Development of dual capillary barrier system using recycled materials

Fakhrur Rozy Harnas

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**DEVELOPMENT OF DUAL CAPILLARY BARRIER
SYSTEM USING RECYCLED MATERIALS**

FAKHRUR ROZY HARNAS

School of Civil and Environmental Engineering

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FAKHRUR ROZY HARNAS

School of Civil and Environmental Engineering

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ABSTRACT

At the end of operation, a landfill should be covered in order to reduce water infiltration. A capillary barrier (CB) is a cover system that uses the contrast in hydraulic properties between fine and coarse-grained layers under unsaturated conditions to minimize downward water movement. The performance of a CB in the tropics is tied to its ability to remove water through lateral diversion. Therefore, increasing the lateral diversion capacity of a capillary barrier is an important research area. The use of recycled concrete (RC) and recycled asphalt pavement (RA) is widely accepted in civil engineering applications such as highway pavement construction. Since coarse-grained soil are most suitable for capillary barrier systems, RC and RA could be used as alternative materials for forming capillary barrier cover systems. However, the use of these materials in capillary barrier cover systems has never been investigated.

The objectives of this study were to develop a modified capillary barrier system to reduce water infiltration into a landfill and to investigate the potential use of RC and RA as capillary barrier forming materials. The study consisted of laboratory experiments and numerical modelling. The laboratory experiments included an investigation of the properties of RC and RA followed by 1-D and 2-D infiltration experiments. Numerical modelling was conducted as preliminary guidance for the development of an experimental plan, and the results of the numerical modelling were then compared to the experimental data. The verified numerical model was subsequently used to perform a parametric study.

This study proposes a modification to the conventional capillary barrier cover (single capillary barrier, SCB). The modified capillary barrier, termed a dual capillary barrier (DCB), consists of two stacks of capillary barriers. The experimental results showed that DCB has a considerably higher water storage than SCB, which leads to a higher lateral diversion capacity. The higher water storage of DCB was caused by higher volumetric water content at breakthrough caused by the thinner thicknesses of the fine-grained layers of DCB and the effect of layering. Both SCB and DCB can be constructed using combinations of RC or RA, as demonstrated in this study. The low water-entry value of coarse RA and the low residual volumetric water content of fine RA results in high storage that make fine and coarse RA ideal materials for the construction of SCB or DCB.

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LIST OF SYMBOLS

h_t	=	total head
h_p	=	pressure head
h_e	=	elevation head
ψ	=	total suction,
$(u_a - u_w)$	=	matric suction,
u_a	=	pore-air pressure,
u_w	=	pore-water pressure,
π	=	osmotic suction
Θ_w	=	volumetric water content,
Θ_s	=	volumetric water content at saturation,
$C(\psi)$	=	a correction factor, which is given by :
e	=	natural number, 2.71828,
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).
Θ_1	=	saturated volumetric water content at the first point
Θ_2	=	saturated volumetric water content at the second point
ψ_1	=	suction at the first point
ψ_2	=	suction at the second point
$k_w (\theta_w)_i$	=	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/m ³).
g	=	gravitational acceleration (m/s ²).
μ_w	=	absolute viscosity of water (N s /m ²).
θ_s	=	volumetric water content at saturation (i.e., $S = 100\%$) (Green and Corey, 1971).
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, 1971).

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.
$k_r(\psi)$	=	the relative coefficient of permeability defined as:
$\Theta^q(\psi)$	=	a correction factor,
Θ	=	the normalized volumetric water content, which is equivalent to degree of saturation and defined as:
q	=	1 (Kunze et al., 1998),
Θ_w	=	volumetric water content,
Θ_s	=	volumetric water content at saturation,
ψ_a	=	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
v_w	=	flow rate of water
k_w	=	coefficient of permeability with respect to water phase,
		hydraulic head gradient in the y-direction, which can be
$\partial h_w / \partial y$	=	designated as i_w ,
C	=	time
	=	coefficient of water volume change with respect to a
m_2^w		change in matric suction, $(u_a - u_w)$,
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_w	=	interaction coefficient associated with the water phase partial differential equation
u_a	=	pore-air pressure
u_v	=	partial pressure of water vapour in air
D_v	=	diffusion coefficient of water vapour through the soil
P	=	precipitation
I	=	water intercepted by vegetation
R	=	surface run-off
ET	=	water lost by evapotranspiration
R_{ech}	=	water lost by groundwater recharge
ΔS	=	water stored in soil
E	=	rate of evaporation (mm/day)
e_s	=	saturation vapor pressure of water at the surface temperature (mmHg or kPa)
e_a	=	vapor pressure of the air in the atmosphere above the water surface (mmHg or kPa)
$f(u)$	=	turbulent exchange function which depends on the mixing characteristics of the air above evaporating surface

I	=	slope of the saturation vapor pressure versus temperature curve at the mean temperature of the air (mmHg/°C)
Q_n	=	all net radiation at the soil surface (mm/day of water)
η	=	psycrometric constant
E_a	=	$f(u) e_a (B-A)$
B	=	inverse of the relative humidity in the air
A	=	inverse of the relative humidity at the soil surface
$[B]$	=	gradient matrix
$[C]$	=	element stiffness matrix
$D1$	=	$Dvd1/\rho$
$[D2]$	=	$[Dvd2/\rho]$
$\{P\}$	=	vector of nodal pressures
$\{y\}$	=	vector of elevation heads
$[K]$	=	element hydraulic conductivity matrix
λ	=	$m_w \lambda_w$
$\langle N \rangle T \langle N \rangle$	=	$[M]$, the mass matrix
$\{P\}, t$	=	change in pressure with time
$q \langle N \rangle$	=	unit flux across the side of an element
Q_{max}	=	maximum total horizontal flux at breakthrough
k_s	=	saturated permeability
Φ	=	angle of the fine-coarse interface with respect to the horizontal
k_r	=	wetting phase relative permeability
k_w	=	unsaturated permeability
ψ	=	moisture potential
α	=	sorptive number
SC	=	storage capacity
z	=	thickness of soil cover
θ_w	=	volumetric water content
q	=	infiltration rate
k	=	coefficient of permeability
b	=	height of fine soil above interface
h_w	=	water-entry head of the coarse-grained layer.
NSC	=	net storage capacity
h_{gwl}	=	height of ground water table from the interface
$S_{DCB \max}$	=	Maximum storage of DCB
S_{DCB}	=	The water storage of DCB
$b1/b2$	=	Thickness of fine-grained layer of DCB

CHAPTER 1

INTRODUCTION

1.1. RESEARCH BACKGROUND

A large volume of waste is produced each day. These waste products are usually incinerated, composted or recycled. In most cases, these processes significantly reduce the volume of waste. However, there remains some amount of waste that must be stored in waste containment systems that are typically called landfills. The purpose of a landfill is to prevent the movement of waste and to control groundwater contamination by leachates from waste. To achieve these purposes, a bottom liner and a surface cover are constructed. The bottom liner is constructed at the beginning of a landfill's operation and therefore may not be present in older landfills. The surface cover is constructed at the end of a landfill's operation and thus may be the only containment system in the absence of the bottom liner, increasing the importance of the surface cover.

