Modified residual soil for the fine-grained layer of capillary barriers

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MODIFIED RESIDUAL SOIL FOR THE FINE-GRAINED LAYER OF CAPILLARY BARRIERS

I GDE BUDI INDRAWAN

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
NANYANG TECHNOLOGICAL UNIVERSITY
SINGAPORE**

2005

Modified Residual Soil for the Fine-Grained Layer of Capillary Barriers

I Gde Budi Indrawan

School of Civil and Environmental Engineering

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Abstract

Capillary barrier, which is commonly defined as an earthen cover system consisting of a fine-grained soil overlying coarse-grained soil, has potential to be used as a slope protection technique. The contrast in hydraulic properties of the fine-grained layer and coarse-grained layer under unsaturated conditions impedes the downward water movement into the underlying protected system as demonstrated in this study.

The Bukit Timah residual soil has been considered to be a suitable material for the fine-grained layer of capillary barrier system. Although residual soils are abundant in Singapore, it has high fine contents that cause the residual soil to have a low permeability and high shrinkage characteristics upon drying. In order to improve the hydraulic properties and the volumetric shrinkage characteristics, the residual soil was mixed with coarse-grained contents (i.e., gravelly sand or medium sand) or lime at different percentages.

Results of the laboratory tests showed that the saturated permeabilities of the residual soil-gravelly sand mixtures and the residual soil-medium sand mixtures increased with the increase in the coarse-grained contents, while the saturated permeabilities of the residual soil-lime mixtures decreased with the increase in the lime percentages. The air-entry values of the soil-water characteristic curves (SWCCs) of the residual soil-gravelly sand mixtures and the residual soil-medium sand mixtures decreased with the increase in the coarse-grained contents and increased with the increase in the lime percentages. The permeability functions of the soil mixtures became steeper with the increase in the percentages of coarse-grained content and lime.

Laboratory infiltration tests conducted through an acrylic soil column of 0.19 m in diameter and 1 m in height, which was equipped with rainfall simulator, tensiometer-transducer system, Time Domain Reflectometry (TDR) waveguides,

and electronic weighing balance, showed that a soil mixture of 50% residual soil and 50% gravelly sand and a soil mixture of 75% residual soil and 25% medium sand were more effective than a soil mixture of 95% residual soil and 5% lime in maintaining matric suctions during a wetting process. However, for the effectiveness in draining water during a drying process, the soil mixture of 95% residual soil and 5% lime was found to be more effective than the soil mixture of 50% residual soil and 50% gravelly sand and the soil mixture of 75% residual soil and 25% medium sand. The comparison of the numerical simulation results and experimental data showed that there was a good agreement between the profiles of pore-water pressure head and volumetric water content as obtained in the numerical simulations and those measured in the laboratory experiments, suggesting that the soil properties used in the numerical simulations were representative of soil conditions in the laboratory and in the field.

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Chapter 1

Introduction

1.1. Research Background

Infiltration is an important controlling factor in rain-induced landslides. Water that infiltrates into an unsaturated zone will result in formation of a wetted zone that may lead to slope failure.

There are a number of preventive methods applicable for preventing or minimizing rainwater infiltration into a slope. One of the possible methods is by applying vegetation cover. This method has been widely used considering the capability of roots to extract soil water and then to release the water to the atmosphere via transpiration. In addition, the roots also reinforce the soil inducing extra soil shear strength and reducing the susceptibility of the soil from erosion. However, the capability of the vegetation cover to prevent water infiltration in tropical areas, which are characterized by intense and prolonged rainfalls, is limited and sometime needs to be combined with other preventive methods.

Another possible method to prevent rainwater infiltration is by applying chunam (i.e., a thin layer of low permeability, cement-lime stabilized, decomposed granite) or shotcrete across a slope surface. However, this type of man-made slope cover systems have several intolerable limitations such as very susceptible to cracks, reducing the aesthetic aspect of the protected slope as the surface of the chunam or the shotcrete cannot be vegetated, and inducing urban heat effect.

Apart from the preventive methods mentioned above, the possibility of using capillary barriers as a method for slope protection looks promising. In fact, capillary

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barriers have been widely applied in geo-environmental engineering as an earthen cover for a landfill system to reduce water infiltration into protected waste materials due to their simplicity and long-term stability. A simple capillary barrier system consists of a fine-grained soil overlying a coarse-grained soil. The interface between the fine-grained layer and the coarse-grained layer is under the influence of negative pore-water pressure or matric suction and serves as a barrier that limits the infiltration of water through the system. A capillary barrier system provides a temporary water storage in the fine-grained layer. The accumulated water is generally removed by evapotranspiration (i.e., removal of water via plant transpiration and near-surface evaporation) and by lateral drainage of water along the interface of the fine-grained layer and the coarse-grained layer if the soil is sloped. The capillary barrier is effective if the combined effect of evapotranspiration and lateral drainage exceeds the infiltration from rainfall. In this case, matric suction in the upper layer can be maintained so that breakthrough of water infiltration into the underlying coarse-grained layer does not occur (Stormont and Anderson, 1999).

Results of the one-dimensional infiltration tests conducted by Yang (2002) showed that the residual soil of Bukit Timah Formation was suitable to be used as a fine-grained layer of a capillary barrier system, especially when the coarse-grained layer was a gravelly sand. In fact, local residual soils are readily available in a large quantity. Residual soil has a relatively high fine content that provides a high water-storage capacity and it is quite cohesive so that the soil can be compacted. In addition, the relatively high fine content allows for vegetation to be planted, where it can reduce erosion, provide aesthetics, and modify the rate of evapotranspiration. Therefore, there is a motivation to develop an earthen capillary barrier system using the local residual soil.

However, results of two-dimensional infiltration tests conducted by Tami (2003) showed that the residual soil of Bukit Timah Formation was less effective to be used as the fine-grained layer of capillary barrier. The high fine content and low permeability caused the infiltrating water to be entrapped in the residual soil layer

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due to the difficulty in draining out the water. When the matric suction of the residual soil decreased to a value lower than the water-entry value of the coarse-grained layer, a significant amount of water eventually infiltrated into the coarse-grained layer.

In addition to the problem associated with the low drainage capacity, the residual soil also shows high shrinkage potential that induces surface cracks upon drying. If the cracks are continuous and reach the lower part of the fine-grained layer, the rainwater can easily percolate into the soil interface of the fine-grained layer and the coarse-grained layer and subsequently infiltrate into the coarse-grained layer, indicating breakthrough.

In order to satisfy the needs, the hydraulic properties and the shrinkage characteristics of the residual soil, therefore, need to be improved by modifying the residual soil and the infiltration characteristics of the modified residual soil need to be investigated.

1.2. Objective and Scope of Study

The main purpose of this research is to improve the hydraulic properties and shrinkage characteristics of the residual soil of Bukit Timah Granite. The improvements were performed by mixing the residual soil with gravelly sand, medium sand, or lime at different percentages. The infiltration characteristics of the modified residual soils or the soil mixtures were then investigated in the laboratory using an infiltration column.

On the whole, the study involved three main parts, i.e., field measurements, laboratory works, and numerical analyses. The first part of the study was one-dimensional field infiltration tests, which were conducted in order to verify the capillary barrier effects in the field. The second part of the study was a comprehensive laboratory programme that consisted of investigation of properties of soils used in the study including basic soil properties, hydraulic properties (i.e., saturated permeability k_s , soil-water characteristic curve SWCC, and unsaturated

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permeability k_w function), and shrinkage characteristics. The laboratory works also involved a number of one-dimensional laboratory infiltration tests, which consisted of upward infiltration tests, rapid drawdown tests, and a series of rainfall tests, through a soil column of 190 mm in internal diameter and 1 m in height. The last part of this study was numerical simulations on infiltration tests conducted in the laboratory.

1.3. Outline of the Thesis

This thesis comprises seven chapters and few appendices and is organized as follows:

Chapter 1 presents the research background, objective and scope of study, and outline of the thesis. *Chapter 2* contains a literature review of residual soils, soil mixtures, an introduction of soil cover and capillary barrier systems, study on the capillary barrier models, and application of capillary barrier for slope stabilization.

Chapter 3 presents an overview of several theories and concepts that are applicable to this study. *Chapter 4* describes research programs of the study. Subsequently, results of the field measurements, laboratory works, and numerical simulations conducted in this study are presented in *Chapter 5*. *Chapter 6* provides discussions of the results obtained from the study. Lastly, *Chapter 7* presents conclusions drawn from the study, as well as some recommendations for future studies.

Appendix A contains information related to investigation of barrier effect in the field. Meanwhile, information related to investigation of hydraulic properties is presented in *Appendix B*. *Appendix C* contains information related to the laboratory infiltration tests. Lastly, information related to results of numerical simulations is presented in *Appendix D*.

Chapter 2

Literature Review

2.1. Introduction

Singapore is located between latitudes 1°09_N and 1°29_N and longitudes 103°36_E and 104°25_E and approximately 137 km north of the Equator. The country consists of one main island, which is about 42 km from east to west and 23 km from north to south, and 60 small islands scattered off its north-east and south. The total land area is 647.5 km² (PWD, 1976).

Singapore is in a typical tropical area that is characterized by uniform temperature, high humidity, and abundant rainfalls throughout the year. The average daily temperature ranges from 23.9°C to 30.9°C and the humidity ranges from 80% to 98%. Rain falls throughout the year with the average annual rainfall amounts to some 1600 mm in the southwest and to 2500 mm in the central catchment area (PWD, 1976).

The following paragraphs describe some information relevant to this study. Section 2.2 describes general characteristics of residual soil and the existing residual soils in Singapore. Subsequently, information related to soil mixing as a method for improving soil properties is presented in Section 2.3. Section 2.4 contains information associated with soil cover and capillary barrier system. Study on one-dimensional capillary barrier model and study on capillary barriers using residual soil of Bukit Timah Granite are briefly reviewed in Section 2.5 and 2.6, respectively. Lastly, application of capillary barrier for slope stabilization is described in Section 2.7.

2.2. Residual Soil

This section briefly describes the general characteristics of residual soils and the characteristics of two main residual soils in Singapore.

2.2.1. General characteristics of residual soils

Residual soil is generally defined as a soil-like material derived from the in situ weathering and decomposition of rock which has not been transported from its original location (Blight, 1997). Some characteristics of residual soils are different from those of transported soils. For example, the permeability of residual soils is usually governed by its micro- and macro-fabric and jointing and by superimposed features such as slickenside, termites, or other bio-channels. Meanwhile, the permeability of transported soils is usually related to the granulometry. Furthermore, the conventional concept of a particle size is inapplicable to many residual soils. Particles of residual soils sometimes consist of aggregates or crystals of weathered minerals that breakdown and become progressively finer as the soils undergo advanced stage of weathering processes.

Residual soils include the group of iron-rich materials, which is usually described as laterites or lateritic soils that are very common in tropical areas (Macari and Hoyos, 1996). The high iron concentration causes the residual soils to become red in color and when dried the soils may harden as a result of the cementing action of the iron and aluminum oxides. Therefore, boundaries between the residual soil layers are often undistinguishable.

A profile of residual soil is generally divided into three zones as shown in Figure 2.1. The lower zone consists of fresh and slightly weathered parent rock. The intermediate zone consists of highly weathered materials but exhibits some features of the parent rock. This zone often contains a pedogenic material such as nodules of calcium or iron salts which may give it a mottled or spotted appearance. The upper zone also consists of a highly weathered and leached soil that is often intersected by root channels and reworked by burrowing animals and insects or by cultivation.

Figure 2. 1 Schematic diagram of classification of tropical residual soil profile (Little, 1969).

2.2.2. Residual soils in Singapore

High temperature and large precipitation in tropical areas induce rapid chemical weathering of the rock formations resulting in thick residual soils. In Singapore, residual soils, which are derived from Bukit Timah Granite and Jurong Sedimentary Rock Formations, occupy about two-thirds of the land area. The Bukit Timah residual soils are found in the central and northern parts of Singapore and on Pulau Ubin while the Jurong Formation residual soils are found on the western side of Singapore.

Weathering has produced a thick overburden (i.e., 10 m to 35 m) of residual soil over the parent rock of Bukit Timah Granite (Poh et al., 1985). According to Winn et al. (2001) the residual soil of Bukit Timah Granite can be divided into two main groups. Group I, which is categorized as a fine-grained soil, consists of reddish brown sandy silty CLAY and yellowish brown sandy clayey SILT. The soil group has clay and silt contents up to 75%. Group II, which is categorized as a coarse-

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grained soil, consists of greenish to whitish grey silty clayey SAND and sandy SILT and has sand contents from 50% to 60%. The grain size distribution of the residual soil is shown in Figure 2.2(a). Results of the permeability tests, which were obtained using the in-situ falling head, laboratory falling head, and consolidation test methods, indicate that the coefficients of permeability at saturation, k_s , of the Bukit Timah residual soil range from 10^{-6} to 10^{-9} m/s.

Figure 2.2 Grain size distribution of residual soils (Leong et al., 2003).

The residual soil of Jurong Formation consists of interbedded layers of predominantly medium plastic silty CLAY, clayey SILT, sandy CLAY, and clayey to silty SAND materials. The coefficients of permeability at saturation, k_s , of the

residual soil obtained from in-situ bore hole and laboratory tests range from 10^{-8} to 10^{-11} m/s (Winn et al., 2001). The grain size distributions of the residual soil are shown in Figure 2.2(b).

2.3. Soil Mixtures

Soil mixing has been applied in engineering practice to improve the performance of fine-grained soils. The improvement is generally intended to improve several soil properties, such as soil strength, volume change behaviour, and saturated permeability. The most common materials used as an additive in the soil mixing involve coarse-grained contents (e.g., gravel) and chemical additives (e.g., lime, fly ash, cement, and slag). Several research works on the soil-gravel mixtures and soil-lime mixtures, which are relevant to this study, are briefly described in the following paragraphs.

2.3.1. Soil-gravel mixtures

The effects of addition of gravel on soil properties, such as void ratio, dry density, saturated permeability, and shrink-swell potential have been investigated by several researchers (e.g., Holtz and Lowitz 1957; Holtz 1985; Shakoor and Cook 1990; Shelley and Daniel 1993). Holtz (1985) suggested to use clayey sands and clayey gravels, instead of heavy clay soils for the design of impermeable soil barriers. The coarse-grained contents were considered to be effective in minimizing undesirable shrinkage and cracking that often occur in heavy clay soils.

Compaction tests on sandy, silty, and clayey soils conducted by Holtz and Lowitz (1957) showed that when the gravel content approached two-thirds of the soil mixture, there were insufficient fine-grained contents to completely fill the macro pores developed between the gravels. In addition, the void ratio of the soil mixtures increased with addition of 40% to 50% gravel even though the soil mixtures were compacted at the optimum water content. Shakoor and Cook (1990) investigated the effects of gravel on saturated permeability of silty clay. The results showed that saturated permeability of soil mixtures increased after addition of more than 50% gravel. This finding agreed with the results obtained by Shelley and Daniel (1993) on kaolinite-gravel mixtures and sieved mine spoils containing different

percentages of gravels. The saturated permeability of kaolinite-gravel mixtures and mine spoils started to increase drastically when the treated soils contained more than 60% gravel as shown in Figure 2.3.

Figure 2.3 Effect of gravel content on saturated permeability of kaolinite and mine spoil (Shelley and Daniel, 1993).

2.3.2. Soil-lime mixtures

The improvements of soil properties in soil-lime mixtures are attributed to two basic reactions, i.e., rapid physico-chemical reactions between the lime and clay minerals (known as soil improvement or modification) and long-term fine content-lime pozzolanic reactions (known as lime stabilization) (Rao and Thyagaraj, 2003). Lime modification reactions occur within several hours after the soil mixing and involve the exchange of cations between the fine-grained soil and calcium ions from lime. Enrichment of the calcium ions on the clay surface reduces the diffuse ion layer around the clay particles and modifies the fine-grained soil into a coarser-textured soil. On the contrary, soil-lime pozzolanic reactions are relatively slow and occur when lime is added in excess of the initial consumption of lime value. Siliceous and aluminous compounds in the clay minerals react with calcium of the lime to form

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calcium silicate hydrate and calcium aluminate hydrate gels that coat the soil particles and subsequently crystallize to bond the soil particles.

When lime is added to a fine-grained soil, it generally increases the plasticity index of low plasticity soils and decreases the plasticity of cohesive soils (El-Rawi and Awad, 1981). Furthermore, the swelling characteristic of a soil is reduced and the shrinkage limit is increased. The effect of lime on soil permeability is still not clear. Some researchers concluded that saturated permeability of soil-lime mixtures should increase, whereas others found a reduction in saturated permeability. Osinubi (1998) reported the increase in saturated permeability of lateritic soil-lime mixtures as a result of addition of up to 4% lime due to additional flocculation caused by cation exchange reaction. With addition of more than 4% lime, the saturated permeability of the soil mixtures was found to decrease. The decrease in saturated permeability was considered to be caused by the formation of insoluble calcium silicates or aluminates or both, which occur due to reaction between calcium ions of the lime and reactive silica or alumina, or both, causing obstruction to water flow through the soil voids.

El-Rawi and Awad (1981) conducted saturated permeability tests on sandy silty clay-lime mixtures and river sand-lime mixtures. For sandy silty clay-lime mixtures, saturated permeability of the soil-lime mixtures compacted at optimum water content and wet of optimum increased as a result of addition of up to 3% lime, after which the saturated permeability of the soil-lime mixtures decreased. Saturated permeability of soil-lime mixtures compacted at dry of optimum was not much affected by the addition of lime and the saturated permeability started to decrease beyond the addition of 5% lime. For river sand-lime mixtures, the saturated permeability decreased with the increase in lime content for the soil mixtures compacted at dry of optimum, optimum, and wet of optimum water contents. Results of saturated permeability tests conducted by El-Rawi and Awad (1981) are shown in Figure 2.4.

