of the soil in kPa,

\( n_t = \) A soil parameter which is primarily a function of the rate of water extraction from the soil, once the air entry value has been exceeded,

\( m_t = \) A soil parameter which is primarily a function of the residual water content,

\( h_r = \) Suction at which residual water content occurs (kPa)

\( h = \) suction (kPa)
Appendix IV
Sample: ZY
Summary of Performed Tests
Bulk Test | Physical | Smectite | Heating at | Metallic Mound
Separation | Treatment | 500 deg C | with Lower Speed

Bulk Test (Before Physical Separation)

After Physical Separation

After Heating
After Physical Separation

Use of Metallic Mould at Lower Speed
MIP data (from Peking Univ, China)

**ZY-R1:**

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| 分选系数 $\sigma$ | 13.8603 |
| 几何因子 $G$ | 0.000 |
| 厚度 $D$ | 0.4635 |
| 透汞率 $10^{-9}$ $\mu$m$^3$ | - |
| 中值压力 $P_{m}$ (MPa) | - |
| 最大孔喉半径 $R_m$ (μm) | 75.0000 |
| 相对分选系数 $C_r$ | 1.1711 |
| 微观均质系数 $\alpha$ | 0.1578 |
| 透汞效率 $W$ (%) | 23.218 |

操作人:                审核人: 
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孔喉度 (%) = 35.600

操作人：

审核人：
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| 平均孔喉半径 R (μm) | 4.4508 | 最大孔喉半径 Rm (μm) | 70.9163 |
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| 造型度 φ | 0.5679 | 退汞效率 W (%) | 10.626 |

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操作人：

审核人：

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# LZ-R1

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操作人： 审核人：
Appendix V
Formulation of FEMWATER \textit{(Lin \textit{et al.}, 1997)}

The computer program used is GMS (Groundwater Modeling System), a software for 3D seepage analysis. The effective part for this study is FEMWATER, which is developed to solve the following system of governing equations along with initial and boundary conditions. The governing equation used in the program for 3D groundwater flow is as follows:

\[
\frac{\rho}{\rho_0} F \frac{\partial p}{\partial t} = \nabla \left[ K \left( \nabla p + \frac{\rho}{\rho_0} \nabla z \right) \right] + \frac{\rho^*}{\rho_0} q
\]
\[
F = \alpha' \frac{\theta}{n} + \beta' \theta + n \frac{ds}{dh}
\]

Where

- \( F \) = storage coefficient
- \( p \) = pressure head
- \( t \) = time
- \( K \) = hydraulic conductivity tensor
- \( z \) = potential head
- \( q \) = source and/or sink
- \( \rho \) = water density at chemical concentration \( C \)
- \( \rho_0 \) = referenced water density at zero chemical concentration
- \( \rho^* \) = density of either the injection fluid or the withdrawn water
- \( \theta \) = moisture content
- \( \alpha' \) = modified compressibility of the medium
- \( \beta' \) = modified compressibility of the water
n = porosity of the medium
S = saturation

**Derivation of a simplified equation from the governing equation**

The original governing equation (Eq. 1) for 3D groundwater flow used in **FEMWATER** is shown in Appendix I. It is assumed that the groundwater is at zero chemical concentration, so \( \rho/\rho_0 \) or \( \rho^*/\rho \), the ratio of water density at chemical concentration \( C \) to the referenced water density at zero chemical concentration, is equal to unity. Therefore Eq. 2 can be reasonably simplified to the following.

\[
F \frac{\partial h}{\partial t} = \nabla \left[ K \left( \nabla p + \nabla z \right) \right] + q
\]  

(3)

Here, \( q \) is the boundary flux, so for an element not at the boundary, \( q \) is zero. When it is assumed that the permeability function is composed by a unique relationship to water content, and after the gradients of the elevation head and the pressure head are combined into the hydraulic gradient, i.e. the total head gradient, the following equation can be obtained.

For a three dimensional case, only \( x \), \( y \) and \( z \) are the directions of interest, thus it can be written as follows,

\[
\frac{\partial}{\partial x} \left( k_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( k_z \frac{\partial h}{\partial z} \right) + q = F \frac{\partial h}{\partial t}
\]  

(4)

In which

\( k_x, k_y, k_z \) – coefficient of permeability with respect to the water phase in \( x \), \( y \), \( z \) directions, respectively
$h$ – hydraulic head (total head)

$q$ – flux at the boundary

$\theta$ - volumetric water content

when only isotropic flow is considered and so the water permeability $k_w = k_x = k_y = k_z$, Eq. 4 can be rewritten as:

$$k_w \left( \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} \right) + \frac{\partial}{\partial x} \left( \frac{\partial k_w}{\partial x} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( \frac{\partial k_w}{\partial y} \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( \frac{\partial k_w}{\partial z} \frac{\partial h}{\partial z} \right) + q = F \frac{\partial h}{\partial t}$$

(5)

In which

$k_w$ – water permeability with respect to moisture content

According to Houston and Houston (1995), it was found through a number of laboratory and field infiltration tests that the gravity gradient was relatively more important compared to the suction gradient. Moreover, it may be useful to understand one-dimensional (1D) flow for ease of interpretation of computed results from any 3D analysis. Thus the equation of 1D flow can be proposed in the following form:

$$\frac{\partial^2 h}{\partial z^2} + \frac{\partial k_w}{\partial z} \frac{\partial h}{\partial z} + \frac{q}{k_w} = F \frac{\partial h}{\partial t}$$

(6)

Eq. 6 implies that the ratio of boundary flux to water permeability, $q/k_w$, can have great influence on groundwater response.