Current landfill surface cover design is based on regulations by the United States Environmental Protection Agency (USEPA) (1991) that prescribe covers for hazardous and municipal solid waste called the Resource Conservation and Recovery Act (RCRA) Subtitle C and Subtitle D, respectively. The barrier for such landfill cover designs is usually a low permeability material, such as compacted clay or a geomembrane. The performance of these materials is known to decrease over time due to desiccation cracking, installation defects, root penetration, etc. (Benson, 2007; Dwyer, 2003; Suter et al., 1993). Furthermore, these materials may not be available at every landfill location or may be very costly to source. This has driven the development of alternative landfill covers, which are recognized and supported by the USEPA (1998).

One of the alternative landfill cover systems is a capillary barrier. A capillary barrier is a mechanism used in a landfill cover system that utilizes unsaturated soil mechanics principles. The system is constructed by layering two materials of

contrasting permeability. The contrast in unsaturated permeability creates a capillary barrier that impedes the infiltrating water. The water is then removed by evaporation, transpiration and lateral diversion processes. Capillary barrier systems have been successfully used in arid and semi-arid regions with low rainfall and are highly dependent on evaporation and transpiration processes to remove water from the landfill cover.

Tami et al. (2004a) and Krisdani et al. (2010) investigated the use of capillary barriers as a slope stabilization technique in a tropical climate. Krisdani's (2010) experimental results showed that lateral diversion is the main water removal process in a capillary barrier system. Therefore, lateral diversion length is an important performance indicator of capillary barrier systems in the tropics. The lateral diversion length can be increased by several methods: increasing the thickness of the soil layer (Parent and Cabral, 2006; Kampf and Montenegro, 1997), increasing the slope angle (Khiire et al., 2000) or inserting an anisotropic transport layer (Morris and Stormont, 1999). Although several methods are available, more research is needed as certain methods may not be applicable in some conditions. For example, inserting an anisotropic transport layer requires a specific material that may not be locally available, or it may not be possible to increase the slope angle of the landfill due to development plans. Alternative approaches by modification of capillary barrier soil layer arrangements have never been investigated.

Capillary barriers have another advantage over other landfill soil cover systems in that they do not require low-permeability cohesive soil. Capillary barriers can be constructed using coarse-grained soil, such as sand or granite chips. Due to the scarcity of natural resources in Singapore, current construction practices try to promote the use of construction and demolition (C&D) waste as a substitute for coarse-grained soils, i.e., sand and granite chips. There are currently two popular C&D waste types used for civil engineering applications: recycled asphalt pavement (RA) and recycled concrete aggregate (RC). These materials are popular because they are easily sourced, environmental friendly and have been used for various application such as the base or sub-base layer for highway pavement construction (Halm, 1980; Ramaswamy et al., 1983), as slope stabilization and soil improvement

materials (Loehr et al., 2000; Consoli et al., 2002) and as a structural concrete aggregate (Viyanant, 2006). Nevertheless, the use of RC and RA as materials for the construction of capillary barriers has not been investigated.

1.2. OBJECTIVE

The present research has two main objectives. The first objective is to develop a modified capillary barrier system that has a higher lateral diversion capacity in order to reduce infiltration into landfills. The second objective is to investigate the potential use of RC and RA as materials for forming a capillary barrier system.

1.3. SCOPE OF WORK

The research consists of laboratory experiments and numerical modeling. The research can be divided into several parts as follows:

- Literature review of current cover systems.
- Proposal of a modified capillary barrier system using theoretical applications.
- Comprehensive laboratory investigation of the properties of RC and RA.
- A laboratory 1-D column test to investigate the suitability of RC and RA as materials for capillary barrier construction, to investigate the effect of evaporation on the capillary barrier and to investigate the increase in water storage of the modified capillary barrier system. Three laboratory column models were investigated.
- A laboratory 2-D box test to investigate the performance of a capillary barrier and modified capillary barrier. Three physical models of capillary barriers and modified capillary barriers were investigated.
- Numerical modeling was performed using SVFlux (Soilvision System Ltd., 2005) as preliminary guidance for the development of an experimental plan, and the results of the numerical modeling were compared to the experimental data to verify the numerical model. A parametric study using the verified numerical model was then conducted to investigate the effect of layer thickness.

1.4. OUTLINE OF THE THESIS

This thesis is organized as follows:

- Chapter 1 introduces the background, objectives and scope of the research.
- Chapter 2 reviews the existing work on the development of landfill cover by previous researchers.
- Chapter 3 presents the relevant equations and theories used in the study of soil cover. Based on this information, a dual capillary barrier system is proposed in this chapter.
- Chapter 4 discusses the research program of the study, which consists of laboratory experiments, numerical modeling and a parametric study. The equipment and procedures are also described in this chapter.
- Chapter 5 provides the results obtained from the laboratory experiments, numerical modeling and parametric study.
- Chapter 6 discusses the results obtained in the study.
- Chapter 7 provides the conclusions of the study and recommendations for future research.

CHAPTER 2

LITERATURE REVIEW

2.1. INTRODUCTION

This chapter reviews several topics related to research on landfill surface cover. An overview of the design of cover systems is presented in Section 2.2. The climatic condition of the landfill location is an important part of the design of a landfill cover system. Therefore, the climatic condition of Singapore is reviewed in Section 2.3. Section 2.4 evaluates current landfill surface cover systems used worldwide. This study proposes the use of construction and demolition (C&D) waste as an alternative landfill cover system. Therefore, the current use of C&D waste in civil engineering is reviewed in Section 2.5. A review of laboratory experiment and numerical modeling on landfill cover is presented in Section 2.6 and is followed by Section 2.7 which presents the concluding remarks.

2.2. COVER SYTEMS AND THE FLUX BOUNDARY PROBLEM

A cover system is one solution for contaminated site remediation and waste containment facilities. The concept of a cover system is very easy to understand, yet there are many challenges in modelling and designing a cover system (Fredlund et al., 2012). A cover system can be viewed as a thin interface placed between the atmosphere and the underlying soil or waste, as shown in Figure 2.1. As most - if not all - cover systems are under unsaturated conditions, cover system design follows the principle of water flow in unsaturated porous media. Water flow in soil under unsaturated conditions follows similar laws as in saturated conditions (i.e., Darcy's Law) (Richards, 1931; Childs and Collins-George, 1950). However, the coefficient of permeability of the materials is not constant, as it depends on the matric suction that the soil experiences. Due to this fact, the unsaturated properties of the material used for the cover system are critical for the performance and effectiveness of the cover system.

Figure 2.1 A cover system is an interface between waste and the atmosphere (Fredlund et al., 2012).

As the net moisture flux at the ground surface is required for the calculation of the net infiltration, a cover system needs to be designed based on local climatic conditions and the local hydrogeology of the site. Most geotechnical seepage problems use hydraulic head or no-flux conditions as the boundary conditions. However, a cover system needs to use moisture flux boundary conditions. These moisture flux boundary conditions are precipitation, evaporation, and transpiration. The determination of each boundary condition is a challenge for geotechnical engineers.