Figure 2.4 Effect of lime content on saturated permeability, (a) Sandy silty clay-lime mixtures; (b) River sand-lime mixtures (El-Rawi and Awad, 1981).

2.4. Soil Cover and Capillary Barrier Systems

This section contains information associated with soil cover and capillary barrier systems. First, a brief review on the development of soil cover system is presented. In the following section, some information related to capillary barrier system is briefly reviewed.

2.4.1. Soil cover system

Soil cover systems have been widely applied to a large number of waste sites, including solid and hazardous waste landfills, closed surface impoundments and reactive mine tailings. The soil covers are used to prevent water infiltration into protected waste materials, minimize upward diffusion of unwanted gases, such as methane, and to reduce oxygen ingress into sulphide-bearing mine waste (Yanful et al. 1999; Simms and Yanful, 1999; Choo and Yanful, 2000). Since the principal design consideration is to create a barrier of fluid migration, most designs feature a barrier layer with a low saturated permeability.

The most common traditional soil cover is a compacted clay barrier. The system offers several advantages, such as low cost and material availability. However, experience with these soil covers shows that the soil covers can suffer from degradation and loss of efficiency overtime. As concluded by Suter et al. (1993), freeze-thaw, shrink-swell, desiccation and subsidence cracking, as well as root and animal intrusion may increase the permeability of the clay.

Recently, synthetic materials, i.e. geomembranes and geosynthetic clay liners, have also been incorporated into designs. Although these multicomponent covers achieve specific functions, however, the systems can be expensive and require strict quality-control procedures, which may significantly increase costs (Morris and Stormont, 1997). The systems also introduce weaknesses during construction and introduce planes of weakness with adjacent materials. In addition, their long term durability and performance are yet to be proven (Stormont, 1996).

2.4.2. Capillary barrier system

This section provides a general review on the development of capillary barrier system. The review describes introduction to capillary barrier and concept of capillary barrier.

2.4.2.1. Introduction to capillary barrier

Capillary barrier is commonly defined as an earthen cover system consisting of a fine-grained soil overlying a coarse-grained soil. The difference in particle size of the fine-grained layer and the coarse-grained layer results in the difference in the hydraulic properties (i.e., soil-water characteristic curve and permeability function) of the soils across the interface of the fine-grained layer and the coarse-grained layer. With a low pore-water pressure at the soil interface under unsaturated conditions, the coarse-grained layer generally has a much lower permeability than the fine-grained layer. The low permeability of the coarse-grained layer limits the downward movement of water and holds the infiltrating water in the fine-grained layer by capillary force.

Capillary barrier is receiving more attention as a possible alternative to compacted clay layers and synthetic materials. The systems have been proposed due to their simplicity and long-term stability. Since capillary barriers do not rely on a low saturated permeability, thus, processes that increase saturated permeability will not affect the performance of the capillary barrier (Morris and Stormont, 1997).

Numerous research works have been conducted on capillary barrier systems, either through field studies (e.g., Yanfull et al., 1993a, 1993b, 1999; Stormont, 1995, 1996), laboratory experiments (e.g., Miyazaki, 1988; Stormont and Anderson, 1999), or numerical analyses (e.g., Ross, 1990; Oldenburg and Pruess, 1993; Morris and Stormont, 1997; Simms and Yanful, 1999; Khire et al., 2000) to evaluate the performance of capillary barriers.

Capillary barrier can be classified as sloping and non-sloping system (Stormont, 1996). In the non-sloping system, the infiltrating water is removed by evaporation and/or plant transpiration, which is collectively referred to as evapotranspiration, or by percolation into the underlying coarse-grained layer, which represents failure of the barrier. The failure, or sometimes it is called breakthrough, can only be prevented if the soil-water storage capacity of the fine-grained layer is sufficient to store the infiltrating water expected from precipitation. In the sloping system, the

infiltrating water in the fine-grained layer is not only removed by evaporation, plant transpiration, and by percolation, but also by a lateral diversion along a transport layer, which is essentially a gravity-driven unsaturated drainage within the fine-grained layer, before breakthrough occurs.

Capillary barriers can be constructed in various forms, ranging from a simple design consisting of a homogeneous fine-grained soil overlying a coarse-grained soil to more complex designs that include multiple layers of fine-grained and coarse-grained soils (Morris and Stormont, 1997; Stormont and Morris, 1997). In the simplest design of a capillary barrier, the fine-grained layer of a sloping system does not only serve as a lateral diversion medium, but also serves as a rooting medium and a soil-water storage medium (Stormont, 1996). A schematic diagram of a simple capillary barrier in a sloping system is shown in Figure 2.5.

Figure 2.5 Schematic diagram of capillary barrier system (Morris and Stormont, 1999).

2.4.2.2. Concept of capillary barrier

A capillary barrier system relies on the contrast in unsaturated permeability between the fine-grained layer and the coarse-grained layer that impedes downward movement of water infiltration. In saturated conditions, the coefficient of permeability at saturation, k_s , of soils depends greatly on the void ratio, e . Since

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most of the pores in a coarse-grained soil are relatively larger than the pores in a fine-grained soil, thus, the saturated permeability of a coarse-grained soil is generally higher than that of a fine-grained soil. On contrary, in an unsaturated condition, the coefficient of permeability with respect to water phase, k_w , of soils is a function of the void ratio, e , and the degree of saturation, S , (or the water content, w). As the matric suction increases, the water content of the soils decreases. The decrease in water content of a coarse-grained soil is relatively faster and larger than that in a fine-grained soil (see Section 3.3.1). As a result, the permeability of the coarse-grained soil decreases drastically to a very small magnitude that can be lower than that of the fine-grained soil.

If the matric suction of the soil decreases in response to increasing water content, which is induced by a surface flux, the permeability of the fine layer will increase gradually while that of the underlying coarse layer will remain extremely low. As water content of the fine layer increases, breakthrough into the coarser layer occurs when the matric suction at the surface of the coarse layer is reduced to a value that permits water to enter and form a continuous network in an initially dry coarse soil. When the matric suction decreases below this value, the permeability of the coarse layer will increase rapidly. In this case, the fine-grained layer overlying coarse-grained layer arrangement is no longer effective to serve as a barrier to the downward water movement.

Using data in Stormont and Anderson (1999), Khire et al. (2000) showed that the breakthrough will occur only when the matric suction at the surface of the coarse layer decreases to the water-entry value, ψ_w , of the wetting soil-water characteristic curve (noted as B_c in Figure 2.6(a)). The corresponding matric suction is ψ_B and the volumetric water content in the coarse layer is Θ_{BC} . Continuity in pore-water pressure requires that matric suction at the bottommost part of the fine-grained layer and matric suction at the surface of the coarse-grained layer be equal. Thus, the matric suction of the fine-grained layer at the soil interface must be equal to ψ_B and correspond to point B_F on the soil water characteristic curve of the fine-grained layer. The volumetric water content of the fine-grained layer at ψ_B is Θ_{BF} which

actually represents the saturated condition of the fine-grained layer. Figure 2.6(b) shows that even when B_F is achieved, only small amount of water enters the coarse-grained layer since the permeability of the coarse-grained layer is still low.

Figure 2.6 (a) Soil-water characteristic curves; (b) Permeability functions of the fine-grained layer and coarse-grained layer (Khire et al., 2000).

Capillary barrier causes temporary water storage in the fine-grained layer until B_F is reached. The stored water is, then, ultimately removed by evaporation and transpiration, lateral drainage if the interface is sloped, or percolation (breakthrough) into the lower layer (Figure 2.5). A capillary barrier is effective if the combined effect of evapotranspiration and lateral drainage exceeds the infiltration from precipitation (Stormont and Anderson, 1999).

2.5. Study on One-Dimensional Capillary Barrier Model

A number of research works have been undertaken to study the infiltration characteristics of capillary barriers in one-dimensional model. For example, Stormont and Anderson (1999) conducted infiltration experiments in 0.203 m diameter and 1 m high Plexiglas cylinders. Three capillary barrier models consisting of silty sand over pea gravel, concrete sand over pea gravel, and silty sand over concrete sand were tested. The fine-grained layer was 0.7 m in height while the height of the coarse-grained layer was 0.05 m or 0.1 m. From the laboratory experiments Stormont and Anderson (1999) concluded that the type of the coarse-grained layer materials influences the effectiveness of a capillary barrier. The concrete sand as a coarse-grained layer formed a barrier effect for water infiltration, but it permitted breakthrough at a much higher matric suction head than the pea gravel. Therefore, a soil that does not take on appreciable water until the matric suction head was lowered to near zero will have a low breakthrough head and serve as a distinct barrier at the interface of the fine-grained layer and the coarse-grained layer. On the other hand, a soil with a more gradual increase in water content in response to decreasing matric suction may break through progressively with increasing flow of water across the interface of the fine-grained layer and the coarse-grained layer. Furthermore, until the soil near the interface reaches the breakthrough head, the pore-water pressure head profiles during the infiltration tests is controlled by the properties of the fine-grained layer and the infiltration rate. At a relatively low infiltration rate, the pore-water pressure head profile prior to breakthrough is close to equilibrium. However, at a relatively higher infiltration rate, the pore-water pressure heads in the upper portion of the overlying fine-grained soil will increase to accommodate the imposed flux.

Choo and Yanful (2000) carried out laboratory experiments using a plexiglas column with an inner diameter of 0.108 m and a length of 1.022 m. The soil column was packed with two different soil layers. The first soil column model was packed with 0.2 m thick till overlying 0.8 m thick medium sand while the second one consisted of a 0.71 m thick coarse sand overlain sequentially by a 0.18 m thick 92%

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sand - 8% bentonite mixture (by weight) and an uppermost 0.126 m thick fine sand layer. In addition to laboratory experiments, they also performed numerical analyses by simulating vertical flow of water in the capillary barriers using analytical solutions derived from Srivastava and Yeh (1991) and a commercially available finite-element model, SEEP-W (1994). Choo and Yanful (2000) concluded that results of the analytical solutions agreed with the results obtained from the finite-element model. Furthermore, the finite-element models also agreed with those measured in the laboratory experiments at times shorter than 3 days, as the presence of discontinuous water pockets in the draining sand at the residual water content produced “locked-in” or “static” nonequilibrium pressures that were not captured by existing methods used for estimating the unsaturated permeability functions in unsaturated flow modeling.

Yang (2002) studied the infiltration characteristics of capillary barriers through a 1 m high and 0.19 m inner diameter acrylic column. The soil column was packed with several combinations of fine-grained layer overlying coarse-grained layer, such as fine sand overlying medium sand, medium sand overlying gravelly sand, fine sand overlying gravelly sand, clayey sand overlying medium sand, and the residual soil of Bukit Timah Granite overlying sequentially fine sand and gravelly sand. The study showed that a k_s -ratio of 2 to 3 orders of magnitude between the coarse-grained layer and the fine-grained layer was generally effective to produce a capillary barrier effect. Since the maximum negative pore-water pressure head developed at the interface of fine-grained layer and coarse-grained layer was approximately equal to the residual matric suction head of the coarse-grained layer, Yang (2002) suggested that the optimum thickness of the coarse-grained layer required in a capillary barrier is approximately equal to the residual matric suction head of the coarse-grained layer. Yang (2002) also concluded that the thickness of the fine-grained layer only affected the amount of the stored water in the fine-grained layer temporarily and the pore-water pressure profile in the fine-grained layer. The thickness of the fine-grained layer will not affect the integrity of the capillary barrier over a prolonged rainfall.

2.6. Study on Capillary Barrier Model using the Residual Soils of Bukit Timah Granite

Several research works on capillary barrier using the residual soils of Bukit Timah Granite as the fine-grained layer of a capillary barrier system have been conducted in either one-dimensional laboratory model or two-dimensional laboratory model. As mentioned in the last section, Yang (2002) studied the one-dimensional infiltration characteristics of capillary barrier using several combinations of fine-grained layer overlying coarse-grained layer. One of the soil layer combinations used in the study was the residual soil of Bukit Timah Granite overlying sequentially fine sand and gravelly sand. Based on the study, it was concluded that the local residual soils of Bukit Timah Granite were found to be suitable to be used as the fine-grained layer of a capillary barrier system.

Research works on two-dimensional capillary barrier model conducted by Tami (2003) also showed that the residual soil of Bukit Timah Granite was quite effective in restricting the downward movement of water infiltration under tropical rainfall conditions. Results of the laboratory experiments showed that the amount of water infiltration and cumulative breakthrough measured in the capillary barrier model of the residual soil overlying the gravelly sand was less than of those measured in the capillary barrier model of the fine sand overlying the gravelly sand under the same rainfall loading. Although the residual soil caused less cumulative breakthrough of water into the underlying gravelly sand, this phenomenon resulted in large amount of runoff that might cause serious problems associated with surface erosion. In addition to the potential surface erosion problems, Tami (2003) also reported that the residual soil exhibited significant shrinkage behaviour upon drying. The relatively high volume change potential of the residual soil could affect the performance of capillary barriers as cracks developed in some locations within the residual soil layer. If the cracks are continuous and reach the interface between the residual soil and the gravelly sand, the rainwater will easily percolate through the cracks and penetrate into the underlying gravelly sand.

The possibility of surface erosion problems and the high volume change potential of the residual soil of Bukit Timah Granite reported by Tami (2003) suggest that the volume change characteristics and the hydraulic properties of the residual soil need to be improved if the residual soil will be used as the fine-grained layer of a capillary barrier system.

2.7. Application of Capillary Barrier for Slope Stabilization

Many parts of tropical regions are covered by residual soils that are commonly located above the groundwater table where the pore-water pressure is usually negative relative to atmospheric conditions during dry periods (Rahardjo et al., 1995). The presence of negative pore-water pressure or matric suction has been considered to play an important role in the stability of the residual soil slopes. However, the additional shear strength that exists in unsaturated soils due to negative pore-water pressures is often lost as a result of rainwater infiltration into the slope (Gasmo et al., 1999). Therefore, residual soil slopes situated in regions experiencing high seasonal rainfalls, including Singapore, are very susceptible to rainfall-induced slope failures.

As mentioned, various soil cover systems have been used essentially to prevent water infiltration into the underlying system. For this purpose, capillary barrier seems to be the most appropriate soil cover system due to its simplicity and long-term stability. Numerous researchers, e.g. Oldenburg and Pruess (1993), Stormont (1996), and Bronsert and Plate (1997), showed that sloping capillary barriers were successful in laterally diverting the infiltrating water and, thus, maintaining the negative pore-water pressures in the coarse layer, as well as preventing water infiltration into the underlying system. Since the performance of the capillary barrier is considered maximum when it is applied to the sloping condition, thus, it seems that capillary barrier has the potential to be utilized as an alternative slope stabilization method to prevent the rainwater infiltration and to maintain the negative pore-water pressure distribution.

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In accordance with the use of capillary barriers in slope stabilization, numerical analyses conducted by Ong (2002) showed that the coarse-grained soil used to form a capillary barrier needs to have a low water-entry value, ψ_w , (i.e., $\psi_w \leq 1$ kPa) and a steep permeability function. Meanwhile, a fine-grained soil with a low coefficient of permeability at saturation, k_s , ($k_s \leq 10^{-6}$ m/s) is preferred to be used to form a capillary barrier.

Chapter 3

Theoretical Considerations

This chapter briefly describes several theories relevant to this research. Characteristics of unsaturated soil will be described in Section 3.1. Theories related to soil-water characteristic curve and water flow in unsaturated soil will be presented in Section 3.2 and Section 3.3, respectively. Lastly, Section 3.4 presents some information related to shrinkage behaviour of soils.

3.1. Characteristics of Unsaturated Soils

The characteristics of unsaturated soil, which are phases of unsaturated soil, soil suction, and capillarity phenomenon, are briefly described in this section.

3.1.1. The phases of an unsaturated soil

An unsaturated soil is commonly referred to as a three phase system, i.e., solid phase, water phase, and air phase. Recently, the air-water interface, which is commonly referred to as the contractile skin, has been postulated as the additional phase (Fredlund and Rahardjo, 1993a). The most distinctive property of the contractile skin is its ability to exert a tensile pull. It behaves like an elastic membrane under tension interwoven throughout the soil structure.

Fredlund and Rahardjo (1993a) described the existence of the contractile skin such as in a shrinkage test involving the air-drying of a small soil sample. The total stresses on the specimen remain unchanged at zero while the specimen undergoes a decrease in volume. As the specimen dries, the pore-water pressure in the specimen becomes negative. However, the existence of the contractile skin, which acts like a thin rubber membrane, leads to the solid particles to stick together.

3.1.2. Theory of soil suction

Soil suction is commonly referred to as the free energy state of soil water. The soil suction can be measured in terms of the relative humidity and, thus, it is commonly called total suction, ψ . The total suction has two components, which are matric suction and osmotic suction. The relationship between the total suction and its components can be expressed as follows:

$$\psi = (u_a - u_w) + \pi \quad (3.1)$$

where:

- ψ = total suction,
- $(u_a - u_w)$ = matric suction,
- u_a = pore-air pressure,
- u_w = pore-water pressure,
- π = osmotic suction.