2.3 SINGAPORE CLIMATIC CONDITION

Singapore is situated in a typical tropical climate that is characterized by uniform temperature, high humidity, and abundant rainfall throughout the year. The average daily temperature ranges from 24°C to 31°C and the humidity ranges from 80% to 98%. The average annual rainfall amounts to 2300 mm in the central region of Singapore (Meteorological Service Singapore, 1997). Rainfall is usually greatest in the months from November to January; however, rain falls in all months of the year. An average of 179 rainy days occurs in a year and rainstorms are short, intense, and generally have a limited spatial extent (Toll et al., 1999). Rainfall intensity typically ranges from 20 to 50 mm/h, although short-term intensities can exceed 100 mm/h.

Figure 2.2 shows the rainfall intensity-duration-frequency (IDF) curve used for drainage design in Singapore. The average annual potential evaporation in Singapore is 2000 mm (Meteorological Service of Singapore, 1982-1995) with the daily evaporation rate in the range of 5 to 7 mm/day (Gasmo, 1997).

Rainfall and evaporation are components of water balance that greatly affect the design of a landfill cover. The amount of rainfall in Singapore is very high compared to arid or semi-arid regions. Therefore, landfill cover designs developed in arid or semi-arid regions may not be applicable to Singapore. The high potential evaporation in Singapore caused by the high temperatures from the sun that Singapore experiences year-round could benefit the design of the landfill cover by removing water from the soil surface.

Figure 2.2 Rainfall intensity-duration-frequency curves for Singapore (PUB, 1999).

2.3. EVALUATION OF CURRENT LANDFILL COVER SYSTEMS

Landfill cover provides three primary functions: 1. minimizing infiltration of water into waste; 2. isolating waste from receptors and controlling the movement of waste due to wind and water; and 3. controlling landfill gases. Laws and regulations governing municipal solid waste landfills used by the industry follow a law in the

United States titled the Resource Conservation and Recovery Act (RCRA). RCRA subtitle D barrier regulates municipal solid waste landfill covers. The prescribed landfill cover is known to not perform well. Therefore, the United States Environmental Protection Agency (USEPA) encourages the development of alternative landfill covers (USEPA, 1998). In the mining industry, cover systems were also used to prevent water and oxygen from reaching underlying waste materials. Guidelines on the design, construction and monitoring for cover systems are summarized in MEND (2004). The following sub-chapters describe in detail the prescribed landfill cover system and the alternatives of capillary barrier systems (CBS) and evapotranspiration cover systems (ET cover).

2.3.1. Prescribed landfill cover system

RCRA Subtitle D only requires a single barrier layer. This is the reason that RCRA Subtitle D is commonly called compacted clay cover. The compacted clay layer needs to have a saturated permeability of less than or equal to 10^{-7} m/s. The minimum thickness of the compacted clay layer is 45 cm and to minimize erosion of the final cover, a minimum of 15 cm thick vegetation layer needs to be installed above the compacted clay layer. The performance of compacted clay cover had been investigated using field measurements. Dwyer (2001) showed that percolation into a landfill covered by compacted clay in New Mexico was around 2% of the total precipitation during a 3 year period. Albright et al. (2004) reported that a landfill in Altamont, California, covered with compacted clay had percolation of less than 1% of the annual precipitation. Despite the good results found by both researchers, Benson et al. (2007) reported changes in compacted clay permeability up to 10000 times greater after 2 to 4 years of service. Similar findings were reported by Albright et al. (2006) for landfills in southern Georgia. Such findings were predicted by Suter et al. (1993) in their review of failure mechanisms for landfills covered by compacted clay. Desiccation cracking, freeze-thaw cycles, and plant root activity are several mechanisms of failure for compacted clay over the long term (Suter et al., 1993).

In order to address limitations of the compacted clay covers, geomembrane is commonly used as a second protection against water infiltration. Geomembranes are thin sheets of polymer (polyethylene or polyvinyl choride) that when put in place below the compacted clay layer form another barrier (USEPA 1991). The performance of compacted clay and geomembrane barrier is dependent on installation workmanship of the installer. Board and Laine (1995) and Crozier and Walker (1995) found an average of five holes per hectare in landfill covered with geomembrane. Several drawbacks of geomembrane are the increase in cost and the introduction of plane of weaknesses when the landfill is sloped. If the installation of geomembrane is performed perfectly, it has a very good performance such as reported by Albright et al. (2004) and Dwyer (2001).

Another geosynthetic material that is common in landfill cover is the Geosynthetic Clay Liner (GCL). It is usually used to replace the compacted clay cover. It consists of a bentonite clay that is sandwiched between geosynthetics. It is easier to install compared to compacted clay cover however the cost of GCL is quite high. The performance of landfill cover constructed using GCL is known to be excellent (Dwyer, 2001).

2.3.2. Alternative landfill cover systems

2.3.2.1 Evapotranspiration (ET) cover system

An ET cover system uses two natural processes to control percolation into the waste. Soil is first used to provide a natural water reservoir and, secondly, plant transpiration and natural evaporation empty the soil water reservoir (Hauser, 2009). An ET cover system usually consists of a single layer of soil with high porosity that can store the maximum amount of water. The cross sections of alternative landill cover systems are shown in Figure 2.3. A cross-section of an ET landfill cover is shown in Figure 2.3a. Several investigators (Nyhan et al., 1990; Anderson et al., 1999) have reported that covers utilizing soil and natural vegetation could prevent percolation into the waste. In addition, Hauser et al. (1999) reported that the construction of an ET cover is less costly than other types of barriers. Despite the

advantages, because of the dependency on plants, the potential of reuse for the landfill is limited.

Figure 2.3 Cover systems: a) ET cover and b) Capillary barrier cover.

2.3.2.2 Capillary barrier cover system

A capillary barrier utilizes the difference in unsaturated permeability between fine-grained and coarse-grained soil layers. Capillary action causes the layer of fine-grained soil overlying the coarser material to store more water than if there were no contrast between the layers. A cross-section of a capillary barrier landfill is presented in Figure 2.3b. The concept of a capillary barrier is presented in greater detail in Chapter 3. A capillary barrier cover system is effective when the amount of water infiltrating the capillary barrier is less than the combined effect of water storage, evaporation, and lateral drainage. Therefore, water does not percolate or break through into the waste layer. The moisture fluxes involved in a capillary barrier system are shown in Figure 2.4.

The fine-grained layer of the capillary barrier acts as storage and drainage of water. Hence, the hydraulic properties of the fine-grained material are very important to the effectiveness of the capillary barrier (Tami et al, 2004b). A study by Tami et al. (2004b) showed that a coefficient of permeability of 10^{-5} m/s is the threshold for the fine-grained layer. Soil with a coefficient of permeability higher than this threshold is usually categorized as silty sand. The selection of sand as the fine-grained layer

of the capillary barrier also reduces the effect of desiccation, cracking, and other volume change problems, e.g., settlement and heave on the performance of the capillary barrier.

Figure 2.4 Moisture fluxes in a capillary barrier system.

For the coarse-grained layer of the capillary barrier, Stormont and Anderson (1999) showed that the coarser the soil, the better the performance of the capillary barrier. Smersud and Selker (2001) suggested that 80% of the maximum lateral diversion can be obtained by having soil in the coarse-grained layer that has a D_{50} 2.5 times greater than that of the soil in the fine-grained layer. Tami et al. (2004b) reported that an important parameter of the soil in the coarse-grained layer is its water-entry value. Therefore, it is recommended to use soil with a water-entry value as low as possible for the coarse-grained layer.

Capillary barrier has also been used in the mining industry (Woyshner and Yanful 1995; Yanful and Choo 1997; Cabral et al. 2000; Adu-Wusu and Yanful 2006; Adu-Wusu et al. 2007). The contrast between fine and coarse-grained layers of capillary barrier restricts water infiltration through the interface between fine and coarse-grained layers. This restriction creates a high saturation in the fine-grained layer which in turns limit oxygen ingress into the waste materials. A few design

procedures have been proposed related to the use of capillary barrier in the mining industry. Yanful and Lin (1998) proposed a design processes and material consideration for a capillary barrier. Parent and Cabral (2006) developed a design method to optimize the thickness of capillary barrier layers based on minimum water infiltration.

Field measurements by Gee and Ward (1997), Warren et al. (1996), and Albright et al. (2004) of eight landfills with capillary barrier cover systems have shown that deep percolation is less than 5% of annual rainfall. Most of the instrumented capillary barriers were constructed in regions with arid or semi-arid climates where the annual precipitation is generally less than 1000 mm. By varying land slope by 5% from 0 to 25%, Nyhan et al. (1997) and Nyhan (2005) reported that an increase in the slope of the landfill decreases percolation into the waste. It is understood that lateral diversion should be the main water removal process in a capillary barrier system and because of this, Stormont (1996) suggested only using capillary barriers for small plots.

Currently, there is limited literature showing the use of a capillary barrier as a landfill cover in tropical climates. This may be because landfill cover systems were mainly evolved in developed countries in Europe and North America which have sub-tropical climates. In the tropics, Rahardjo et al. (2007) used a capillary barrier system as a slope stabilization technique to reduce rainfall-induced slope failures. Several slopes stabilized using a capillary barrier system have been instrumented (Rahardjo et al., 2012). Instrumentation results show that a capillary barrier system is effective in maintaining negative pore-water pressure below the capillary barrier system (Rahardjo et al., 2012). The effectiveness of a capillary barrier system in limiting water percolation was investigated by Tami et al. (2004a). Their laboratory experiments showed that water storage and lateral diversion capacity are the main factors affecting the performance of a capillary barrier (Tami et al., 2004b). The success of the capillary barrier system as a slope stabilization technique may be due to the fact that the slopes have a high inclination angle of 20 to 30 degrees. Landfill covers are usually built with a mild slope angle of 5 to 10 degrees. The performance of capillary barrier systems with mild slope angles in the tropics requires further research.

Besides increasing the slope angle, the lateral diversion of a capillary barrier can be increased by several other methods. Increasing the thickness of the fine-grained layer, as suggested by Tami et al (2004b), could increase its water storage ability and, in return, increase the lateral diversion of the capillary barrier (Ross, 1994). Increasing the saturated coefficient of permeability of the fine-grained layer could also significantly increase the lateral diversion. Stormont and Morris (1997) proposed the use of an unsaturated drainage layer or transport layer between the fine and coarse-grained layers of the capillary barrier. An illustration of a capillary barrier system with a transport layer is shown in Figure 2.5. In a parametric study exercise performed by Morris and Stormont (1999), the addition of a transport layer significantly improved the capillary barrier. The material for the transport layer should have a grain size between the fine-grained and coarse-grained layer materials.

Figure 2.5 A simple capillary barrier (left) and a capillary barrier with a transport layer (right).

The methods to increase the lateral diversion proposed by previous research require changing the fine-grained layer material properties, which may increase the cost of constructing the capillary barrier. Similarly, the addition of a transport layer requires another material, and this may not be available near the landfill site. Therefore, innovative methods are needed to increase the lateral diversion of a capillary barrier using only the existing materials.

2.4. THE USE OF CONSTRUCTION AND DEMOLITION WASTE IN CIVIL ENGINEERING

Substantial economic and population growth generate a large amount of construction and demolition waste. Around 1.70 million tonnes of construction and demolition waste were generated in Singapore in 2014 (MEWR, 2014). There are two main types of construction and demolition (C&D) waste: recycled concrete (RC) and recycled asphalt (RA). RC is generated through demolition of roads, airport runways, and buildings; RA is generated by the removal of pavement material that contains asphalt and aggregates. The removed RC and RA are initially crushed and screened before having any metal (if applicable) removed, followed by another crushing process to reach the desired gradation. Figure 2.6 shows pictures of RC and RA.

RC has been used in highway construction in the United States since the 1940s (Halm, 1980). Ramaswamy et al. (1983) found that RC is suitable as a base material in road pavement. Slope stabilization and soil improvement are other applications where RC can be used, as reported by Loehr et al. (2000) and Consoli et al. (2002), respectively. Similarly, the use of RA as pavement aggregates is accepted in the majority of states in the United States (Bosquez Jr, 2009). A study by Saeed et al. (1996) showed that RA is the best recycled material for road base construction aggregates as compared to seven other recycled materials. Coarse RA is also used as backfill material for embankment bases, while finer RA is mixed with fine-grained soil to improve its shear strength and volume change properties (FHWA, 2000). There is very limited literature reporting the use of RC or RA as a material for landfill cover.

As RC and RA are mainly recycled for road construction, suppliers commonly process RC and RA into fine and coarse RC/RA with similar sizes as the requirements for natural aggregates used in road construction. However, it is common to find more fine materials in RC and RA than in the corresponding natural aggregates as RC and RA contain other deleterious materials, such as cement and asphalt binders. The existence of cement that encapsulates the aggregates in RC makes RC have a lower specific gravity and higher water

adsorption compared to natural aggregates (FHWA, 2000). Finer RC has an even lower specific gravity and higher water adsorption than coarser RC due to the fact that more cement is found in finer RC. The cement that surrounds the aggregates may also contain un-hydrated concrete that can react with water to create secondary cementation (Poon et al., 2006). The cementation of RC has been acknowledged in the literature (Arm, 2001; Blankenagel et al., 2006; Poon et al., 2006); however, very limited research has been performed to understand the process and effects of secondary cementation on the engineering properties of RC. If RC was encapsulated by cement, RA was coated by asphalt binders. The asphalt binders result in contrasting water absorption properties as compared to RC. This property is commonly called hydrophobicity (Letey et al., 2000). A hydrophobic material usually has a contact angle of more than 90° and a drop of water cannot spontaneously infiltrate the soil surface. (Bauters et al., 2000).

Figure 2.6 Recycled materials: a. Recycled concrete aggregate (RC) and b. Recycled asphalt pavement (RA).

Previous studies have focused on evaluating the properties of RC/RA, specifically as replacements of natural aggregates for road and structural concrete (Viyanant, 2006). Hence, strength and stiffness properties were the main consideration. It has been found that concrete made from RC has a comparable performance to concrete

made using natural aggregates (Cuttel et al., 1997; Barkdale et al., 1992). Direct shear tests of RC and natural aggregates conducted by O'Mahony and Malligan (1991) showed that the shear strength properties of RC are similar to those of natural aggregates with similar gradation. A study comparing the strength of RA and natural aggregates by Bejarano et al. (2003) also showed that RA has a comparable shear strength and stiffness to natural aggregates with similar gradation.