The matric suction is associated with the capillary phenomenon arising from the surface tension of water as it is described in the following section, while the osmotic suction is related to the presence of dissolved salts in a soil (Fredlund and Rahardjo, 1993a).

3.1.3. Capillarity

Some information associated with the phenomenon of capillarity and water pressure distribution in a simple capillary tube model, which can illustrate the pore-water conditions in nature, is briefly presented in this section.

3.1.3.1. Capillary height

The pores in unsaturated soil are occupied by water and air. An interfacial tension acts at the interface between these phases and the magnitude of the tension depends on the curvature of the air-water interface, and consequently on the degree of saturation. A simple model of water in a capillary tube can be used to describe the capillary phenomenon in a soil as shown in Figure 3.1(a).

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If a small glass tube is inserted into water under atmospheric conditions, water will rise up in the glass tube as a result of the surface tension in the contractile skin and the adhesive forces (i.e., the tendency of water to wet the surface of the glass tube) between the glass tube and the water until equilibrium is reached between the capillary forces (directed upwards) and the gravitational forces (directed downwards), and a meniscus is formed.

Figure 3.1 Simple model and phenomenon related to capillarity. (a) Capillary tube; (b) Water pressure distribution (Fredlund and Rahardjo, 1993a).

The height of the water in the glass tube can be formulated by analyzing the vertical force equilibrium of the capillary water in the tube. The vertical resultant of the surface tension (i.e., $2\pi r T_s \cos \alpha$) is responsible for holding the weight of water column, which has a height of h_c (i.e., $\pi r^2 h_c \rho_w g$) :

$$2\pi r T_s \cos \alpha = \pi r^2 h_c \rho_w g \quad (3.2)$$

where:

- r = radius of the capillary tube,
- T_s = surface tension of water,

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- α = contact angle,
 h_c = capillary height,
 g = gravitational acceleration.

Thus, the maximum height of water, h_c , can be assessed by rearranging equation (3.2):

$$h_c = \frac{2T_s}{\rho_w g R_s} \quad (3.3)$$

where :

R_s = radius of curvature of the meniscus (i.e., $r / \cos \alpha$).

The contact angle between the contractile skin of pure water and clean glass is zero (i.e., $\alpha = 0$). When the α value is zero, the radius of curvature, R_s , is equal to the radius of the tube, r . Therefore, the capillary height of pure water in a clean glass is

$$h_c = \frac{2T_s}{\rho_w g r} \quad (3.4)$$

Equation (3.4) shows that the capillary height of water is inversely proportional to the radius of the glass tube. The radius of the glass tube is analogous to the pore radius in the soils. The smaller the pore radius in the soil, the higher will be the capillary height of water.

3.1.3.2. Capillary pressure

Figure 3.1(b) shows the water pressure distribution in the capillary tube. Points A, B, and C in the capillary tube are in hydrostatic equilibrium. Since points A and B are located on the water surface that is considered as the datum for the system, the hydraulic heads, h , (i.e., elevation head, h_e , plus pressure head, h_p) at points A and B are equal to zero and the elevation head, h_e , at points A and B are also equal to zero. As a result, the water pressure, u_w , either at point A or at point B, is equal to zero or atmospheric.

Point C is located at a height of h_c from the datum. The hydrostatic equilibrium requires that the hydraulic heads at point A, B, and C be equal (i.e., $h_A = h_B = h_C = 0$). Therefore, the pressure head at point C is equal to the negative value of the elevation head at point C. The water pressure at point C can then be calculated as:

$$u_w = -\rho_w g h_c \quad (3.5)$$

As shown in Figure 3.1(b), the water pressures above point A are negative, while the water pressures below point A are positive due to the hydrostatic pressure condition. Since the air pressure at point C is atmospheric (i.e., $u_a = 0$) and the water pressure is negative (i.e., $u_w = -\rho_w g h_c$), the matric suction, $(u_a - u_w)$, at point C can then be written as,

$$(u_a - u_w) = \rho_w g h_c \quad (3.6)$$

3.2. Soil-Water Characteristic Curve

This section describes several theories related to soil-water characteristic curve (SWCC), as well as its governing equations.

3.2.1. Definitions

Soil-water characteristic curves (SWCC) can be defined as a plot representing the relationship between volumetric water content and suction of a soil. The water content in unsaturated soil is a function of the suction present in the soil. The higher the matric suction is, the lower the water content will be.

SWCCs are commonly measured using pressure plate extractor, Tempe pressure cell, and volumetric pressure plate extractor that operate on the same principle as the pressure plate apparatus as described in ASTM D2325 (2000). According to Fredlund and Rahardjo (1993a), the equipment principally uses the axis-translation technique introduced by Hilf (1956), where air pressure is imposed to the soil specimen that is placed on a high-air entry ceramic disk in a pressure chamber. During the test, the air pressure in the chamber can be raised to a value above

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atmospheric pressure (i.e., above zero gauge pressure) while the water pressure inside the compartment is kept at atmospheric (i.e., $u_w = 0$) by maintaining the water level inside the compartment through the use of burette that is filled with water. The use of a high-air entry disk allows the air phase to be separated from the water phase. The difference between the air and water pressures is known as matric suction. As the air pressure is applied to the soil specimen, the pore-water drains out of the specimen. At equilibrium, the soil has a water content corresponds to the controlled matric suction. Therefore, a soil-water characteristic curve is obtained by measuring water contents of the specimen at different matric suctions.

Figure 3.2 shows a typical plot of a SWCC of a silty soil, along with some key characteristics. The air-entry value or bubbling pressure of the soil, ψ_a , is the matric suction where air starts to enter the largest pores in the soil. The residual water content, θ_r , can be defined as the water content below which a large increase in suction is required to remove additional water from the soil. The corresponding matric suction at this point is called residual soil suction, ψ_r . Based on experiment conducted by Croney and Coleman (1961) and thermodynamic considerations (Richards, 1965), the total suction corresponding to zero water content for a variety of soils is close to a value of 10^6 kPa.

Figure 3.2 also illustrates two curves, the main curve is a drying or desorption curve, while the other one is a wetting or adsorption curve. Both curves have a similar form; however, they are different in the water content for a given matric suction as a result of hysteresis. Furthermore, the end point of the adsorption curve may differ from the starting point of the desorption curve.

Hysteresis is a common feature of an unsaturated soil. Drying and wetting SWCCs describe drying and wetting processes occurred in the soil. In a drying process, when the matric suction of a soil increases, air replaces water in soil pores, and, thus, the water content of the soil decreases.

Figure 3.2 SWCC along with some its key characteristics (modified from Fredlund and Xing, 1994).

On contrary, in the wetting process water replaces air. As a result, water content of the soil increases, while the matric suction decreases. However, during the wetting process, the wetting path of SWCC does not follow the drying path of SWCC in the reverse direction. Several factors, such as the non-uniformity of pore size distribution in the soil, the presence of entrapped air in the soil, and the difference in contact angle between an advancing interface during the wetting process and a receding interface during the drying process, are considered as the main causes for hysteresis in the SWCC (Fredlund and Rahardjo, 1993a).

Typical drying SWCCs for different soils are shown in Figure 3.3. The saturated volumetric water content, θ_s , and the air-entry value, ψ_a , generally increase with the plasticity of the soil. Other factors such as stress history are considered to affect the shape of the SWCC (Fredlund and Xing, 1994).

Figure 3.3 Typical soil-water characteristic curves for various soils (modified from Fredlund and Xing, 1994).

3.2.2. Equation for soil-water characteristic curve

Recently, SWCC is receiving more attention because it has been accepted that there is a relationship between the SWCC and the properties of unsaturated soil. For example, the coefficient of permeability with respect to water phase, k_w , is generally assumed to be uniquely related to the SWCC, as shown in Figure 3.4. The relationships indicate that the coefficient of permeability with respect to water phase, k_w , of a soil decreases to magnitudes lower than the coefficient of permeability at saturation, k_s , when the soil starts to desaturate.

A number of equations have been proposed to establish the SWCC functions. One of the equations that has been widely used is the equation proposed by Fredlund and Xing (1994). The equation has been proven to accurately fit with experimental data over the entire suction range (Leong and Rahardjo, 1997a).

Fredlund and Xing (1994) attempted to establish a theoretical basis for the shape of the SWCC by considering the pore-size distribution curve for the soil.

Figure 3.4 Relationship between the SWCC and the permeabilities of sand and clayey silt (Fredlund and Rahardjo, 1993a).

The relationship between the volumetric water content and suction of the soil is expressed as follows:

$$\theta_w = C(\psi) \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{(u_a - u_w)^n}{a} \right) \right] \right\}^m} \quad (3.7)$$

where:

- θ_w = volumetric water content,
- θ_s = volumetric water content at saturation,
- $C(\psi)$ = a correction factor, which is given by :

$$C(\psi) = \frac{\ln(1 + (u_a - u_w)/(u_a - u_w)_r)}{\ln[1 + (1,000,000/(u_a - u_w)_r)]} \quad (3.8)$$

- e = natural number, 2.71828,

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- a = soil parameter related to the air entry value of the soil, ψ_a (kPa),
 n = soil parameter related to the slope at the inflection point of the SWCC,
 m = soil parameter related to the residual water content, θ_r ,
 $(u_a - u_w)$ = matric suction (kPa),
 $(u_a - u_w)_r$ = matric suction that corresponds to the residual volumetric water content, θ_r (kPa).

Parameter a is closely related to the air-entry value, ψ_a , of the soil. Generally, parameter a will have a higher value than the air-entry value, ψ_a . However, for small values of m , parameter a can be used to define the air-entry value, ψ_a , of the soil (Fredlund and Xing, 1994). Parameter m is related to the value of the residual volumetric water content, θ_r , of the soil, while parameter n controls the slope of the SWCC.

Leong and Rahardjo (1997a) recommended that Equation (3.7) can be used with the correction factor, $C(\psi)$, equal to 1, since the term ψ_r in the correction factor $C(\psi)$ affects the initial portion of the SWCC. Therefore, the equation was then revised to:

$$\theta_w = \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{(u_a - u_w)^b}{a} \right) \right] \right\}^c} \quad (3.9)$$

where a , b , and c are constants and represent the parameters of a , n , and m in Equation (3.7), respectively.

3.3. Water Flow in Unsaturated Soils

The governing equations of water flow in an unsaturated soil and in seepage analyses, as well as the methods used in the prediction of permeability functions, are briefly described in this section.

3.3.1. Flow law for unsaturated soils

Darcy's law is commonly used to describe the flow of water in a saturated soil. However, as concluded by Buckingham (1907), Richards (1931), and Childs and Collis-George (1950) the law is also applicable to the flow of water through an unsaturated soil. Darcy's law states that the rate of water flow through a soil mass is proportional to the hydraulic head gradient as follows:

$$v_w = -k_w \frac{\partial h_w}{\partial y} \quad (3.10)$$

where:

v_w = flow rate of water,

k_w = coefficient of permeability with respect to water phase,

$\partial h_w / \partial y$ = hydraulic head gradient in the y-direction, which can be designated as

$$i_{wy}$$

The coefficient of permeability with respect to water phase, k_w , is a measure of the space available for water to flow through a soil. The negative sign indicates that water flows from a high hydraulic head to a low hydraulic head.

In a saturated soil, the coefficient of permeability at saturation, k_s , is relatively constant and a function of the void ratio, e (Lambe and Whitman, 1979). Since the pores in a coarse-grained soil are relatively larger than the pores in a fine-grained soil, therefore the saturated permeability of the coarse-grained soil is generally higher than that of the fine-grained soil. On the other hand, in an unsaturated soil, the coefficient of permeability with respect to water phase, k_w , is a variable and affected by combined changes in the void ratio, e , and the degree of saturation, S , (or water content, w) of the soil. Again, since water flows only through the voids that are filled with water, the percentage of the voids filled with water is an important factor. Refer to SWCCs for the sandy soil and the clayey soil as shown in Figure 3.3 where the sandy soil represents a coarse-grained soil while the clayey soil represents a fine-grained soil. By definition, SWCC relates the volumetric

water content of a soil to the matric suction. As the matric suction increases, the water content of the soil decreases. The first pores of a soil to be emptied are the relatively large ones that cannot retain water (Hillel, 1998). Since most of the pores of a coarse-grained soil are relatively large, a small increase in matric suction causes the water content of the soil to decrease rapidly. Therefore, the permeability of the soil also decreases rapidly. In the fine-grained soil, although most of the pores are relatively small, the pore size distributions of the fine-grained soil are more uniform than that of the coarse-grained soil, so that an increase in the matric suction causes a more gradual decrease in the water content of the soil. Thus, the permeability of the fine-grained soil also decreases gradually to a value that is relatively higher than that of the coarse-grained soil.

3.3.2. Governing equations for seepage analysis

The slow movement of water through soil is commonly referred to as seepage or percolation. As mentioned in the last section, the flow of water through saturated and unsaturated soils is governed by Darcy's law. Also, the formulation of the partial differential flow equation is similar in the case of saturated soils (Fredlund and Rahardjo, 1993a). The main difference between water flow in saturated and unsaturated soils is that the coefficient of permeability at saturation, k_s , is assumed to be constant for saturated soils, while the coefficient of permeability with respect to water phase, k_w , for unsaturated soils is a function of water content or matric suction, as explained in Section 3.3.1. In addition, the pore-water pressure in a saturated soil is generally positive, while the pore-water pressure in an unsaturated soil is negative.

The governing partial differential equation for unsteady-state water flow in an isotropic soil is expressed as follows (Fredlund and Rahardjo, 1993a):

$$\frac{\partial u_w}{\partial t} = -C_w \frac{\partial u_a}{\partial t} + c_v^w \frac{\partial^2 u_w}{\partial x^2} + \frac{c_v^w}{k_w} \frac{\partial k_w}{\partial x} \frac{\partial u_w}{\partial x} + c_v^w \frac{\partial^2 u_w}{\partial y^2} + \frac{c_v^w}{k_w} \frac{\partial k_w}{\partial y} \frac{\partial u_w}{\partial y} + c_g \frac{\partial k_w}{\partial y} \quad (3.11)$$

where:

u_w = pore-water pressure,

u_a = pore-air pressure,

t = time,

C_w = $(1 - m_2^w / m_1^w) / (m_2^w / m_1^w)$; interaction constant associated with the water phase partial differential equation,

m_1^w = coefficient of water volume change with respect to a change in net normal stress, $(\sigma - u_a)$,

σ = total stress,

m_2^w = coefficient of water volume change with respect to a change in matric suction, $(u_a - u_w)$,

c_v^w = $k_w / (\rho_w g m_2^w)$; coefficient of consolidation with respect to the water phase for the x - and y -directions,

k_w = coefficient of permeability with respect to water phase for the x - and y -directions (i.e., isotropic soil),

ρ_w = density of water,

g = gravitational acceleration,

c_g = $1 / m_2^w$; called the coefficient associated with the gravity term and only applicable to the flow of water in the y -direction since this term is derived from the elevation head gradient.

3.3.3. Permeability functions

The coefficient of permeability at saturation, k_s , is constant and a function of void ratio, e , while the coefficient of permeability with respect to water phase, k_w , is not only a function of void ratio, e , but also a function of water content, w , of the soil (Lambe and Whitman, 1979) .

For an unsaturated soil, the coefficient of permeability with respect to water phase, k_w , can be obtained by either direct measurement or indirect measurement. Direct measurement of permeability in the laboratory is a tedious and time-consuming process, particularly when the soil has reached low water content conditions. Indirect measurements of permeability are usually performed by establishing

permeability functions based on the fact that the coefficient of permeability with respect to water phase, k_w , is a relatively unique function of the volumetric water content, θ_w , which in turn, depends on the suction, i.e. either matric suction, $(u_a - u_w)$ or total suction, ψ , which presents in soil. As a result, numerous empirical and statistical equations have been proposed to predict the permeability function for an unsaturated soil using the coefficient of permeability at saturation, k_s , and the SWCC of the soil. In this section, three widely used methods to calculate the permeability functions are described.

3.3.3.1. Brooks and Corey (1964) method

The method of permeability prediction proposed by Brooks and Corey (1964) is based on the fact that the SWCC is an indication of the configuration of water-filled pores. The procedure of this method involve dividing the SWCC of the soil into “ m ” equal intervals of volumetric water content as shown in Figure 3.5.

Figure 3.5 Prediction of the permeability function from the SWCC (from Fredlund and Rahardjo, 1993a).

The matric suction corresponding to the midpoint of each interval is then used to predict the permeability function $k_w(\theta_w)$ based on the following equation:

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$$k_w(\theta_w)_i = \frac{k_{sm}}{k_{sc}} \frac{T_s^2 \rho_w g}{2\mu_w} \frac{\theta_s^p}{N^2} \sum_{j=i}^m \left\{ (2j+1-2i)(u_a - u_w)^{-2} \right\} \quad (3.12)$$

where:

- $k_w(\theta_w)_i$ = calculated coefficient of permeability with respect to water phase (m/s) for a specified volumetric water content, $(\theta_w)_i$, corresponding to the i^{th} interval.
- i = interval number which increases with the decreasing volumetric water content; for example, $i = 1$ identifies the first interval that closely corresponds to the volumetric water content at saturation, (θ_s) ; $i = m$ identifies the last interval corresponding to the lowest volumetric water content, (θ_L) , on the experimental SWCC.
- j = a counter from “ i ” to “ m ”.
- k_{sm} = measured coefficient of permeability at saturation (m/s).
- k_{sc} = calculated coefficient of permeability at saturation (m/s).
- T_s = surface tension of water (kN/m).
- ρ_w = water density (kg/m³).
- g = gravitational acceleration (m/s²).
- μ_w = absolute viscosity of water (N s /m²).
- θ_s = volumetric water content at saturation (i.e., $S = 100\%$) (Green and Corey, 1971a).
- p = a constant which accounts for the interaction of pores of various sizes; the magnitude of “ p ” can be assumed to be equal to 2.0 (Green and Corey, 1971a).
- m = total number of intervals between the volumetric water content at saturation, θ_s , and the lowest volumetric water content, θ_L , on the experimental SWCC.
- N = total number of intervals computed between the volumetric water content at saturation, θ_s , and zero volumetric water content (i.e., $\theta_w = 0$).
- $(u_a - u_w)_j$ = matric suction (kPa) corresponding to the midpoint of the j^{th} interval.