In contrast to the strength and stiffness properties, the hydraulic properties of RC and RA, especially under unsaturated conditions, have not been well investigated. These properties are important parameters that govern the flow of water in these materials, which is critical for the design of landfill surface cover systems. Some data on hydraulic properties are available (Nokkaew et al., 2012; Rahardjo et al., 2009, 2013). However, the effect of cementation or hydrophobicity of RC or RA on the soil-water characteristic curve or the saturated permeability of RA/RC has never been discussed.

2.5. LABORATORY EXPERIMENTS AND NUMERICAL MODELING OF LANDFILL COVER

This section provides a summary on previous research works related to landfill cover system. The section is divided into two sub-sections. The laboratory experiments used in studying landfill cover system is presented in Section 2.5.1. This is followed by the summary of previous numerical studies on landfill cover system in Section 2.5.2.

2.5.1. Laboratory experiments

Laboratory experiments are often preferred over field tests when studying landfill cover systems because of the ability to control the tests under laboratory conditions with lower costs. Such laboratory experiments usually involve a full-scale model of the landfill cover. Soil column tests have been performed by several researchers to evaluate the performance of landfill covers in 1-D conditions. The existing experiments differed in the type of test, the material used, and the installed instrumentations, as illustrated in Table 2.1.

The major objective of the 1-D infiltration test is to observe the barrier effect provided by the interface between the fine-grained and coarse-grained layers or geosynthetics in a capillary barrier system, as performed by Stormont and Morris (2000), Yang et al. (2004a), Indrawan et al. (2007), McCartney and Zornberg (2010), and Sun et al. (2010). Other objectives include investigation of the effect of rainfall intensity on the performance of landfill cover (Bathurst et al., 2007) and the effect of evaporation on the performance of landfill cover (Yang and Yanful, 2002; McCartney and Zornberg, 2010).

Stormont and Morris (2000) showed that a capillary barrier is effective when the coarse-grained layer has a lower air-entry value. Yang et al. (2004a) confirmed the above and added that a ratio of 2 to 3 orders of magnitude in the coefficient of permeability is effective for producing the capillary barrier effect and the minimum thickness of the coarse-grained layer should be close to the water-entry value of the coarse-grained layer. Both Yang and Yanful (2002) and McCartney and Zornberg (2010) reported that evaporation does not produce a significant decrease in moisture after a certain depth. The depth affected by evaporation is governed by the soil-water characteristic curve, water table location, and diffusivity parameters (Yang and Yanful, 2002).

For landfill cover that relies on lateral diversion, such as a capillary barrier, a 1-D soil column test may not show the real performance of the landfill cover. A 2-D model is more suitable for this purpose. Miyazaki (1988) was one of the early researchers who constructed a 2-D model to investigate the diversion of a capillary barrier. He used several box dimensions and sandy loam as the fine-grained layer and gravel as the coarse-grained layer. His study showed that the capillary barrier is effective in removing water by diverting water at the interface of the fine and coarse-grained soil layers.

Kampf et al. (1997) performed a model test in a box with a length of 8 m and a height of 1 m to investigate the performance of a capillary barrier. His study showed that the diversion length is greatly affected by the slope angle. Bussiere et al. (2003) also performed a 2-D model test and their results showed that the

infiltration rate and slope angle are parameters that control the performance of a capillary barrier.

Tami et al. (2004b) also performed a 2-D model test and showed that the performance of a capillary barrier is controlled by the storage capacity and thickness of the fine-grained layer. Krisdani (2007) used residual soil and geosynthetics as a surface cover and showed that cracks can occur during evaporation and that geosynthetics can be used as the coarse-grained layer of a capillary barrier system. Zhan et al (2014) tested a physical model of a capillary barrier with a transport layer under high rainfall conditions and showed that the capillary barrier was effective in limiting water percolation. The results of the 2-D models by the various researchers mentioned above show that a 2-D model test is an adequate tool for investigating water flow for a landfill cover.

Table 2.1 Summary of 1-D soil column studies from the literature

Researchers	Type of test	Material	Instrumentation
Stormont and Morris (2000)	Infiltration	sand and geosynthetics	Tensiometers
Yang and Yanful (2002)	Infiltration and Evaporation	clayey sand	Tensiometers, TDR and thermocouple
Yang et al. (2004a)	Infiltration	sand	Tensiometers, TDR
Indrawan et al. (2007)	Infiltration	soil mixtures	Tensiometers, TDR
Bathurst et al. (2007)	Infiltration	geosynthetics	Tensiometers, TDR
Lee et al. (2010)	Infiltration	residual soils	Tensiometers, gypsum blocks
McCartney and Zornberg (2010)	Infiltration and Evaporation	sand and geosynthetics	Tensiometers, TDR and thermocouple
Sun et al. (2010)	Infiltration	soil mixtures	TDR

2.5.2. Numerical modelling

Water flow through a landfill cover is a complex and highly non-linear problem. This causes the analytical solution to be difficult to obtain without some assumptions and simplifications that can distort the correct answer. Therefore,

numerical modelling of water flow through a landfill cover is a popular and necessary technique in the analysis and design of landfill cover systems.

Morris and Stormont (1997) used the finite difference program Hydrologic Evaluation of Landfill Performance (HELP) developed by the US Environmental Protection Agency (USEPA) to investigate the water balance of three types of landfill cover. The HELP program is widely used because of its ease of use and the in-built weather data input. However, it cannot be used to evaluate capillary barrier cover performance because HELP uses a water routing routine that does not adequately represent the expected physical process found in the capillary barrier (Schroeder et al., 1994). Because of the in-built weather data, the HELP program is still being used to generate weather data input for other programs, such as the work by Morris and Stormont (1998) where they evaluated the performance of a capillary barrier cover. The weather data output from HELP are then inputted to another finite difference program, such as TRACR3-D or UNSAT-H, that are capable of simulating transient unsaturated flow in porous media (Travis and Birdsell, 1991).

Dwyer (2003) used UNSAT-H to investigate the water balance of six test covers. Dwyer (2003) noted that UNSAT-H is very sensitive to changes in soil properties, which is caused by the scalar multiplier used to calculate the unsaturated coefficient of permeability of the soil. A similar observation was reported by Fayer et al. (1992). Despite its shortcomings, Fayer and Gee (1997) reported that UNSAT-H can reasonably predict the water balance of a capillary barrier cover.

HELP, TRACR3-D, and UNSAT-H are finite difference programs. The finite difference method has a weakness in handling complex geometry as compared to other methods such as the finite element method (FEM) or finite volume method (FVM). VADOSE/W (Geo-Slope International, 2007) has been used by several researchers to investigate the water balance of landfill covers. Benson et al. (2004) showed that VADOSE/W can predict water balance in a landfill cover system more accurately than UNSAT-H. Similar results were reported by Adu-wusu et al. (2007) in their prediction of the water balance in Whistle Mine, Ontario. Another study by Song and Yanful (2008) demonstrated good agreement between field and predicted water balance simulated by VADOSE/W.

To ensure that a transient numerical modelling result converges to the correct solution, the time-stepping and size of the mesh should be adequately designed. An algorithm to adjust the time steps is available on VADOSE/W; however, the size of the mesh is not adjustable. SVFLUX (SoilVision International, 2009) is a finite element program developed to tackle water flow problems, especially in unsaturated soils. Unlike VADOSE/W which can only adjust the time-step sizes, SVFLUX can adjust both time-step and mesh sizes. This is critical, especially if the model has elements that require high numerical stability, such as drains.

Siemens and Bathurst (2010) performed a numerical investigation using SVFLUX to validate the performance of a soil-geotextile column and showed that the dynamic meshing available in SVFLUX is useful for maintaining numerical stability during the iteration process. Chen and Zhang (2006) also used SVFLUX to investigate the infiltration of water and seepage flow to a dam structure. The study successfully identified the probable locations of seepage failure that helped maintenance schedule planning for the dam.

2.6. CONCLUDING REMARKS

A review of the literature showed that RC and RA have been used as alternative materials for civil engineering applications. Nevertheless, RC and RA have never been used as materials for landfill cover. Capillary barriers have an advantage over other cover systems with regard to the use of RC and RA as capillary barriers do not require cohesive soil. However, the literature showed that the effectiveness of capillary barriers as landfill cover is highly dependent on the ability of the capillary barrier to laterally divert the infiltrated water. Therefore, increasing the lateral diversion of a capillary barrier is critical for it to be successful. In order to investigate the performance of capillary barrier cover, 1-D and 2-D laboratory experiments were conducted by previous researchers. The advantages of laboratory experiments as compared to field experiments are the lower cost and more controllable environment. The complex nature of water flow problems for solving cover system problems requires numerical modelling tools. Several software programs are available to perform transient numerical modelling.

CHAPTER 3

APPLICABLE THEORY

3.1. INTRODUCTION

Literature review shows that the concept of a landfill cover is based on unsaturated flow theory. Characteristics of unsaturated soils including the soil-water characteristic curve and permeability functions of soils are reviewed in Section 3.2. The governing equation of water flow including the equations on evaporation process and the numerical modeling formulation of water flow problems are presented in Section 3.3. The theoretical concept of capillary barrier cover including the analytical calculation for lateral diversion of capillary barrier is presented in Section 3.4. By considering the literature review and the applicable theory, a modified capillary barrier system is proposed and presented with supporting theoretical considerations in Section 3.5.

3.2. CHARACTERISTIC OF UNSATURATED SOILS

3.2.1. Background

An unsaturated soil is commonly referred to as a three phase system, i.e., solid phase, water phase, and air phase. The air-water interface, which is commonly referred to as the contractile skin, has been postulated as the additional fourth phase of unsaturated soil (Fredlund and Rahardjo, 1993). The soil above the groundwater table usually called as vadose or unsaturated zone can be divided into three parts depending on their degree of saturation, S . The first part is called the capillary fringe with S equal to or near to 100%. The second part is unsaturated with S being less than 100% with a continuous air phase and the third part is unsaturated with S near to 0% with air filling most voids.

3.2.2. Soil-water interaction

Potential energy of water in the soil governs the interaction between water and soil particles. The energy was developed by the difference in pressure, change in volume or other force product. Childs (1967) definition the potential energy as the amount of work which must be by an externally applied force in order to move a specified amount of water from one point to the other. The total or hydraulic potential is the sum of pressure potential and gravitational potential. The expression of potential in terms of a measure of head is shown below:

$$h_t = h_p + h_e \quad (3.1)$$

where:

$$\begin{aligned} h_t &= \text{total head} \\ h_p &= \text{pressure head} \\ h_e &= \text{elevation head} \end{aligned}$$

The pressure in a soil considered to be positive when soils are saturated below a water table whereas the pressure in the soil considered to be negative above the water table. This negative pressure commonly known as suction.

3.2.3. Theory of soil suction

Suction in unsaturated soils has two components, which are matric suction and osmotic suction. Matric suction is associated with the capillary phenomenon arising from the surface tension of water, while osmotic suction is related to the presence of dissolved salts in a soil (Fredlund and Rahardjo, 1993). In the field, the change in osmotic suction is relatively less significant as compared with the change in matric suction (Krahn and Fredlund, 1972). The relationship between total suction and its components can be expressed as follows:

$$\psi = (u_a - u_w) + \pi \quad (3.2)$$

where:

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

Matric suction is usually described using capillary model shown in Figure 3.1. 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 points 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:

$$u_w = -\rho_w g h_c \quad (3.3)$$

where:

g	=	acceleration due to gravity
ρ_w	=	density of water,

As shown in Figure 3.1 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,

$$(u_a - u_w) = \rho_w g h_c \quad (3.4)$$

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

3.2.4. Soil-Water Characteristic Curve (SWCC)

Soil-water characteristic curve (SWCC) can be defined as a relationship between volumetric water content and suction of a soil. The typical shape of SWCC is a sigmoidal curve where the water content of the soil decreases when the matric suction increases following the drying curve and water content increases when matric suction decreases following the wetting curve as shown in Figure 3.2.

Figure 3.2 shows some key characteristics of a SWCC curve. 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 in the drying curve. The air-entry value is the matric suction value that must be exceeded before the sample desaturates. Before the air entry value, the volumetric water content of soils remain close or equals to the saturated volumetric water content. Whereas beyond air the entry value, relatively large changes in of water content occurred under small changes in pressure. 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 . The water-entry value,

ψ_w is defined as the matric suction value at which water first enters the pores of soil in the wetting path. In many cases, the water-entry value is assumed to be equal to residual water content. If water is added continuously, the water content of soil increases until the residual air content, θ_a . The residual air content can be defined as the air content that is trapped in the soil in an occluded form. The slope of the SWCC indicates how much water retained in the soil for a given pressure change. The slope can be very mild or very steep depending on the pore-size distribution of the soil.

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

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, 1997).

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

$$\theta_w = C(\psi) \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{(u_a - u_w)^n}{a} \right) \right] \right\}^m} \quad (3.5)$$

where:

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

$$C(\psi) = \frac{\ln[1 + (u_a - u_w)/(u_a - u_w)_r]}{\ln[1 + (1,000,000/(u_a - u_w)_r)]} \quad (3.6)$$

- e = natural number, 2.71828,
 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.

3.2.5. Hysteresis

Hysteresis is caused by differences in size between the primary pores and interconnecting pore throats during the wetting and drying cycles which can be explained by using the "ink bottle" effect as shown in Figure 3.3. If an empty thin tube is placed in a water tank, the water inside the tube will rise until it reaches the maximum capillary height or its equilibrium state. This can be described as the wetting SWCC in Figure 3.2. If a full tube is placed in a water tank, the water inside the tube will drain until it reaches the equilibrium state. This can be assumed as the drying SWCC in Figure 3.2. If the radius of the tube is similar, the equilibrium state of both cases is at the same height. However, if the radius of the tube is different, the equilibrium condition will be different. Figure 3.3b shows that in the presence of enlarged bottleneck, the neck will prevent the water to rise beyond the base of the neck. On the other hand Figure 3.3c shows that in the presence of enlarged neck, the neck will prevent the water to drain beyond the top of the neck.

Figure 3.3 Ink bottle effect in a capillary tube (Taylor, 1984, after Fredlund and Rahardjo, 1993)

3.2.6. Permeability Functions

The coefficient of permeability of unsaturated soil, 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 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 , of the soil. Numerous empirical and statistical equations have been proposed to predict the permeability function for an unsaturated soil using the k_s and the SWCC of the soil.