The complete procedure for calculating the permeability function using this method is described in detail in Fredlund and Rahardjo (1993a).

3.3.3.2. Fredlund et al., (1994) method

Fredlund et al., (1994) proposed a method of permeability function prediction based on the fact that both permeability function and the SWCC can be determined from the pore-size distribution of the soil and the fact that the ease of water flow through the soil is a function of the amount of water in the soil matrix. The method offers an accurate and wide- range of permeability function prediction, since it is based on the entire SWCC (i.e., from saturation down to a water content of zero or suction of 10^6 kPa). In addition, the method does not involve any evaluation of the residual water content as there does not appear to be a generally accepted procedure for determining the residual water content of a soil. The proposed equation is given by:

$$k_r(\psi) = \Theta^q(\psi) \frac{\int_{\ln(\psi)}^b \frac{\theta_w(e^y) - \theta_w(\psi)}{e^y} \theta'_w(e^y) dy}{\int_{\ln(\psi_s)}^b \frac{\theta_w(e^y) - \theta_s}{e^y} \theta'_w(e^y) dy} \quad (3.13)$$

where:

$k_r(\psi)$ = the relative coefficient of permeability defined as:

$$k_r(\psi) = \frac{k(\psi)}{k_s} \quad (3.14)$$

$\Theta^q(\psi)$ = a correction factor,

Θ = the normalized volumetric water content, which is equivalent to degree of saturation and defined as:

$$\Theta = \frac{\theta_w}{\theta_s} = S \quad (3.15)$$

q = 1 (Kunze et al., 1998),

θ_w = the volumetric water content,

θ_s = the volumetric water content at saturation,

- θ'_w = the derivative of Equation (3.7),
 ψ_a = the air-entry value of the soil under consideration,
 b = $\ln(1,000,000)$,
 y = a dummy variable of integration representing suction,
 e = natural number, 2.71828.

3.3.3.3. Leong and Rahardjo (1997b) method

The Leong and Rahardjo (1997b) method was essentially proposed to best fit the predicted permeability data. The method was based on a generalized relationship between the relative coefficient of permeability, k_r , and the normalized volumetric water content of the soil, Θ , proposed by previous researchers. The relationship is expressed as follows:

$$k_r = \Theta^p \quad (3.16)$$

where:

- k_r = relative coefficient of permeability or ratio of coefficient of permeability with respect to water phase, k_w , and coefficient of permeability at saturation, k_s ,
 Θ = normalized volumetric water content or $(\Theta_w - \Theta_r)/(\Theta_s - \Theta_r)$, where the subscripts s and r denote saturated and residual, respectively,
 p = is a constant.

Since the normalized volumetric water content of the soil, Θ , has been proven to be a function of matric suction, $(u_a - u_w)$, thus, k_r must also be a function of the matric suction, $(u_a - u_w)$. Furthermore, by adopting the SWCC equation proposed by Fredlund and Xing (1994) with $C(\psi) = 1$, the Equation (3.15) can be rearranged as follows :

$$k_r = \frac{1}{\left\{ \ln \left[e + \left(\frac{(u_a - u_w)^b}{a} \right) \right] \right\}^{cp}} \quad (3.17)$$

where a , b , and c are constants as determined in Equation (3.9). The parameter p has quite a wide range of values and it is determined by best fitting the permeability function with the predicted permeability data. Fredlund et al., (2001) presented the analyses of a number of experimental data set and provided guidance for selection of the most suitable value for parameter p , as listed in Table 3.1.

Table 3.1 Statistics of parameter p for various soils (Fredlund et al., 2001).

Statistics	Clay	Clay loam	Loam	Sand	Sandy clay loam	Silty clay	Silty clay loam	Silt loam	Sandy loam	Loamy sand
Mean	4.34	3.58	3.78	2.37	2.80	5.59	3.22	3.52	2.86	2.67
Median	4.71	2.62	3.56	2.36	2.62	4.77	3.18	3.46	2.85	2.59
Mode	3.00	2.80	3.25	2.20	2.75	4.60	4.05	3.15	2.75	4.63
Standard deviation	1.50	1.81	1.15	0.49	1.00	1.31	1.36	1.09	0.84	0.68
Sample variance	2.25	3.29	1.34	0.24	0.99	1.73	1.86	1.19	0.71	0.46
Range	5.42	5.92	4.14	2.25	4.71	6.33	4.97	5.99	4.20	4.01
Minimum	1.52	1.84	1.63	1.25	0.64	1.11	1.28	0.83	1.02	1.41
Maximum	6.94	7.76	5.76	3.49	5.35	7.44	6.25	6.82	5.22	5.42
Number of sets	21	18	12	49	17	34	18	74	30	29

3.4. Shrinkage Curve

The shrinkage curve for a soil is part of the constitutive behaviour of the soil and has a relationship to other unsaturated soil property functions (Fredlund et al., 2002). The shrinkage curve can be used in the determination of volume-mass unsaturated soil property functions that are generally required in numerical analyses involving unsaturated soils.

According to Marinho (1994), there are three basic types of shrinkage curve that an initially saturated specimen can exhibit, as shown in Figure 3.6. A specimen may shrink at different water contents. Curve A illustrates a specimen that shrinks to its smallest volume while under a saturated condition. Meanwhile, curve B and curve C show a specimen that changes in volume once the specimen starts to desaturate. In other words, as the water content of a specimen decreases, the volume of the specimen also decreases and reaches the smallest value when the water content is

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close to zero. At a certain water content, the decrease in the volume of the specimen represented by curve B is greater than that of the specimen represented by curve C. The different soil shrinkage behaviour mainly depends on particle size distribution and stress history.

A typical behaviour of the drying of a slurried soil is shown in Figure 3.7. The soil specimen is initially fully saturated. When the drying process occurs, initially the shrinkage curve follows the saturation line until air begins to enter the soil voids. The point when the shrinkage curve starts to move away from the saturation line corresponds to air-entry value, ψ_a , of the soil. As the soil continues to dry, the void ratio reaches a minimum value at which there is no further volume change. The water content at the intersection between the minimum void ratio and the saturation line is generally referred to as the Shrinkage Limit.

Figure 3.6 Three basic characteristics of shrinkage curves (Marinho, 1994).

Figure 3.7 Volume-mass relationships for the drying curve of an initially slurried soil specimen (Fredlund et al., 2002).

Chapter 4

Research Programme

This chapter describes the research programme adopted in this study in detail. The study consists of three main programs, which are investigation of barrier effect in the field, laboratory investigation program and numerical analysis. The field investigation program is described in Section 4.1, whilst the laboratory investigation program and numerical analysis are discussed in Section 4.2 and Section 4.3, respectively.

4.1. Investigation of Barrier Effect in the Field

Field infiltration tests were conducted in order to verify the performance of the capillary barrier model developed in the laboratory. To do so, one of the capillary barrier models developed in the previous study (Yang, 2002), i.e. medium sand overlying gravelly sand, was simulated in the field setting. The one-dimensional field infiltration test was conducted using a double-ring infiltrometer and pore-water pressure and volumetric water content measuring devices. As comparison, another field infiltration test was also performed on the original sand layers which represented a non-capillary barrier system.

The field infiltration tests were conducted at Changi Reclamation Area 'A' Contract 3 (Appendix A), which was located in the eastern part of Singapore. The area was reclaimed using sand from offshore source. The total thickness of the sand of the reclaimed area was about 12 m and the depth of water table was about 3 m below the ground surface.

4.1.1. Determination of the field soil density and setting up the test

One of the field investigation programs was a field infiltration test conducted through a 60 cm thick reclamation sand, named medium sand of Changi (MSoC). As the measuring probes needed to be installed at certain intervals in the sand layer, the medium sand of Changi was first removed by excavating a pit of 0.5 x 0.5 square meter area. After installing the probes, the medium sand of Changi was then re-compacted to its initial density. In order to obtain the same sand density, therefore, it was necessary to determine the initial field density of the medium sand of Changi prior to excavation.

The field density of the medium sand of Changi was measured using the sand replacement method in accordance with ASTM Designation D 1556. For this purpose, a Sand-Cone Density Apparatus and Ottawa Sand were used. The sand cone apparatus, which was manufactured by ELE International, consisted of a 11.5 cm diameter and 30 cm long cylinder that was connected to a metal funnel of 12 cm high and a metal base plate with a flanged center hole for receiving the funnel (Figure 4.1).

The metal base plate was placed on a flat surface and a hole was hand-excavated through the center hole of the base plate. The total mass of the replaced medium sand was then weighed using an electronic weighing balance (Shinko®, DJ-6000P Model), which had a maximum capacity of 6 kg and an accuracy of 0.1 g.

The sand cone was filled with a sufficient amount of Ottawa Sand and the initial mass of the Ottawa Sand and the sand cone was measured. The Ottawa Sand was then allowed to flow from the sand cone, which had been placed on the base plate, into the dug hole by opening the cylindrical valve. Once the sand inside the sand cone stopped flowing, the mass of the sand cone and the remaining Ottawa Sand was measured. From the predetermined dry density of the Ottawa Sand and the mass of the Ottawa Sand poured into the hole, the volume of the dug hole was computed. The field density of the reclaimed sand was then computed from the

calculated volume of the dug hole and the total mass of the replaced medium sand from the hole.

Figure 4.1 Sand cone apparatus and the metal base plate.

Once the field density of the existing sand was determined, three pits of 60 cm deep were dug in a parallel setting (Figure 4.2). The biggest one, which was provided for the placement of the tensiometer-bourdon gauge systems (i.e., pore-water pressure measuring devices) and Trase system (i.e., a volumetric water content measuring device) and the area of the measurement, was one square meter area, while the two smaller pits of 0.5 x 0.5 square meter area were provided for setting up the models.

Figure 4.2 Schematic diagram of the field infiltration test.

4.1.2. Infiltration measurement apparatus

The one-dimensional field infiltration tests were conducted through two sets of double ring infiltrometer (Figure 4.3). One set of double-ring infiltrometer, which had 30 cm inner-ring diameter and 55 cm outer-ring diameter, was placed above 60 cm thick of the existing medium sand. The other set, which had 32 cm inner-ring diameter and 57 cm outer-ring diameter, was placed above the capillary model of the existing medium sand of Changi overlying gravelly sand. Using a driving plate, the inner ring was driven to a depth of 10 cm, while the outer ring was hammered to a depth of 15 cm below the ground surface. By this configuration, it was expected that a one-dimensional downward flow of water occurred below the inner ring, and water that infiltrated through the outer ring acted as a barrier that reduced lateral movements of water from the inner ring.

Small tip tensiometers (Model 2100F Soilmoisture Probe, Soilmoisture Equipment Corporation, CA, USA) were used to study the development of pore-water pressure during the field infiltration tests. Basic components of a tensiometer include a porous ceramic cup, a plastic body tube, and a pressure measuring device (Figure 4.4).

The porous ceramic cup (Soilmoisture 2100F-200CR), which has a high air-entry value of 1Bar, is 6mm in diameter and 25mm in length and it is attached to the plastic body tube via a coaxial polyethylene tube assembly of 1800mm in length. Before use, the ceramic cup was submerged in water in a vacuum container for saturation.

The tensiometer-bourdon gauge system was designed for field measurement, where the readings were taken manually through a vacuum dial or bourdon gauge (Soilmoisture 2060FG4). The vacuum dial gauge can be used to measure matric suctions ranging from 0kPa to 90kPa.

After installation of tensiometer or during the test, a highly negative pore-water pressure may develop and lead to development of air bubbles within the

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tensiometer. If air bubbles are allowed to accumulate in the tube, the transducer will read the pore-water pressure inaccurately. As the air bubbles need to be removed regularly, a jet fill water reservoir was then installed on top of the plastic tube for easy servicing and flushing.

Figure 4.3 Field infiltration test apparatus.

Figure 4.4 Tensiometer-bourdon gauge system.

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Before use, the sensitivity of the tensiometer-bourdon gauge was verified by inserting the ceramic cup into a soil. A sensitive system, which was indicated by fast movement of the pointer on the dial gauge, was, then, selected for the pore-water pressure measurement.

The volumetric water content of the soil was measured using a portable Trase system of 6050X1 Model (Soilmoisture Equipment Corporation, CA, USA). The system uses Time Domain Reflectometry (TDR) to determine the velocity of the electromagnetic pulse of energy transmitted into the soil through a transmission line, which consists of coaxial cable and waveguides inserted into the soil. The velocity of the microwave pulse traveling in the soil is a measure of the apparent dielectric constant, K_a , of the material in contact with and surrounding the waveguides. The higher is the dielectric constant, the slower will be the velocity. In general, soil is composed of air, mineral and organic particles, and water which have the dielectric constant, K , ranging from 1 for air, between 2 to 4 for most mineral and organic particles, and 80 for liquid water. Since the great difference in the dielectric constant of water from other soil components, the travel velocity of a microwave pulse is very dependent on the water content of the soil. By knowing the relationship of the K_a value to the volumetric water percentage that is commonly established by careful measurements of K_a in test cells prepared with known volumes of water in the soil, a K_a reading can be converted to the volumetric water content of the soil.

The Trase model is specially designed for field use where the commands are entered through the keypad, data are output to the display screen, and electrical power is supplied from two sets of sealed, gelled electrolyte rechargeable batteries with capacity 6 Volt, 7 Ampere Hour. The Trase has a volumetric water content measuring range from 0 to 100% and can be operated in a temperature range of 0 to +45°C.

The pulse is transferred through one set of 15 cm long stainless steel waveguides that was inserted into waveguide sockets, which are provided in the waveguide

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connector (Soilmoisture 6002F1 Model, Figure 4.5). When making readings, the BNC connector on the coaxial cable assembly of the waveguide connector is plugged into the BNC Port located on the control panel of the TDR processor. The Trase has a standard 256 kb memory with a storage capacity greater than 5,400 readings. In the field infiltration test, the waveguide rods were only placed at the interface of medium sand of Changi and gravelly sand, as there was only one measurement allowed using the portable Trase system.

Figure 4.5 Waveguide.

The ceramic cups of the tensiometer and the waveguide rods of the Trase system were installed during soil compaction. As the medium sand removed from the dug pits were not placed in a proper container, it was difficult to maintain its water content. The medium sand of Changi was then left to air-dry for few hours before being compacted at its original dry density. Based on the measurement of air-dry samples collected and brought to the laboratory, the water content of the air-dried medium sand of Changi was found to be less than 0.5%.

The soils were placed back into the pits in 100 mm thick lifts and the mass of the soils for each lift was measured prior to compaction. The soils were compacted with

a 2.5 kg Proctor hammer for 20 to 25 drops for each layer of 100 mm thick soil (Appendix A). The field soil compaction followed the method of soil compaction in the laboratory soil column (see Section 4.2.2.2).

After compaction, the soils and the installed ceramic cups of the tensiometers and waveguide rods of the Trase system were left for a few days to let the pore-water pressure in the compacted soils to be in equilibrium with the surrounding sands and the profiles of the pore-water pressure to develop. Once the pore-water pressure profiles developed, the field infiltration tests were started.

Note that the pore-water pressure profiles developed in the field infiltration tests might not be a hydrostatic profile since it would take a long time for the pore-water pressure in the compacted soils to be in a hydrostatic equilibrium.

The field infiltration tests were conducted using a constant head method by applying a maximum 2 cm water pressure head in the double ring infiltration. The water pressure head was maintained throughout the tests by adjusting the flow regulator installed on the water pipe. Water used for the field infiltration tests came from the surrounding groundwater that was pumped to the surface by a water pump of Onga brand with a maximum capacity of 150 liter per minute (Figure 4.6). The groundwater was flown through a water pipe of 3 cm in diameter. The flow rates of the water infiltration were measured using a flow meter that was installed at the other end of the water pipe. Temperatures of the water and ground surface were measured during the field infiltration tests using a long-type thermometer of Traceable® brand with a temperature measuring range from -50 to 150° C.

The field infiltration tests were started from the maximum negative pore-water pressure developed throughout the soils and stopped once the pore-water pressure of the bottom most layers reached zero. The pore-water pressure and volumetric water content readings, as well as the flow rates of the water infiltration, were taken manually in each 30 seconds interval time.

Figure 4.6 Flow meter used in the field investigation.

4.2. Laboratory Investigation Program

Test methods and equipments used during laboratory works are described in this section. The laboratory investigation program involves investigation of soil properties (Section 4.2.1) and investigation of infiltration characteristics of capillary barriers (Section 4.2.2).

4.2.1. Investigation of soil properties

Investigation of soil properties conducted in this study includes investigation of basic soil properties (Section 4.2.1.1), investigation of soil-water characteristic curves (Section 4.2.1.2), determination of permeability function of the soils used (Section 4.2.1.3), and investigation of shrinkage characteristics (Section 4.2.1.4).