Fredlund et al. (1994) proposed a permeability function prediction method 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 as:

$$k_r(\psi) = \Theta^q(\psi) \int_{\ln(\psi)}^b \frac{\theta_w(e^y) - \theta_w(\psi)}{e^y} \theta'_w(e^y) dy \bigg/ \int_{\ln(\psi_a)}^b \frac{\theta_w(e^y) - \theta_s}{e^y} \theta'_w(e^y) dy \quad (3.7)$$

where:

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

$$k_r(\psi) = \frac{k(\psi)}{k_s} \quad (3.8)$$

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

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

$$\Theta = \frac{\theta_w}{\theta_s} = S \quad (3.9)$$

where:

q	=	1 (Kunze et al., 1998),
θ_w	=	volumetric water content,
θ_s	=	volumetric water content at saturation,
θ'_w	=	derivative of Eq. (3.5),
ψ_a	=	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. WATER FLOW IN UNSATURATED SOILS

The main function of a landfill cover system is to limit water infiltration into the underlying waste materials. The landfill cover is commonly in unsaturated condition. Therefore, the design of landfill cover system is related to water flow in unsaturated soils. The equation that governs water flow in unsaturated soils is presented in Section 3.3.1. Boundary conditions and solutions of these problem need to satisfy the general water balance of the landfill cover. The general components of the water balance are explained in Section 3.3.2. There are two major water balance components that affect a landfill cover, infiltration and evaporation processes. These components are described in Section 3.3.3 and 3.3.4. As the problem of water flow in unsaturated soil is generally a non-linear problem, numerical modeling of water flow in unsaturated soils are necessary. The procedure and numerical modeling equation of water flow in unsaturated soils are described briefly in Section 3.3.5.

3.3.1 Governing equation for water flow in unsaturated soils

Darcy's law is commonly used to describe the flow of water in a saturated soil. Buckingham (1907), Richards (1931), and Childs and Collis-George (1950) indicated that Darcy's 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_w ,

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

$$\frac{\partial}{\partial x} \left(k_w \frac{\partial h_w}{\partial x} \right) + \frac{\partial}{\partial y} \left(k_w \frac{\partial h_w}{\partial y} \right) = m_2^w \rho_w g \frac{\partial h_w}{\partial t} \quad (3.11)$$

where:

m_2^w	=	coefficient of water volume change with respect to a change in matric suction, $(u_a - u_w)$,
k_w	=	coefficient of permeability with respect to water phase for the x - and y - directions (i.e., isotropic soil),

The Eq. 3.11 is only applicable in an isothermal condition. For a non isothermal condition, additional terms are introduced to incorporate water flow due to diffusion and advection processes (Wilson, 1990). The flow equation for a non isothermal condition in 1D condition (Y-direction) is as below:

$$\frac{\partial u_w}{\partial t} = -c_w \frac{\partial u_a}{\partial t} + \frac{1}{\rho_w g m_2^w} \frac{\partial}{\partial y} \left(k_w \frac{\partial u_w}{\partial y} \right) + \frac{\bar{u}_a + \bar{u}_v}{\bar{u}_a} + \frac{1}{\rho_w g m_2^w} \frac{\partial}{\partial y} \left(D_v \frac{\partial \bar{u}_v}{\partial y} \right) \quad (3.12)$$

where:

C_w	=	Interaction coefficient associated with the water phase partial differential equation
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u_a	=	Pore-air pressure
u_v	=	Partial pressure of water vapour in air
D_v	=	Diffusion coefficient of water vapour through the soil

3.3.2. Water balance in the vadose zone

Soil water balance consists of two main basic inputs, namely: atmospheric water balance and surface radiation balance. Blight (1997) stated that the soil water balance controls the state of moisture in the unsaturated soil. Figure 3.4 shows the component of water balance in the vadose zone as proposed by Blight (1997).

Blight (1997) proposed a soil water balance formula where water input is equal to water output and water stored in the soil. Water input or infiltration can be defined as the amount of precipitation minus the water intercepted by the vegetation and the surface runoff. The water output can be defined as water lost by the evapotranspiration and the groundwater recharge. Water stored in the soil can be defined as the total change in moisture in the soil. The Blight equation can be written as:

$$P - (I + R_{off}) = ET + R_{ech} + \Delta S \quad (3.13)$$

where:

P	=	Precipitation
I	=	Water intercepted by vegetation
R	=	Surface run-off
ET	=	Water lost by evapotranspiration
R_{ech}	=	Water lost by groundwater recharge
ΔS	=	Water stored in soil

Figure 3.4 Water balance in the vadose zone (From Blight, 1997)

3.3.3 Infiltration process

Infiltration can be defined as the penetration of water into soils. Infiltration can be a result of precipitation or irrigation. Intensity of precipitation and saturated coefficient of permeability (k_s) are the main factor that affect infiltration process. When the precipitation intensity is lower than the saturated coefficient of permeability, the water can infiltrate the soil surface without any ponding. However, if the precipitation intensity is higher than the saturated coefficient of permeability, the water will pond on the soil surface.

Analytical calculations have been proposed for steady-state and transient-state 1-dimensional infiltration. Kisch (1959) presented a study on infiltration of fine overlying coarse-grained soils. By combining Eq. 3.1 and 3.10, the following relationship is created:

$$dh_e = \frac{-dh_p}{(q/k(\phi)) + 1} \quad (3.14)$$

where:

q = Infiltration rate

$K(\phi)$ = Coefficient of permeability at corresponding suction,

By starting at a known pressure or boundary, the steady-state pore-water pressure profile can be calculated and then using the soil-water characteristic curve, the volumetric water content profile can also be drawn. The Kisch (1959) method can also be implemented in a spreadsheet solution such as presented in Barbour (1990).

A simpler method called linear method initially proposed by Bews et al. (1997) was used by Parent and Cabral (2006) to determine the pore-water pressure profile under steady-state condition. The method utilized the permeability function of the material and the rainfall infiltration rate. The assumption use in the linear method is that the coefficient of unsaturated of permeability across the soil was equal to the rainfall infiltration rate. By using the permeability function of the material, the suction value at a specified infiltration rate can be discovered. This suction value is defined as critical suction $((u_a - u_w)_c)$. It is assumed that suction increases linearly with elevation until $(u_a - u_w)_c$ is reached. The suction becomes constant above $(u_a - u_w)_c$.

3.3.4 Evaporation process

Evaporation is a change in water state from liquid to vapour due to an increase of water kinetic energy. During evaporation, hydrogen bonds breaks down and the water vapor diffuses from higher to lower vapor pressures, i.e., from the ground surface to the atmosphere. Evaporation from soil is an important flux boundary condition that should be defined to calculate the soil water balance for a landfill cover.