The main objective of this study is to improve the performance of the residual soil of Bukit Timah Granite by mixing the residual soil (RS) with gravelly sand (GS), medium sand (MS), or lime (L) at percentages of 25%, 50%, and 75% by dry mass. The name given for the soil mixtures consists of a designation and a number. The number immediately following the designation of the residual soil-gravelly sand mixtures (RG), residual soil-medium sand mixtures (RM), and residual soil-lime mixtures (RL) indicates the dry mass percentage of the residual soil in the soil mixtures. For example, RG-50 stands for a soil mixture of 50% residual soil and

50% gravelly sand; RM-75 indicates a soil mixture of 75% residual soil and 25% medium sand; RL-95 indicates a soil mixture of 95% residual soil and 5% lime by dry mass.

Prior to tests, each soil was compacted at a specified dry density, which was obtained from the compaction test in the infiltration column (see Section 4.2.2.2). Therefore, the same compaction method was applied to all soils used in this study.

4.2.1.1. Basic soil properties

Investigation of basic soil properties involves grain-size analyses, specific gravity tests, compaction tests, and saturated permeability tests. The grain-size analyses and soil classifications were performed in accordance with ASTM D 422 and ASTM D 2487, while specific gravity and compaction tests were conducted following ASTM D 854 and ASTM D 698, respectively. The saturated permeability of the soils was measured by the constant head method (ASTM D 2434), the falling head method (ASTM D 5856), and the constant head in a Flexible wall permeameter (ASTM D 5084).

4.2.1.2. Soil-water characteristic curve

The drying soil-water characteristic curve (SWCC) of the soils was obtained through a test using Tempe pressure cell and Pressure plate apparatus, while the wetting SWCC was measured by the capillary rise tube method. The investigation of the drying and the wetting SWCC is described in Fredlund and Rahardjo (1993a).

4.2.1.2.1. Determination of drying SWCC using Tempe pressure cell

The drying SWCCs of the soils used in this study were determined using a Tempe pressure cell manufactured by Soilmoisture Equipment Corporation, CA, USA. The Tempe cell is divided into three main components, which are the top and base caps, the 60 mm high and 85 mm diameter brass cylinder, and the 1 bar high-flow high-air entry ceramic disk (Figure 4.7). The top and base plates are sealed with O-rings and fastened by nuts. The saturated high air-entry ceramic disk allows water to flow freely across it, but it stops air to flow as long as the disk is fully saturated. According to Fredlund and Rahardjo (1993a), the saturated permeability of a 1Bar

high-flow high air-entry ceramic disk is 8.60×10^{-8} m/s, which can sufficiently minimize the impedance of water flow during the test.

A required amount of oven-dried soil specimen was placed on the saturated ceramic disk and compacted to a target density (Figure 4.7). Then, the outlet tube, which was provided in the water compartment beneath the ceramic disk, was connected to a burette filled with water for saturation of the soil specimen. After saturation, the top and the base caps were tightened together. In order to maintain the saturation of the ceramic disk during the test, which allowed soil water to flow out easily through the ceramic disk, the Tempe cell was then placed in a support ring that was placed in a pan filled with water.

Figure 4.7 Cross sectional view of Tempe cell with soil sample (Soilmoisture Equipment Corp., 1995).

Air pressure was supplied through the inlet tube on the top plate. Once the air pressure was applied, water in the specimen started draining out through the ceramic disk until equilibrium was reached. The change of water volume in the soil specimen was measured by weighing the Tempe cell periodically. In the initial stage of matric suction application, generally, the water flowed out rapidly and the volume of water in the soil specimen decreased drastically. As time elapsed, water continued flowing out at a smaller rate. Once the decrease in the volume of water

became insignificant, the measurement of volume of water under the applied matric suction value was stopped and the air pressure was subsequently increased to a higher value. In order to confirm the equilibrium condition in each applied matric suction, the volume of water in the soil specimen was plotted against logarithm of time during the test.

The procedure was repeated at higher applied air pressures (i.e. higher matric suction). Once the highest air pressure had been applied, the soil specimen was then moved to a Pressure plate apparatus for measurements at higher matric suction values.

Since the air-entry values of the residual soil-gravelly sand mixtures and the residual soil-medium sand mixtures were very low (i.e., less than 5 kPa), it was necessary to start the test at a very small matric suction value. Since the minimum air pressure supplied through the pressure gauge was only accurate to 10 kPa, the air pressure system was then modified and water head was used to control the air pressure applied to the soil (Figure 4.8). The valves were provided to release some of the air pressure, so that as small as 1 kPa air pressure (i.e., equals to 1 cm water pressure head) could be controlled.

Figure 4.8 Setting up of the Tempe cell test. (a) Schematic diagram; (b) photograph.

As mentioned above, water level was maintained at the bottom of the soil specimen in the Tempe cell during the test in order to maintain the saturation of the ceramic disk. When equilibrium is reached, the maintained water level allows a hydrostatic negative pore-water pressure profile to develop in the soil above the water level. When an air pressure is applied from the top of the soil, then, it is required to obtain the actual matric suction value of the soil, which is the average value of matric suction of the soil in the Tempe cell rather than matric suction at the soil surface. Therefore, the actual matric suction of the soil in the Tempe cell is calculated as follows:

Matric suction:

$$\text{- at the top of the soil} \quad = u_a - u_w = u_a - (-\rho_w gh) = u_a + \rho_w gh \quad (4.1)$$

$$\text{- at the bottom of the soil} \quad = u_a - u_w = u_a - 0 = u_a \quad (4.2)$$

Therefore,

$$\text{The actual (or the average) matric suction} \quad = ((u_a + \rho_w gh) + u_a) / 2 \quad (4.3)$$

$$= u_a + 0.5\rho_w gh \quad (4.4)$$

where :

ρ_w = water density (1 Mg/m³),

g = gravitational acceleration (9.81 m/s²),

h = the height of the soil specimen (m).

Equation 4.1 indicates that there is a correction value of $0.5\rho_w gh$ (kPa) to the matric suction due to the applied air pressure, u_a . It shows that even under zero air pressure ($u_a = 0$), the average matric suction in the Tempe cell is $0.5\rho_w gh$. The shorter the brass cylinder is, the smaller the correction will be. Since the cell used in this study was 60 mm high, the average matric suction developed under zero air pressure became 0.294 kPa. As the tests started from a low matric suction value, the correction value became significant. For high matric suctions the correction may become negligible.

4.2.1.2.2. Determination of drying SWCC using Pressure plate apparatus

As mentioned in the last section, in order to obtain the drying SWCC at higher matric suction values, the soil was then removed from the Tempe cell and then

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placed on a 5 bar high-flow high-air entry ceramic plate inside a pressure container. The soil was removed along with the brass cylinder in order to maintain the integrity of the soil during the tests in the Pressure plate apparatus.

The setting up of the pressure plate test is shown in Figure 4.9. The Pressure plate apparatus consists of a chamber containing a 5 Bar high-flow, high-air entry ceramic plate and a water compartment and a burette. The height of water in the burette was always maintained at the bottom of the soil specimen throughout the test. An applied air pressure to the soil specimen gives a matric suction value as matric suction is the difference between pore-air pressure (u_a) and pore-water pressure (u_w).

Figure 4.9 Setting up of Pressure plate test.

Before the test started, the 5 bar high-flow high-air entry ceramic plate was submerged in water in a vacuum container for saturation. Once the ceramic plate had become saturated, the soil and the brass cylinder, which were weighed in advance, were placed on the ceramic plate. Then, the container and the lid were tightened by the clamping bolts.

During the test, the high air pressure inside the container forced pore water in the soil to flow out through the pores in the ceramic plate. Similar to the determination

of SWCC using Tempe Cell, the change of water volume in the soil specimen was measured by weighing the soil together with the brass cylinder, periodically. In order to confirm the equilibrium condition in each applied matric suction, the volume of water in the soil specimen was also plotted against log time during the test.

Once the highest matric suction had been applied, the soil specimen was then removed from the Pressure Plate and the final water content of the soil specimen was measured by oven drying. This water content, together with the previous changes in mass, were used to back-calculate the water contents corresponding to different matric suctions. The water contents were then plotted against their corresponding matric suctions to give the soil-water characteristic curve.

4.2.1.2.3. Determination of wetting SWCC using Capillary rise tube method

In addition to drying SWCC, wetting SWCC was also measured in this study. The measurement of wetting SWCC was performed only for MRG-50, MRM-75, and MRL-95 that were the fine-grained soils of the capillary barrier columns.

In the capillary rise tube method, an oven-dried soil specimen was compacted in a polyvinyl chloride (PVC) cylinder of 100 mm internal diameter and 500 mm high (Figure 4.10). The cylinder was equipped with tensiometer-transducer system for pore-water pressure measurement. Several holes of 50 mm spacing interval were provided for water content measurement at the end of the test. A soil specimen was compacted at a certain density that was the same as the soil density used in other tests. Once the soil specimen was compacted, the top of the cylinder was covered to prevent evaporation and the cylinder was placed in a tray filled with water. As the wetting test was started, the tensiometers measured the change in pore-water pressures until equilibrium of pore-water pressures were established throughout the soil. Based on the measurement, water reached the upper part of the soil cylinder in less than 2 days (Appendix B21 and B-22). Even though the equilibrium of pore-water pressure based on tensiometer reading was established in such short a period of time, the test was still continued until approximately 20 days, after which the test was stopped and soil samples were collected for water content measurement using

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oven-drying method. The wetting period was extended in order to obtain a uniform water content distribution across each cross section. Based on trial tests, water content of the soil specimen at the center of the cylinder when measured in less than 2 days was slightly different from that of the soil specimen at the edge of the cylinder. This might be caused by the non-uniformity of the upward water movement in the non-homogeneous soil specimen.

Figure 4.10 Setting up of Capillary rise test.

4.2.1.2.4. *Fitting curves for the laboratory SWCC data*

For the purpose of the numerical modeling performed in this study, a complete relationship between volumetric water content and matric suction data were required. Unfortunately, Tempe cell and Pressure plate tests could only provide a limited set of data. In order to obtain a continuous and wide range of data, the experimental data were fitted with Fredlund & Xing (1994) equation using the correction factor, $C(\psi)$, equal to 1 as recommended by Leong and Rahardjo (1997a).

4.2.1.3. Determination of permeability functions

In addition to the SWCC, the continuous permeability function of a soil was also required in the numerical analyses. The coefficient of permeability at saturation, k_s , obtained from the laboratory upward infiltration test and the fitting curve SWCC were used in the prediction of coefficient of permeability with respect to water phase, k_w , using a computer program, SoilVision (1997).

In order to obtain the permeability function of each soil, a prediction of permeability function was initially performed based on the Brooks and Corey (1964) method (see Section 3.3.3.1). Meanwhile, a permeability function to best fit the predicted permeability data was also developed based on the Leong and Rahardjo (1997b) method as explained in Section 3.3.3.3. This permeability function was then fitted with the predicted permeability function obtained from Brooks and Corey (1964) method by changing the parameter p . The best-fitted permeability function was then used in the numerical analyses.

4.2.1.4. Shrinkage characteristics

As mentioned, the main objective of this study is to improve the performance of the residual soil that will be used as the fine-grained layer of a capillary barrier system by mixing the residual soil with gravelly sand, medium sand, or lime by certain dry mass percentages. The purpose of the mixing is not only to modify the hydraulic properties of the residual soil, but also to reduce the volume change potential of the soil, particularly during drying.

The shrinkage characteristics of the soils were measured through shrinkage tests conducted in the laboratory. A required amount of oven-dried soil was compacted at a target dry density in a polyvinyl chloride (PVC) cylinder of 100 mm internal diameter and 100 mm height. Prior to compaction, the cylinder was placed on a stainless plate with holes, over which a filter paper was placed in order to prevent downward soil migration. The holes were provided for soil saturation and downward drainage during the drying process. The soil was then placed in a

container filled with water for saturation. During saturation, the height of water was increased gradually until it was at the same level as the soil surface as shown in Figure 4.11(a). By applying this method, it was expected that there was no significant disturbance on soil properties caused by the water pressure. Therefore, the soil properties, mainly the void ratio e , after saturation would remain the same as the soil properties after compaction.

Figure 4.11 Setting up of the shrinkage test (a) Saturation process; (b) Measurement of soil volume using a caliper.

After the saturation or nearly saturation was achieved, the cylinder was removed and the soil was left to air-dry. The mass and the volume of the soil specimen were measured periodically during the test. The height and diameter of the soil specimen were measured using a caliper with 0.01 mm accuracy as shown in Figure 4.11(b). Once the last measurement of the soil mass and volume was performed, the soil was oven-dried to obtain the final water content and the water contents corresponding to soil volume changes were then back-calculated. The changes in water content and void ratio of the soil during the test were then plotted arithmetically as described in Fredlund and Rahardjo (1993a).

4.2.2. Investigation of infiltration characteristics of capillary barriers

One dimensional infiltration tests were conducted in the laboratory in order to study the infiltration characteristics of capillary barriers. In this study, three series of

laboratory infiltration tests, which involved an upward infiltration test, a rapid drawdown test, and various rainfall tests, were performed using an infiltration column and sets of pore-water pressure and volumetric water content measuring devices.

This section describes test methods, materials, and equipments used in the laboratory infiltration tests. The infiltration column apparatus is described in Section 4.2.2.1 and soils used in the laboratory infiltration tests and types of the laboratory infiltration tests are described in Section 4.2.2.2 and Section 4.2.2.3, respectively.

4.2.2.1. Infiltration column apparatus

The infiltration column apparatus consists of five main parts, i.e. the acrylic column, the tensiometer-transducer system, the data acquisition system, Trase system, and the water-flow system. A schematic diagram of the apparatus is shown in Figure 4.12 and the photograph is shown in Figure 4.13.

4.2.2.1.1. *The acrylic column*

The one dimensional infiltration tests in the laboratory were conducted through a 1 m high infiltration column. The infiltration column was made from an acrylic glass with a 5 mm thick wall and 190 mm inner diameter. Threaded holes were provided on the column wall for the installation of ceramic tips and wave guides. The top of the column was covered by a top cover, which was also made of an acrylic glass for easy monitoring, while the bottom of the column was fixed on the stainless steel base plate. The base plate, which was 600 mm above the floor level, was supported with four steel rods that were fixed on the concrete floor. The base plate was made partially hollow to receive a porous stone of high permeability. The joints between the column and the top cover and the joints between the column and the base plate were sealed with rubber O-rings in grooves and fastened with bolts and nuts to prevent water leakage during the laboratory infiltration test.

The top of the soil was always covered by the acrylic top cover in order to prevent evaporation, except during the application of rainfall.

4.2.2.1.2. *Tensiometer-transducer system*

Small tip tensiometers (Model 2100F Soilmoisture Probe), as used in the field (see Section 4.1.2), were also used to study the development of pore-water pressure during the laboratory infiltration tests. The small tip tensiometer consisted of a porous ceramic tip, a plastic body tube, and a pressure measuring device.

Figure 4.12 Schematic diagram of the infiltration column apparatus.

Figure 4.13 Infiltration column apparatus.

The ceramic tip was installed during the soil compaction in order to ensure that the ceramic cup was in good contact with the soil. The good contact is required to establish continuity between pore-water in the soil and the water inside the tube. A specially designed stainless steel connector, which was equipped with O-ring and thread tape to form good seal, was used to attach the ceramic cup and tube assembly on the wall of the acrylic column through the hole.

The pressure measuring device used in the laboratory infiltration tests was a piezoresistive pressure transducer of Kistler brand, RAN25A2B C1K type (Kristal

Instrumente AG, Winterthur, Switzerland). The transducer has a pressure measuring range from -175 to 175 kPa and requires a power supply of 12V to 30V DC. The transducer was connected through a cable to a data acquisition system. Before the laboratory infiltration tests started, the transducer and the data acquisition system were calibrated by applying water pressure head ranging from -0.60 to 0.60 m (-5.89 to 5.89 kPa). The calibrations showed a linear relationship between the applied pressure heads and the voltage readings (Appendices C-1 and C-2). In addition to the pore-water pressure calibration, the pore-water pressure head recorded at any position in the soil column was also corrected for the elevation head corresponding to the water column in the tensiometer. The equations obtained from the calibration were then used to convert the voltage readings into pore-water pressure head readings.

During trial tests, significant fluctuations of pore-water pressure head readings were observed. These might be caused by environmental changes, mainly temperature changes, that affected the performance of the transducers and the data loggers, as the infiltration tests were conducted in a room with slight variations in temperature. In order to eliminate the fluctuations so that correct pore-water pressure head readings could be obtained, a reference transducer for one data logger was exposed to air and placed close to other tensiometers. The pore-water pressure head readings from the installed tensiometers were then corrected with the reading from the reference transducer.

4.2.2.1.3. Data acquisition system

Two units of data logger, which were connected to a personal computer, were used in the laboratory infiltration tests (Figure 4.14). The measured real time data of pore-water pressure from the tensiometer-transducer system were transferred into a data logger and further processed and stored in a personal computer through communication software (DASYLab[®]) under Microsoft[®] Windows[®] environment.

The data logger was of DataShuttle[™] brand, DS-16-8-GP type (IOtech, Inc., USA) that had an integrated DC power supply inside. Each unit consisted of eight

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differential analogue inputs and allowed eight tensiometer-transducer assemblages to be connected.

The data stored in the personal computer were set in a format of pressure head versus real time for a reading interval of 1-minute, for accurate measurements.

Figure 4.14 Data logger.

4.2.2.1.4. Trase system

The volumetric water content of the soils during the laboratory infiltration tests was also measured using a Trase system.

Basically, the system includes mini buriable waveguides (Soilmoisture 6111 Model number, Figure 4.15(a)) and a unit of step pulse generator, Trase (Soilmoisture BE 6050X2 Model number, Figure 4.15(b)). The Trase is designed for use in a stationary location where electrical power is available, either from an AC source or from an external battery. The waveguide rods are 8 cm long, with 2.5 cm spacing between the outer rods. The Trase system is also equipped with accessories including a 76-channel Multiplexer Enclosure 6020B05 (Figure 4.15(c)), a 16-channel TDR Switch Board 6021C16, a 4MB memory card 6058C10 and Wintrase Kit 6300K2-BE.

Figure 4.15 (a) Mini buriable waveguide; (b) Trase 6050X2; (c) Multiplexer.

A specially designed brass connector, which was equipped with O-ring and thread tape to form good seal, was used to attach the waveguide and the coaxial cable on the wall of the acrylic column through the hole.

The waveguide was placed in the soil during compaction and attached to the multiplexer to allow measurements to be made automatically at 7 different locations using the Trase unit. The Trase system was programmed to make and store data automatically and was connected to a portable computer through an RS-232 port to enable remote operation of the system or downloading of the stored data.

4.2.2.1.5. *Water flow system*

The water flow system of the infiltration column consisted of inflow control, overflow discharge, percolation discharge, and electronic weighing balance.

The inflow control included a water storage tank, a constant head water tank, a flow regulator, and rainfall distributor. The water storage tank that was supported by an angled-steel frame had a storage capacity of 70L. Beneath the water storage tank was the constant head water tank, which had a storage capacity of 3.1L and a constant head of 110mm.

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Water in the constant head water tank, which was supplied from the water storage tank, flowed constantly into a rainfall distributor through a flow regulator. The rainfall distributor was made of a plastic nozzle with a manufactured small hole. The nozzles were arranged into two or four combinations so that a desired water flow rate was obtained. Furthermore, various flow rates could also be obtained by adjusting the flow regulator and the rainfall distributor arrangement. The flow rate was calibrated before and after rainfall tests, as the trials showed that the flow rates of water from the rainfall distributor were found to be decreasing with time. This problem might be caused by the clogging of the rainfall distributor by fine soil particles. As the flow rate decreased with time, the applied rainfall intensity would be the average of the rainfall intensities few hours before and after a rainfall test (Appendix C-3).

Two layers of filter paper were placed in good contact with the soil surface to spread out the water flowing from the rainfall distributor. By this arrangement, it was expected that water could be distributed uniformly to the soil surface and a uniform rainfall distribution could be simulated. The uniformity of the rainfall distribution was verified by placing two ceramic tips of tensiometers at the same elevation in the soil column. One ceramic tip was placed at the center of the soil column and the other one was installed at the edge of the soil column. Results of the laboratory infiltration tests showed that the pore-water pressure changed almost simultaneously in the two tensiometers, indicating that the rainfalls were distributed relatively uniformly (Appendix C-4).

Two outlets, which were located at the same level as the soil surface, were provided on the upper part of the soil column to discharge the upward flow of water during the steady upward infiltration test. In addition, the outlets could also be used to discharge overflow of water during the rainfall tests when the rainfall intensity higher than the saturated permeability of the upper soil layer was applied to the soil column.

Two outlets of the same elevation were also provided on the base plate. The first outlet, which was located on one side of the base plate, was provided for the water drainage. The second outlet, which was located on another side of the base plate, was connected to a large water tank of 0.4 x 0.4 square meter area and 0.6m high that was filled with water. By adjusting the level of water within the water tank, a desired water table could be produced while the percolating water in the soil column could drain out freely through the other outlet.

The quantity of the drainage during the laboratory infiltration tests was measured to calculate the mass-balance of water. An electronic weighing balance of model PS-12K manufactured by UWE was used in this study to allow a continuous data measurement as shown in Figure 4.16(a).

Figure 4.16 Components of water flow measurements. (a) Electronic weighing balance; (b) Digital indicator.

The instrument had a load cell with a capacity of 12 kg and a resolution of 1g. The acquisition system of the weighing balance was connected to a digital indicator of model KL-D1000 manufactured by Kubota as shown in Figure 4.16(b). In order to make readings automatically and to download the stored data, a personal computer was connected to the digital indicator through an RS-232 interface.

4.2.2.2. Soil compaction in the infiltration column

The capillary barrier models tested in this study were named as RGgs-50, RMgs-75, and RLgs-95. The RGgs-50 column consisted of 50 cm thick RG-50 (i.e., a mixture of 50% residual soil and 50% gravelly sand based on dry mass), which served as the

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fine-grained layer, and 50 cm thick gravelly sand, which served as the coarse-grained layer. The RMgs-75 column was 50 cm thick RM-75 (i.e., a soil mixture of 75% residual soil and 25% medium sand based on dry mass) overlying 50 cm thick gravelly sand. Lastly, the RLgs-95 was a soil column of 50 cm thick RL-95 (i.e., a soil mixture of 95% residual soil and 5% lime based on dry mass) overlying 50 cm thick gravelly sand.

The RG-50, RM-75, and RL-95 were selected as the fine-grained layers of capillary barrier models because they were considered to have the best hydraulic properties and shrinkage characteristics among other soil mixtures (see Section 6.3). The selection was based on the criteria that a fine-grained layer of capillary barriers must have reasonably high saturated permeability and significantly low volumetric shrinkage. For the residual soil-gravelly sand mixtures, the saturated permeability of the RG-50 was higher than of the RG-75 and the volumetric shrinkage of the RG-50 was lower than that of the RG-75. Meanwhile, the RM-75 was selected to be used as the fine-grained layer of the capillary barrier model because the volumetric shrinkage of the RM-75 was comparable to that of the RM-50. In addition, the saturated permeability of the RM-75 was considered to be high enough, although the saturated permeability of the RM-75 was much lower than that of the RM-50. In case of the residual soil-lime mixtures, the saturated permeability of the RL-95 was the highest, whilst the volumetric shrinkage of the RL-95 was comparable to that of other residual soil-lime mixtures.

An oven-dried soil was filled into the infiltration column in maximum 100 mm thick lifts and the mass of soil for each lift was measured. The soil was, then, compacted with a 2.5 kg Proctor hammer by applying 25 drops for one layer of 100 mm thick soil. This compaction method was applied considering the strength of the infiltration column, which was made from acrylic glass, and the problem associated with the difficulty in preparing large amount of soil with a uniformly low water content.

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During soil compaction, a thin high permeable filter cloth was placed at the interface of the fine-grained layer overlying the coarse-grained layer in order to prevent migration of the fine-grained soil into the coarse-grained layer. The ceramic tips of the tensiometers and the waveguides of the TDR system were placed at the desired elevations and the coaxial tubes of the ceramic tips and the cables of the waveguides were extended out of the infiltration column through their connectors.

4.2.2.3. Types of laboratory infiltration tests

The infiltration tests conducted in the laboratory involved upward infiltration test, drawdown test, and rainfall test. Details of the infiltration tests conducted in this study are shown in Table 4.1.

In Table 4.1, FI-MSoC and FI-MSoCgs refer to field infiltration tests conducted on medium sand of Changi (MSoC) and medium sand of Changi (MSoC) overlying gravelly sand, respectively. Details of the field infiltration tests are described in Section 4.1. In the laboratory infiltration tests, IC-RGgs-50-Ui, IC-RMgs-75-Ui, and IC-RLgs-95-Ui indicate upward infiltration tests conducted on RGgs-50 column, RMgs-75 column, and RLgs-95 column, respectively. IC-RGgs-50-Dr, IC-RMgs-75-Dr, and IC-RLgs-95-Dr indicate rapid drawdown tests conducted on RGgs-50 column, RMgs-75 column, and RLgs-95 column, respectively. Subsequently, IC-RGgs-50-R1 to IC-RGgs-50-R4, IC-RMgs-75-R1 to IC-RMgs-75-R5, and IC-RLgs-95-R1 to IC-RLgs-95-R6 indicate rainfall tests conducted on RGgs-50 column, RMgs-75 column, and RLgs-95 column, respectively.

4.2.2.3.1. Upward water infiltration test

The upward water infiltration test was conducted to assess the saturated permeability of the soils in the infiltration column that would be used in the numerical analyses. Results of the upward water infiltration tests could be compared with those obtained from the falling head, the constant head, and the constant rate water flow permeability tests.

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Table 4.1 Details of the infiltration tests conducted in this study.

No	Test	Description	Rainfall Duration (h)	Rainfall Intensity (q)		Boundary Conditions	
				A=0.0283390 m ²		Top	Bottom
				mm/h	m/s		
1	FI-MSoC	Field infiltration test	-	-	-	$h_p=0.02$	-
2	FI-MSoCgs	Field infiltration test	-	-	-	$h_p=0.02$	-
3	IC-RGgs-50-Ui	Upward infiltration test	-	-	-	-	$h_p=1.52$
4	IC-RGgs-50-Dr	Rapid drawdown test	-	-	-	-	$h_p=1$ to 0
5	IC-RGgs-50-R1	Rainfall test	6	3.0	8.3E-07	$q < k_s$	$h_p = 0$
6	IC-RGgs-50-R2	Rainfall test	12	1.7	4.7E-07	$q < k_s$	$h_p = 0$
7	IC-RGgs-50-R3	Rainfall test	6	6.1	1.7E-06	$q < k_s$	$h_p = 0$
8	IC-RGgs-50-R4	Rainfall test	2	9.0	2.5E-06	$q < k_s$	$h_p = 0$
9	IC-RMgs-75-Ui	Upward infiltration test	-	-	-	-	$h_p=1.52$
10	IC-RMgs-75-Dr	Rapid drawdown test	-	-	-	-	$h_p=1$ to 0
11	IC-RMgs-75-R1	Rainfall test	6	3.0	8.5E-07	$q < k_s$	$h_p = 0$
12	IC-RMgs-75-R2	Rainfall test	6	1.3	3.6E-07	$q < k_s$	$h_p = 0$
13	IC-RMgs-75-R3	Rainfall test	12	0.9	2.2E-07	$q < k_s$	$h_p = 0$
14	IC-RMgs-75-R4	Rainfall test	6	5.5	1.5E-06	$q < k_s$	$h_p = 0$
15	IC-RMgs-75-R5	Rainfall test	2	9.9	2.8E-06	$q < k_s$	$h_p = 0$
16	IC-RLgs-95-Ui	Upward infiltration test	-	-	-	-	$h_p=1.52$
17	IC-RLgs-95-Dr	Rapid drawdown test	-	-	-	-	$h_p=1$ to 0
18	IC-RLgs-95-R1	Rainfall test	3	1.7	4.6E-07	$q < k_s$	$h_p = 0$
19	IC-RLgs-95-R2	Rainfall test	2	9.7	2.7E-06	$q < k_s$	$h_p = 0$
20	IC-RLgs-95-R3	Rainfall test	12	4.0	1.1E-06	$q < k_s$	$h_p = 0$
21	IC-RLgs-95-R4	Rainfall test	6	1.7	4.7E-07	$q < k_s$	$h_p = 0$
22	IC-RLgs-95-R5	Rainfall test	12	0.3	7.9E-08	$q < k_s$	$h_p = 0$
23	IC-RLgs-95-R6	Rainfall test	2	1.1	3.0E-07	$q < k_s$	$h_p = 0$

Remarks: h_p : water pressure head (m); k_s : saturated permeability obtained from the upward infiltration test.

A constant water pressure head was applied to the bottom of the soil column ($z = 0$) and the upward water infiltration occurred under a saturated condition. The test was conducted in a few days until a steady-state upward water infiltration was achieved. Water was allowed to overflow freely from the top of the soil column ($z = 1\text{m}$) and the flow rate of water was measured once a steady-state water infiltration was achieved. During the test, the changes in pore-water pressure head were measured by the tensiometers installed at different locations in the soil column. The homogeneity of the soil layers could be examined from the total head (i.e., the pressure head plus the elevation head) profile developed during the steady-state upward infiltration test considering that the flow of water was determined by the permeability of the soil and the hydraulic head gradient. Based on the measured water flow rate and the total head, the coefficient of permeability at saturation, k_s , of the soil layers could be calculated following Darcy's Law. At the end of the upward infiltration test, the valve located at the bottom of the soil column was closed.

4.2.2.3.2. Drawdown test

Rapid drawdown tests were conducted in order to study the development of negative pore-water pressure, as well as volumetric water content of capillary barrier models in a drying process starting from a saturated or nearly saturated condition. In addition, the equilibrium conditions of pore-water pressure head and volumetric water content developed at the end of a drawdown test were used as the initial conditions of those for the subsequent rainfall test.

As mentioned above, the valve located at the bottom of the soil column was closed at the end of the upward water infiltration test. Soils in the infiltration column were left in a saturated condition with water table at the top of the soil column ($z = 1\text{m}$) and, therefore, an equilibrium profile of pore-water pressure (i.e., total head h equals to 1m) was developed throughout the soil column. A rapid drawdown test of water table was then carried out by opening the valve at the bottom of the column ($z = 0\text{m}$) that allowed water to drain out freely from the outlet fixed at the bottom of the infiltration column. Once the positive pore-water pressure had completely dissipated, water table was maintained constant at the bottom of infiltration column ($z = 0\text{m}$) until the end of this test.

4.2.2.3.3. Rainfall test

The infiltration characteristics of the capillary barrier models developed in this study were studied in rainfall tests that represented the wetting and drying processes occurred in the fine-grained layer and coarse-grained layer. After the drawdown test, negative pore-water pressure would develop to a certain extent until a maximum negative pore-water pressure head was attained. Then, a rainfall test was applied to the top of the soil column under a certain intensity of rainfall for a specified duration. Once the rainfall was terminated, the top of the soil column was covered to prevent evaporation and water table was maintained constant at the bottom of the column ($z = 0$) throughout the measurement of pore-water pressure and volumetric water content.

Different rainfall tests were carried out by applying different intensities of rainfall with different durations. Different rainfall tests might be started from different initial conditions, since it was not always possible to attain hydrostatic conditions prior to all rainfall tests. Results of the rainfall tests showed that it would require an overly long time for the soils to reach hydrostatic conditions or perhaps hydrostatic conditions could never be reached in the controlled laboratory conditions.

4.3. Numerical Analysis

The numerical analyses performed in this study included simulations of results obtained from the laboratory infiltration tests (i.e., the drawdown and rainfall tests). The numerical analyses were performed using a comprehensive finite element-based computer software, SVFlux version 3.09 (SoilVision Systems Ltd., Canada, 2003). The computer software, which is under Windows operation system, essentially consists of three main components, which are user interface, solving interface, and plotting interface. The SVFlux user interface allows a problem to be entered, managed, and edited easily. SVFlux uses a powerful generic finite element solver called FlexPDE as a solving interface to solve the problems. The solver has the capability to automatically generate and refine the solution mesh, as well as to refine the time step increment specified by the user. Results of a problem are then exported to the widely used Tecplot[®] plotting software, which provides the standard plots required for an accurate description of the results from a seepage model.

As applied in the laboratory, an infiltration column of 1m high and 190 mm internal diameter was modeled in the numerical analyses. The infiltration column was divided into two regions of 0.5 m high each that represented the regions of the fine-grained layer and the coarse-grained layer of a capillary barrier column. Once the regions of a problem were defined, the boundary conditions of the regions were then specified. The soil properties (i.e., SWCCs and permeability functions) used in the numerical analyses were those obtained from the laboratory drying and wetting SWCC tests.

The laboratory infiltration tests were simulated in two stages. The first stage was a simulation of the drawdown test and it was subsequently continued by a simulation of rainfall tests using various rainfall intensities and durations.

A zero-flux boundary was applied to the walls of the soil column as the walls were impermeable. The boundary conditions of the top and bottom of the soil column in the drawdown tests were different than those in the rainfall test. In simulations of the drawdown tests, the boundary conditions of the top of the soil column were of zero-flux ($q = 0$), while the boundary conditions of the bottom of the soil column were functions of the total head versus time obtained from the laboratory drawdown tests. Based on the laboratory drawdown tests of water table, the positive pore-water pressures of the fine-grained layer and coarse-grained layer were dissipated in approximately 10 seconds. This condition was then applied to the simulation of the drawdown test where the total head at the bottom of the soil column ($z = 0\text{m}$) was set from $h = 1\text{m}$ at $t = 0\text{s}$ until $h = 0\text{m}$ at $t = 10\text{s}$.

In the simulations of rainfall tests, the initial condition of the first rainfall test was the result of the drawdown test. Subsequently, the result of the first rainfall test was used as the initial condition of the second rainfall test, and so forth. The boundary conditions at the bottom of soil column were constant head condition ($h = 0\text{m}$) throughout the rainfall test, while the boundary conditions of the top of soil column were consistent with the flux and duration of rainfalls applied in the respective laboratory rainfall tests. As the rainfall intensity of the laboratory rainfall tests was not constant and found to be decreasing with time (see Section 4.2.2.1.5), a linear interpolation method of data flux was applied in each rainfall test simulation.

Chapter 5

Presentation of Results

This chapter contains presentation of the results obtained from the works carried out during this study, which consist of field measurements (i.e., investigation of barrier effect), laboratory works (i.e., investigations of soil properties and infiltration characteristics of capillary barrier models), and numerical analyses (i.e., simulations of the laboratory infiltration study). The results obtained from the investigation of barrier effect in the field are presented in Section 5.1. Subsequently, the results of investigations of the soil properties and the results of investigation of infiltration characteristics are shown in Section 5.2. The results obtained from the numerical analyses are presented in Section 5.3, while the comparison of numerical simulation results and experimental data is presented in Section 5.4.

5.1. Barrier Effect in the Field

The results of one-dimensional infiltration tests in the field are shown in Figure 5.1 and Figure 5.2. The arrow above the profiles indicates relative movement of the pore-water pressure profiles during the test. During the test, a maximum 2 cm of water pressure head was maintained in the double-ring infiltrometer, both in the inner-ring and the outer-ring, so that the changes in pore-water pressure in the soil layers could be well logged.

Figure 5.1 shows the development of pore-water pressure head during an infiltration test conducted on 60cm thick of medium sand of Changi (MSoC), which represents a non-capillary barrier system, while Figure 5.2 shows a gradual movement of pore-water pressure profiles during an infiltration test conducted on 40cm thick MSoC overlying 20cm thick gravelly sand (GS), which represents a capillary barrier system.

As indicated by the movement of pore-water pressure head profile in Figure 5.1, the wetting front reached $z = 0.2\text{m}$ (or the depth of 40cm below the ground surface) in between $t = 6\text{min}$ and $t = 10\text{min}$. During the time interval, the pore-water pressure head increased slightly from -2.2m to -1.9m . Meanwhile, the wetting front reached the interface of the MSoC and the GS, which was also located at $z = 0.2\text{m}$, after $t = 10\text{min}$ (i.e., in between $t = 10\text{min}$ and $t = 15\text{min}$; see Figure 5.2). Again, the pore-water pressure head at the bottommost part of the non-capillary barrier system reached zero in $t = 25\text{min}$ (Figure 5.1) while the pore-water pressure head at the bottommost part of the coarse layer of the capillary barrier system was still negative (i.e., about -1.2 m) at $t = 25\text{min}$ (Figure 5.2).

The above comparisons illustrate that a capillary barrier existed at the interface of the medium sand of Changi overlying the gravelly sand. The contrast in unsaturated permeability between the finer layer (i.e., medium sand) and the coarser layer (i.e., gravelly sand), seems to impede the downward movement of water into the underlying layer.

Figure 5.1 Profiles of pore-water pressure during field infiltration test on the medium sand of Changi (MSoC).

Figure 5.2 Profiles of pore-water pressure during field infiltration test on the medium sand of Changi (MSoC) overlying the gravelly sand (GS).

5.2. Soil Properties and Infiltration Characteristics

In this section, results of laboratory experiments, which involve the results of investigation of soil properties (Section 5.2.1) and the results of investigation of infiltration characteristics of capillary barriers in the laboratory (Section 5.2.2), are presented.

5.2.1. Soil Properties

Soil properties investigated in this study involved basic soil properties, soil hydraulic properties (i.e., SWCC and permeability function), and soil shrinkage characteristics. Basic soil properties of the soils are shown in Section 5.2.1.1. Results of the measurement of SWCCs, both drying SWCCs and wetting SWCCs, are shown in Section 5.2.1.2, whilst the permeability functions predicted from the respective SWCCs are presented in Section 5.2.1.3. Lastly, results of the shrinkage tests are given in Section 5.2.1.4.

5.2.1.1. Basic soil properties

Grain-size distribution curves of the residual soil-gravelly sand mixtures, the residual soil-medium sand mixtures, and the residual soil-lime mixtures are presented in Figure 5.3, 5.4, and 5.5, respectively. Basic properties of the soil mixtures are listed in Table 5.1, 5.2, 5.3, and 5.4.

Figure 5.3 Grain-size distribution curves of the residual soil-gravelly sand mixtures.

The residual soil (RS) was reddish brown in color and a weathering product of Bukit Timah Granite. The soil was taken from an on-going construction project near the Singapore Island Golf Club. The gravelly sand (GS), which essentially a mixture of coarse sand (50.1%) and fine gravel (49.9%), and the medium sand (MS) were light grey to white in color and were obtained commercially. The medium sand of Changi (MSoC), which was taken from the Changi Reclamation Area, was greenish grey in color and contained some shells. The lime was in the form of powder and it was obtained commercially. The lime was essentially hydrated lime ($\text{Ca}(\text{OH})_2$) and contained chemical composition of minimum 92% Calcium

Hydroxide ($\text{Ca}(\text{OH})_2$), 2% Magnesium Oxide (MgO), and 3% Calcium Carbonate (CaCO_3).

Figure 5.4 Grain-size distribution curves of the residual soil-medium sand mixtures.

Figure 5.5 Grain-size distribution curves of the residual soil-lime mixtures.

The dry densities of the residual soil, gravelly sand, medium sand, and soil mixtures listed in Table 5.1, 5.2, 5.3, and 5.4 were obtained from compaction tests conducted in the infiltration column using 2.5 kg Standard Proctor hammer. These dry densities were then used as target densities in other tests in the laboratory experiments.

Table 5.1 Basic properties of the residual soil, gravelly sand, medium sand, and medium sand of Changi.

Properties	Soils			
	RS	GS	MS	MSoC
1. Unified Soil Classification System				
Group symbol	SC	SP	SP	SP
Group name	Clayey sand	Poorly graded sand with gravel	Poorly graded sand	Poorly graded sand
2. Specific gravity, G_s	2.590	2.66	2.65	2.67
3. Grain size distribution				
D_{60} (mm)	0.64	5.39	0.74	0.91
D_{30} (mm)	0.02	3.52	0.49	0.38
D_{10} (mm)	0.0005	2.68	0.35	0.21
Coefficient of uniformity, C_u	1280	2.01	2.11	4.33
Coefficient of curvature, C_c	1.25	0.86	0.92	0.76
Gravel content ($> 4.75\text{mm}$)	0.19%	47.51%	0.00%	3.02%
Fines content ($< 0.075\text{mm}$)	37.68%	0%	0.01%	0.46%
4. Relative density test results				
Maximum void ratio, e_{max}	1.14	0.85	0.91	-
Minimum void ratio, e_{min}	0.66	0.53	0.15	-
5. Compaction test results				
Max. dry density, $\rho_d \text{ max}$ (Mg/m^3)	1.79	-	-	-
Optimum water content, w_{opt}	14.5 %	-	-	-
6. Coefficient of permeability at saturation, k_s (m/s)				
Constant head method	-	7.60×10^{-2}	3.28×10^{-3}	8.46×10^{-4}
Falling head method	1.39×10^{-6}	-	-	-
Constant head method (FWP)	1×10^{-7}	-	-	-
7. Soil properties used in this study (based on soil column test)				
Dry density of soil, ρ_d (Mg/m^3)	1.47	1.62	1.557	1.73
Void ratio, e	0.76	0.64	0.699	0.55
Water content at saturation, w	29.39%	24.09%	26.34%	20.39%
Coefficient of permeability at saturation, k_s (m/s)				
Upper layer	$1.35 \times 10^{-6*}$	9.61×10^{-3}	-	-
Lower layer	$1.94 \times 10^{-7*}$	4.27×10^{-3}	-	-
k_v (equivalent)	-	6.10×10^{-3}	-	-

Remarks: *Yang (2002)

Table 5.2 Basic properties of the residual soil-gravelly sand mixtures.

Properties	Soils		
	RG-25	RG-50	RG-75
1. Unified Soil Classification System			
Group symbol	SW-SC	SC	SC
Group name	Well-graded sand with clay and gravel	Clayey sand with gravel	Clayey sand
2. Specific gravity, G_s	2.632	2.631	2.626
3. Grain size distribution			
D_{60} (mm)	4.76	3.15	1.25
D_{30} (mm)	2.84	0.66	0.10
D_{10} (mm)	0.30	0.01	0.001
Coefficient of uniformity, C_u	15.87	315	1250
Coefficient of curvature, C_c	5.65	13.83	8.00
Gravel content ($> 4.75\text{mm}$)	40%	22.90%	9%
Fines content ($< 0.075\text{mm}$)	7.6%	18.34%	28.96%
4. Relative density test results			
Maximum void ratio, e_{max}	0.74	0.78	0.94
Minimum void ratio, e_{min}	0.22	0.29	0.32
5. Coeff. of permeability at saturation, k_s (m/s)			
Constant head method	8.32×10^{-4}	-	-
Falling head method	-	6.26×10^{-6}	1.56×10^{-6}
Constant head method (FWP)	-	3.0×10^{-7}	1.0×10^{-7}
6. Soil column test results			
Dry density of soil, ρ_d (Mg/m ³)	1.79	1.74	1.66
Void ratio, e	0.47	0.51	0.58
Water content at saturation, w	17.76%	19.44%	22.14%
Coeff. of permeability at saturation, k_s (m/s)			
Upper layer	-	2.14×10^{-4}	-
Middle layer	-	1.85×10^{-4}	-
Lower layer	-	2.09×10^{-4}	-
k_v (equivalent)	-	2.10×10^{-4}	-

As listed in Table 5.1, 5.2, 5.3, and 5.4 the saturated permeabilities of the soils obtained from laboratory permeability tests vary several orders of magnitude. The results of the permeability tests obtained from the falling head method show a reasonable agreement with the results of the permeability tests obtained from the constant head method in the Flexible wall permeameter (FWP). Furthermore, for the soil mixtures selected as the fine-grained layer of capillary barrier models (i.e., RG-50, RM-75, and RL-95), the coefficient of permeability at saturation k_s obtained from the upward infiltration tests in the infiltration column is about 2 orders of magnitude higher than the coefficient of permeability at saturation k_s obtained from the other laboratory permeability tests (i.e., falling head method and constant head method in the Flexible wall permeameter).

Table 5.3 Basic properties of the residual soil-medium sand mixtures.

Properties	Soils		
	RM-25	RM-50	RM-75
1. Unified Soil Classification System			
Group symbol	SW-SC	SC	SC
Group name	Well-graded sand with clay	Clayey sand	Clayey sand
2. Specific gravity, G_s	2.608	2.601	2.598
3. Grain size distribution			
D_{60} (mm)	0.70	0.64	0.54
D_{30} (mm)	0.42	0.32	0.08
D_{10} (mm)	0.08	0.01	0.00
Coefficient of uniformity, C_u	8.80	118.95	535.36
Coefficient of curvature, C_c	3.11	30.37	11.76
Gravel content (> 4.75mm)	0.00	0.00	0.00
Fines content (< 0.075mm)	0.10	0.20	0.30
4. Relative density test results			
Maximum void ratio, e_{max}	0.98	1.00	1.05
Minimum void ratio, e_{min}	0.37	0.37	0.37
5. Coeff. of permeability at saturation, k_s (m/s)			
Constant head method	1.24×10^{-4}	-	-
Falling head method	-	4.65×10^{-6}	7.09×10^{-7}
Constant head method (FWP)	-	3.00×10^{-6}	4.00×10^{-7}
6. Soil column test results			
Dry density of soil, ρ_d (Mg/m ³)	1.526	1.520	1.516
Void ratio, e	0.709	0.711	0.714
Water content at saturation, w	27.16%	27.32%	27.44%
Coeff. of permeability at saturation, k_s (m/s)			
Upper layer	-	-	9.13×10^{-5}
Middle layer	-	-	8.14×10^{-5}
Lower layer	-	-	9.01×10^{-5}
k_v (equivalent)	-	-	8.4×10^{-5}

Table 5.4 Basic properties of the residual soil-lime mixtures.

Properties	Soils			
	RL-91	RL-93	RL-95	RL-97
1. Unified Soil Classification System				
Group symbol	SC	SC	SC	SC
Group name	Clayey sand	Clayey sand	Clayey sand	Clayey sand
2. Specific gravity, G_s	2.382	2.357	2.314	2.310
3. Grain size distribution				
D_{60} (mm)	0.52	0.55	0.57	0.61
D_{30} (mm)	0.02	0.02	0.02	0.02
D_{10} (mm)	0.0001	0.0002	0.0003	0.0004
Coefficient of uniformity, C_u	5159.41	2752.10	1885.87	1528.58
Coefficient of curvature, C_c	5.01	2.99	1.96	1.59
Gravel content (> 4.75mm)	0.17%	0.17%	0.18%	0.18%
Fines content (< 0.075mm)	43.29%	42.04%	40.79%	39.55%
4. Coeff. of permeability at saturation, k_s (m/s)				
Constant head method	-	-	-	-
Falling head method	8.1×10^{-8}	7.9×10^{-8}	1.3×10^{-7}	1.6×10^{-7}
Constant head method (FWP)	3.0×10^{-8}	5.0×10^{-8}	6.0×10^{-8}	8.0×10^{-8}
5. Soil column test results				
Dry density of soil, ρ_d (Mg/m ³)	1.47	1.47	1.47	1.47
Void ratio, e	0.57	0.59	0.60	0.62
Water content at saturation, w	24.72%	25.27%	25.57%	25.98%
Coeff. of permeability at saturation, k_s (m/s)				
Upper layer	-	-	1.20×10^{-5}	-
Middle layer	-	-	1.90×10^{-5}	-
Lower layer	-	-	1.18×10^{-5}	-
k_v (equivalent)	-	-	1.58×10^{-5}	-

The difference in the saturated permeability may be caused by the difference in the soil density, even though the applied compaction method was used in all the tests. Unlike the fine-grained layers of the capillary barrier models, the coefficient of permeability at saturation, k_s , of the gravelly sand obtained from the upward infiltration test is about 1 order of magnitude lower than that obtained from the constant-head permeability test. This finding can be explained by the fact that the gravelly sand in the infiltration column received additional compaction energy during compaction of the fine-grained layer causing the density of the gravelly sand in the infiltration column to increase. As a result, the saturated permeability of the gravelly sand in the capillary barrier arrangement was lower than that of a single layer of gravelly sand as in the constant head permeability test.

5.2.1.2. Soil-water characteristic curves

The drying soil-water characteristic curves (SWCCs) obtained from Tempe cell and Pressure plate tests of the residual soil-gravelly sand mixtures, the residual soil-medium sand mixtures, and the residual soil-lime mixtures are shown in Figures 5.6, 5.7, and 5.8, respectively, while the plots of the water volume in the soil versus logarithm of time for each matric suction application are shown in Appendix B-1 to Appendix B-20. In addition to the drying SWCC, the wetting SWCC of the RG-50, RM-75, and RL-95 were also measured using the Capillary rise tube method, as shown in Figures 5.9, 5.10, and 5.11.

In order to obtain a complete relationship between volumetric water content and matric suction data, the laboratory test results were fitted with Fredlund and Xing (1994) equation using the correction factor $C(\psi) = 1$ as recommended by Leong and Rahadjo (1997a). The fitting was performed automatically using the Microsoft Excel solver tool and, then, some key parameters of the SWCC (i.e., air entry value, ψ_a , residual matric suction, ψ_r , residual volumetric water content, θ_r , and water-entry value, ψ_w) were obtained manually from the developed fitting curve of SWCC. The drying SWCC parameters are presented in Tables 5.5, 5.6, 5.7, and 5.8, whilst the wetting SWCC parameters are listed in Table 5.9.

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As shown in Figures 5.6, 5.7, 5.8, 5.9, 5.10, and 5.11, the laboratory data are plotted using symbols, while the fitting curves are presented in the form of lines. It is clearly shown that the drying SWCCs of the residual soil-gravelly sand mixtures lie in between the drying SWCCs of the residual soil and the gravelly sand and the drying SWCCs of the residual soil-medium sand mixtures are located in between the drying SWCCs of the residual soil and the medium sand.

Figure 5.6 Drying SWCC of the residual soil-gravelly sand mixtures and the MSoC.

Figure 5.7 Drying SWCC of the residual soil-medium sand mixtures.

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Figure 5.8 Drying SWCC of the residual soil-lime mixtures.

Table 5.5 Parameters of the drying SWCC of the residual soil (RS), gravelly sand (GS), medium sand (MS), and medium sand of Changi (MSoC).

No	Description	Symbol	Soil			
			RS	GS	MS	MSoC
1	Volumetric water content at saturation	θ_s	0.432	0.382	0.411	0.352
2	Air-entry value (kPa)	ψ_a	4.687	0.058	0.369	0.188
3	Residual matric suction (kPa)	ψ_r	14.88	0.500	1.852	0.452
4	Residual volumetric water content	θ_r	0.232	0.026	0.015	0.021
5	Fitting parameters	a	0.321	0.100	0.661	0.321
		n	2.468	3.521	3.040	2.468
		m	0.937	1.322	1.491	0.937

Table 5.6 Parameters of the drying SWCC of the residual soil-gravelly sand mixtures.

No	Description	Symbol	Soil		
			RG-25	RG-50	RG-75
1	Volumetric water content at saturation	θ_s	0.318	0.339	0.368
2	Air-entry value (kPa)	ψ_a	0.113	0.204	0.770
3	Residual matric suction (kPa)	ψ_r	2.337	8.000	18.984
4	Residual volumetric water content	θ_r	0.094	0.149	0.221
5	Fitting parameters	a	0.196	0.478	1.509
		n	2.177	1.155	1.818
		m	0.511	0.467	0.303

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Table 5.7 Parameters of the drying SWCC of the residual soil-medium sand mixtures.

No	Description	Symbol	Soil		
			RM-25	RM-50	RM-75
1	Volumetric water content at saturation	θ_s	0.414	0.415	0.416
2	Air-entry value (kPa)	ψ_a	0.463	0.509	0.602
3	Residual matric suction (kPa)	ψ_r	1.874	2.323	7.000
4	Residual volumetric water content	θ_r	0.091	0.122	0.152
5	Fitting parameters	a	0.661	0.673	1.080
		n	3.040	5.319	2.567
		m	1.491	0.448	0.456

Table 5.8 Parameters of the drying SWCC of the residual soil-lime mixtures.

No	Description	Symbol	Soil			
			RL-91	RL-93	RL-95	RL-97
1	Volumetric water content at saturation	θ_s	0.363	0.371	0.376	0.382
2	Air-entry value (kPa)	ψ_a	6.072	5.571	5.039	3.783
3	Residual matric suction (kPa)	ψ_r	200.731	155.073	62.843	126.871
4	Residual volumetric water content	θ_r	0.100	0.167	0.170	0.199
5	Fitting parameters	a	22.006	17.032	9.164	9.045
		n	1.127	1.006	2.305	1.411
		m	1.182	0.840	0.449	0.458

Figure 5.9 Drying and wetting SWCC of the RG-50.

Figure 5.10 Drying and wetting SWCC of the RM-75.

Figure 5.11 Drying and wetting SWCC of the RL-95.

Table 5.9 Parameters of the wetting SWCC of the RG-50, RM-75, and RL-95.

No	Description	Symbol	Soil		
			RG-50	RM-75	RL-95
1	Volumetric water content at saturation	θ_s	0.339	0.416	0.376
2	Water-entry value (kPa)	ψ_w	3.061	4.900	28.605
3	Fitting parameters	a	0.179	0.338	2.609
		n	1.626	1.273	1.675
		m	0.469	0.702	0.553

5.2.1.3. Permeability functions

In addition to the fitting SWCC, a continuous and wide range of permeability function was also required in the numerical analyses. The coefficient of permeability with respect to water phase, k_w , of the soils was determined from the measured coefficient of permeability at saturation, k_s , and the SWCC data. As previously mentioned, the SWCC data (both drying SWCC and wetting SWCC) were fitted with Fredlund and Xing (1994) equation using the correction factor $C(\psi) = 1$ as recommended by Leong and Rahardjo (1997a). The fitting curve of the SWCC and the measured coefficient of permeability at saturation, k_s , were then used to predict the permeability function of the respective soil based on the method proposed by Brooks and Corey (1964) using the commercially available software, SoilVision (1997). Another permeability function was also determined based on Leong and Rahardjo (1997b) method and, then, best fitted with the permeability function predicted using Brooks and Corey (1964) method. The drying permeability functions of the soils predicted from the coefficient of permeability at saturation, k_s , (i.e., constant head or constant head permeability tests in the Flexible wall permeameter) and the drying SWCC are shown in Figures 5.12, 5.13, and 5.14, while the fitting parameter of the drying permeability functions is presented in Table 5.10.

Figure 5.12 Drying permeability functions of the residual soil-gravelly sand mixtures.

Figure 5.13 Drying permeability functions of the residual soil-medium sand mixtures.

Figure 5.14 Drying permeability functions of the residual soil-lime mixtures.

Table 5.10 Fitting parameter of the drying permeability functions of the soils.

No	Soils	Parameter p
1	Gravelly sand (GS)	2.4
2	RG-25	3.2
3	RG-50	4.0
4	RG-75	4.0
5	Residual soil (RS)	4.0
6	RM-25	2.6
7	RM-50	2.6
8	RM-75	3.1
9	Medium sand (MS)	2.1
10	Medium sand of Changi (MSoC)	2.5
11	RL-91	3.1
12	RL-93	3.7
13	RL-95	3.4
14	RL-97	3.9

As mentioned previously (see Section 4.3.), the coefficient of permeability at saturation, k_s , of the fine-grained layers of the capillary barrier models (i.e., RG-50, RM-75, and RL-95) used in the numerical analyses was the one that was obtained from each upward infiltration test. Figures 5.15 and 5.16 show the drying and wetting permeability functions of the fine-grained layers of the capillary barrier models using the coefficient of permeability at saturation, k_s , measured in the upward infiltration tests.

Figure 5.15 Drying permeability functions of the RG-50, RM-75, and RL-95.

Figure 5.16 Wetting permeability functions of the RG-50, RM-75, and RL-95.

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The fitting parameters of the wetting permeability functions using the coefficient of permeability at saturation, k_s , measured in the upward infiltration tests are given in Table 5.11. The fitting parameters of the drying permeability functions using the coefficient of permeability at saturation, k_s , measured in the upward infiltration tests are practically the same as those of the drying permeability functions using the coefficient of permeability at saturation, k_s , measured in the falling head or the constant head permeability tests as shown in Table 5.10.

Table 5.11 Fitting parameter of the wetting permeability functions of the RG-50, RM-75, and RL-95.

No	Soils	Parameter p
1	RG-50	3.5
2	RM-75	3
3	RL-95	3.4

5.2.1.4. Shrinkage characteristics

In order to reduce the volume change potential of the residual soil, which will be used as the fine layer of a capillary barrier system, the residual soil was mixed with gravelly sand, medium sand, or lime at different percentages. Furthermore, shrinkage tests were conducted in order to study the shrinkage characteristics of the residual soil and the soil mixtures. Shrinkage relationships or plots of water content versus void ratio of the soils are presented in Figure 5.17, 5.18, 5.19, and 5.20. It is shown that the shrinkage curves of the soils are further from the saturation line ($S=1$) as the water content of the soils decreases. The effects of the addition of the gravelly sand, medium sand, or lime on the shrinkage characteristics of the residual soil are discussed in detail in Chapter 6.

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Figure 5.17 Shrinkage relationships of the residual soil-gravelly sand mixtures.

Figure 5.18 Shrinkage relationships of the residual soil-medium sand mixtures.

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Figure 5.19 Shrinkage relationships of the residual soil-lime mixtures (1 of 2).

Figure 5.20 Shrinkage relationships of the residual soil-lime mixtures (2 of 2).

5.2.2. Infiltration Characteristics of Capillary Barriers

One dimensional infiltration tests through a 1 m high soil column were conducted in order to study the infiltration characteristics of capillary barrier models under laboratory conditions. In this study, three series of infiltration tests on capillary barrier models were performed. Each series of infiltration test involved an upward infiltration test, a drawdown test, and a number of rainfall tests under various rainfall intensities and durations. The tested capillary barrier models involve columns of IC-RGgs-50, IC-RMgs-75, and IC-RLgs-95. The IC-RGgs-50 consisted of 50 cm thick RG-50 overlying 50 cm thick gravelly sand. The IC-RMgs-75 comprised 50 cm thick RM-75 overlying 50 cm thick gravelly sand, whilst the IC-RLgs-95 consisted of 50 cm thick RL-95 underlain by 50 cm thick gravelly sand.

The capillary barrier models were equipped with tensiometer-transducer system in order to allow the measurement of pore-water pressures. A TDR system, which consisted of seven waveguides, was also installed in order to measure the volumetric water content during the infiltration test. In addition, the amount of water coming out from the capillary barrier models was measured using an electronic weighing balance for continuous data measurement.

The arrow above the pore-water pressure head and the volumetric water content profiles indicates the movement direction of the profiles relative to their initial conditions. The upward infiltration test was conducted in order to measure the coefficient of permeability at saturation, k_s , of the fine-grained layer and coarse-grained layer. The k_s value measured during the upward infiltration test will be used in the numerical analyses. The drawdown test was performed in order to describe a drying process that was started from a fully saturated condition, as well as to provide the initial condition for the subsequent rainfall test. In the rainfall test, a transient process of wetting and, then, drying was investigated. The infiltration characteristics of capillary barrier models developed in this study are discussed in detail in Chapter 6.

5.2.2.1. Infiltration characteristics of RGgs-50 column

The infiltration characteristics of the capillary barrier model of the RG-50 (i.e., consists of 50% residual soil and 50% gravelly sand) overlying the gravelly sand, named IC-RGgs-50, are presented in this section. The infiltration tests consisted of an upward infiltration test, a drawdown test, and four rainfall tests with different rainfall intensities and durations.

The profiles of pore-water pressure head and total head in the upward infiltration test of the RG-50 overlying the gravelly sand are presented in Figure 5.21. The profiles of pore-water pressure head during the drawdown test are shown in Figure 5.22, whilst the profiles of pore-water pressure head during the rainfall tests are presented in Figures 5.25, 5.26, 5.30, 5.31, 5.35, 5.36, 5.40, and 5.41. Figure 5.23 shows the profiles of volumetric water content during the drawdown test. The profiles of volumetric water content during the rainfall tests are shown in Figures 5.27, 5.28, 5.32, 5.33, 5.37, 5.38, 5.42, and 5.43. The water infiltration rate (drainage velocity) during the drawdown test is given in Figure 5.24, whilst the drainage velocities during rainfall tests are shown in Figures 5.29, 5.34, 5.39, and 5.44. The curves of cumulative drainage volume during rainfall tests are presented in Appendix C-5 to Appendix C-9.

Figure 5.21 Profiles of pore-water pressure head and total head in the RGgs-50 column during the upward infiltration test of IC-RGgs-50Ui.

Figure 5.22 Profiles of pore-water pressure head in the RGgs-50 column during the drawdown test of IC-RGgs-50Dr.

Figure 5.23 Profiles of volumetric water content in the RGgs-50 column during the drawdown test of IC-RGgs-50Dr.

Figure 5.24 Drainage velocity in the RGgs-50 column during the drawdown test of IC-RGgs-50Dr.

Figure 5.25 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R1 (wetting process).

Figure 5.26 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R1 (drying process).

Figure 5.27 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R1 (wetting process).

Figure 5.28 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R1 (drying process).

Figure 5.29 Drainage velocity in the RGgs-50 column during the rainfall test of IC-RGgs-50R1.

Figure 5.30 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R2 (wetting process).

Figure 5.31 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R2 (drying process).

Figure 5.32 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R2 (wetting process).

Figure 5.33 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R2 (drying process).

Figure 5.34 Drainage velocity in the RGgs-50 column during the rainfall test of IC-RGgs-50R2.

Figure 5.35 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R3 (wetting process).

Figure 5.36 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R3 (drying process).

Figure 5.37 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R3 (wetting process).

Figure 5.38 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R3 (drying process).

Figure 5.39 Drainage velocity in the RGgs-50 column during the rainfall test of IC-RGgs-50R3.

Figure 5.40 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R4 (wetting process).

Figure 5.41 Profiles of pore-water pressure head in the RGgs-50 column during the rainfall test of IC-RGgs-50R4 (drying process).

Figure 5.42 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R4 (wetting process).

Figure 5.43 Profiles of volumetric water content in the RGgs-50 column during the rainfall test of IC-RGgs-50R4 (drying process).

Figure 5.44 Drainage velocity in the RGgs-50 column during rainfall test of IC-RGgs-50R4.

5.2.2.2. Infiltration characteristics of RMgs-75 column

In this section, results of the infiltration tests on the soil column of the RM-75 overlying the gravelly sand, designated as IC-RMgs-75, are presented. The RM-75, which acted as the fine-grained layer, was a soil mixture of 75% residual soil and 25% medium sand. The infiltration tests performed on the IC-RMgs-75 involved an upward infiltration test, a drawdown test, and five rainfall tests with different rainfall intensities and durations.

Profiles of pore-water pressure head and total head in the upward infiltration test of IC-RMgs-75 are shown in Figure 5.45. The profiles of pore-water pressure head during the drawdown test are shown in Figure 5.46, whilst the profiles of pore-water pressure head during the rainfall tests are presented in Figures 5.49, 5.50, 5.54, 5.55, 5.59, 5.60, 5.64, 5.65, 5.69, and 5.70. Figure 5.47 shows the profiles of volumetric water content during the drawdown test. The profiles of volumetric water content during the rainfall tests are presented in Figures 5.51, 5.52, 5.56, 5.57, 5.61, 5.62, 5.66, 5.67, 5.71, and 5.72. The drainage velocity during the drawdown test is shown in Figure 5.48, whilst the drainage velocity during the rainfall tests are presented in Figures 5.53, 5.58, 5.63, 5.68, and 5.73. The cumulative drainage volume during rainfall tests are shown in Appendix C-10 to Appendix C-15.

Figure 5.45 Profiles of pore-water pressure head and total head in the RMgs-75 column during the upward infiltration test of IC-RMgs-75Ui.

Figure 5.46 Profiles of pore-water pressure head in the RMgs-75 column during the drawdown test of IC-RMgs-75Dr.

Figure 5.47 Profiles of volumetric water content in the RMgs-75 column during the drawdown test of IC-RMgs-75Dr.

Figure 5.48 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75Dr.

Figure 5.49 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R1 (wetting process).

Figure 5.50 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R1 (drying process).

Figure 5.51 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R1 (wetting process).

Figure 5.52 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R1 (drying process).

Figure 5.53 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75R1.

Figure 5.54 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R2 (wetting process).

Figure 5.55 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R2 (drying process).

Figure 5.56 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R2 (wetting process).

Figure 5.57 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R2 (drying process).

Figure 5.58 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75R2.

Figure 5.59 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R3 (wetting process).

Figure 5.60 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R3 (drying process).

Figure 5.61 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R3 (wetting process).

Figure 5.62 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R3 (drying process).

Figure 5.63 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75R3.

Figure 5.64 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R4 (wetting process).

Figure 5.65 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R4 (drying process).

Figure 5.66 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R4 (wetting process).

Figure 5.67 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R4 (drying process).

Figure 5.68 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75R4.

Figure 5.69 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R5 (wetting process).

Figure 5.70 Profiles of pore-water pressure head in the RMgs-75 column during the rainfall test of IC-RMgs-75R5 (drying process).

Figure 5.71 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R5 (wetting process).

Figure 5.72 Profiles of volumetric water content in the RMgs-75 column during the rainfall test of IC-RMgs-75R5 (drying process).

Figure 5.73 Drainage velocity in the RMgs-75 column during the rainfall test of IC-RMgs-75R5.

5.2.2.3. Infiltration characteristics of RLgs-95 column

The infiltration characteristics of the soil column of the RL-95 overlying the gravelly sand, named as IC-RLgs-95, are shown in this section. As mentioned previously, the RL-95 is a soil mixture of 95% residual soil and 5% lime. The infiltration tests conducted on the IC-RLgs-95 consisted of an upward infiltration test, a drawdown test, and six rainfall tests with different intensities and durations.

The profiles of pore-water pressure head and total head in the upward infiltration test on RL-95 overlying gravelly sand are presented in Figure 5.74. The profiles of pore-water pressure head during the drawdown test are shown in Figure 5.75, while the profiles of pore-water pressure head during the rainfall tests are presented in Figures 5.78, 5.79, 5.83, 5.84, 5.88, 5.89, 5.93, 5.94, 5.98, 5.99, 5.103, and 5.104. Figures 5.76 shows the profiles of volumetric water content during the drawdown test, while Figure 5.80, 5.81, 5.85, 5.86, 5.90, 5.91, 5.95, 5.96, 5.100, 5.101, 5.105, and 5.106 show the profiles of volumetric water content during the rainfall tests. The water infiltration rate (drainage velocity) measured during the drawdown test is shown in Figure 5.77 and the drainage velocity measured during the rainfall tests

are shown in Figures 5.82, 5.87, 5.92, 5.97, 5.102, and 5.107. The curves of cumulative drainage volume during the drawdown and rainfall tests are presented in Appendix C-16 to Appendix C-22.

Figure 5.74 Profiles of pore-water pressure head and total head in the RLgs-95 column during the upward infiltration test of IC-RLgs-95Ui.

Figure 5.75 Profiles of pore-water pressure head in the RLgs-95 column during the drawdown test of IC-RLgs-95Dr.

Figure 5.76 Profiles of volumetric water content in the RLgs-95 column during the drawdown test of IC-RLgs-95Dr.

Figure 5.77 Drainage velocity in the the RLgs-95 column during rainfall test of IC-RLgs-95Dr.

Figure 5.78 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R1 (wetting process).

Figure 5.79 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R1 (drying process).

Figure 5.80 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R1 (wetting process).

Figure 5.81 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R1 (drying process).

Figure 5.82 Drainage velocity in the RLgs-95 column during the rainfall test of IC-RLgs-95R1.

Figure 5.83 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R2 (wetting process).

Figure 5.84 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R2 (drying process).

Figure 5.85 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R2 (wetting process).

Figure 5.86 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R2 (drying process).

Figure 5.87 Drainage velocity in the RLgs-95 column during the rainfall test of IC-RLgs-50R2.

Figure 5.88 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R3 (wetting process).

Figure 5.89 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R3 (drying process).

Figure 5.90 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R3 (wetting process).

Figure 5.91 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R3 (drying process).

Figure 5.92 Drainage velocity in the RLgs-95 column during the rainfall test of IC-RLgs-95R3.

Figure 5.93 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R4 (wetting process).

Figure 5.94 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R4 (drying process).

Figure 5.95 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R4 (wetting process).

Figure 5.96 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R4 (drying process).

Figure 5.97 Drainage velocity in the RLgs-95 column during the rainfall test of IC-RLgs-95R4.

Figure 5.98 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R5 (wetting process).

Figure 5.99 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R5 (drying process).

Figure 5.100 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R5 (wetting process).

Figure 5.101 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R5 (drying process).

Figure 5.102 Drainage velocity in the RLgs-95 column during the rainfall test of IC-RLgs-95R5.

Figure 5.103 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R6 (wetting process).

Figure 5.104 Profiles of pore-water pressure head in the RLgs-95 column during the rainfall test of IC-RLgs-95R6 (drying process).

Figure 5.105 Profiles of volumetric water content in the RLgs-95 column during the rainfall test of IC-RLgs-95R6 (wetting process).