Evaporative flux modeling has been limited to methods that predict the unit flux evaporation rate based on a potential evaporation (PE) value. The potential evaporation term introduced by Thornthwaite (1948) is defined as the upper limit or maximum rate of evaporation from a saturated ground surface under given climatic

conditions. The potential rate of evaporation may be computed using the Dalton-type equation as follows (Gray, 1970):

$$E = f(u)(e_s - e_a) \quad (3.15)$$

where:

E	=	Rate of evaporation (mm/day)
e_s	=	Saturation vapor pressure of water at the surface temperature (mmHg or kPa)
e_a	=	vapor pressure of the air in the atmosphere above the water surface (mmHg or kPa)
$f(u)$	=	Turbulent exchange function which depends on the mixing characteristics of the air above evaporating surface

Equation 3.14 has become the basis of widely used Penman equation (1948) for PE calculation. It is well known that the rate of actual evaporation (AE) only equals to potential evaporation (PE) when the soil is saturated. The AE decreases as the soil surface becomes unsaturated and the supply of water becomes limited (Gray, 1970). Figure 3.5 shows a typical relationship for the ratio of actual and potential evaporation (AE/PE) against the water availability for the sand surface. The shape of the curve is described by Hillel (1980) as having three stages of drying. Stage I is the maximum rate of drying that occurs when the soil surface is near saturation. Stage II is when the conductive properties of soil becomes so low that restrict sufficient flow of water to the surface and the maximum rate of evaporation can not be maintained. Stage III is after the soil surface becomes desiccated and the liquid-water phase becomes discontinuous. It can be seen that the actual rate of evaporation is controlled by both climatic conditions and also the soil properties such as hydraulic conductivity and vapor diffusivity.

Figure 3.5 The relationship between the ratio of actual evaporation and potential evaporation against water availability (Hillel, 1980)

Wilson (1990) proposed a method for the calculation of evaporation rate from unsaturated soil surface. The method is based on modification of the Penman (1948) equation. The equation is as follow:

$$E = \frac{\Gamma Q_n + \eta E_a}{\Gamma + \eta A} \quad (3.16)$$

where:

Γ	=	Slope of the saturation vapor pressure versus temperature curve at the mean temperature of the air (mmHg/°C)
Q_n	=	All net radiation at the soil surface (mm/day of water)
η	=	Psychrometric constant
E_a	=	$f(u) e_a (B-A)$
B	=	Inverse of the relative humidity in the air
A	=	Inverse of the relative humidity at the soil surface

The Wilson equation (1990) reduces to conventional Penman (1948) equation when the surface is saturated as the soil will have a relative humidity of 100% and A of one. The relative humidity of the soil surface is evaluated by simultaneously solving the coupled moisture and heat flow as shown in Section 3.3.1.

3.3.4. Modeling of water flow in unsaturated soil

Steady state and transient water flow in soil through unsaturated soil can be analyzed by solving the governing equations of water flow as explained in Section 3.3.1. The equation can be solved using the finite element method (FEM). The procedures involved in the solving process are: 1. Discretization of the area or geometry into elements. 2. Determination of permeability matrix for each element. 3. Construction of matrix for the entire problem. 4. Boundary condition implementation. 5. Solving the equation. 6. Calculation of results.

The finite element formulation of water flow with vapor coupling (Geoslope, 2007) can be written as:

$$\int_V ([B]^T [C] [B]) dv \{P\} + \int_V ([B]^T [D_2] [B]) dv \{T\} + \int_V ([B]^T [K] [B]) dv \{y\} + \int_V (\lambda \langle N \rangle^T \langle N \rangle) dv \{P\}, t = q \int_A (\langle N \rangle^T) dA \quad (3.17)$$

where:

$[B]$	=	gradient matrix
$[C]$	=	element stiffness matrix
$D1$	=	$Dvd1/\rho$
$[D2]$	=	$[Dvd2/\rho]$
$\{P\}$	=	vector of nodal pressures
$\{y\}$	=	vector of elevation heads
$[K]$	=	element hydraulic conductivity matrix
λ	=	$m_w \lambda_w$
$\langle N \rangle^T \langle N \rangle$	=	$[M]$, the mass matrix
$\rangle$	=	change in pressure with time
$\{P\}, t$	=	unit flux across the side of an element
$q \langle N \rangle$	=	vector of interpolating function

The hydraulic heads are the unknown variables in Equation 3.17. The hydraulic heads are calculated using a string of iterative procedures. The initial boundary condition provide the model with an initial value of coefficient of permeability. The permeability was then used to calculate a new sets of hydraulic head depending on the flux boundary applied. The new sets of hydraulic head was used to obtain another set of coefficient of permeability for the calculation of the following hydraulic head. This process is repeated until the error value for both the hydraulic head and permeability functions is smaller than a specified tolerance. The convergence on hydraulic head and permeability function is largely dependent on the degree of non linearity of permeability function and the discretization of the problem.

3.4. CONCEPT OF CAPILLARY BARRIER

A capillary barrier is a cover system that takes advantage of a capillary phenomenon. In its simplest form, it consists of a fine-grained soil layer underlain by a coarse-grained soil layer. The permeability function of the layers forming the capillary barrier is the most important attribute affecting the performance of the capillary barrier. The contrast in the unsaturated permeability forms the hydraulic impedance that limits downward moisture movement (Khire et al., 2000).

Figure 3.6 and Figure 3.7 show a typical SWCC and permeability function for fine-grained and coarse-grained layers in a capillary barrier system. The mechanism of the capillary barrier can be explained by considering Figure 3.6. At high matric suction values, the coefficient of permeability of the coarse-grained layer is lower than the coefficient of permeability of the fine-grained layer. Under this condition, water does not flow into the coarse-grained layer. Water is stored and laterally diverted through the fine-grained layer. As the water infiltration continues, the volumetric water content of the fine-grained layer increases and the coefficient of permeability of the fine-grained layer also increases. The matric suction of the coarse-grained layer decreases when the infiltrating water reaches the fine-coarse interface. Water can only enter the coarse-grained layer when the matric suction at the interface decreases to the water-entry value of the coarse-grained layer. If the

matric suction value at the interface reaches a value lower than the water-entry value of the coarse-grained layer, water will rapidly move into the coarse-grained layer because the permeability increases to the saturated permeability of the coarse-grained layer and the barrier effect is diminished.

Figure 3.6 Permeability function of typical capillary barrier materials (after Harnas et al., 2013)¹

Figure 3.7 SWCC of typical capillary barrier materials (after Harnas et al., 2013)¹

¹ The fine-grained soil and coarse-grained soils in the graphs are FRCA and CRAP materials in Harnas et al. (2013).

3.4.1. Water storage of a capillary barrier

The total storage of a simple soil cover is the amount of water that the soil can store and can be described by:

$$SC = \int_0^z \theta_w dz \quad (3.18)$$

where:

SC	=	storage capacity
z	=	thickness of soil cover
θ_w	=	volumetric water content

In order to calculate the amount of water storage in a capillary barrier, a method proposed by Stormont and Morris (1998) is commonly used. The suction profile in the fine-grained layer at breakthrough needs to be estimated in order to get the moisture content profile. The suction profile can be calculated using the Kisch method (1959) or linear method (Bews et al., 1997). Stormont and Morris (1998) utilized the fact that breakthrough occurs when the water-entry value of the coarse-grained layer has been reached. Based on this fact, the water-entry value of the coarse-grained layer can be selected as the boundary in the pore-water pressure profile of the fine-grained layer of the capillary barrier. For a slow infiltration, it is assumed that the suction profile in the fine-grained layer behaves as a unit slope or unit gradient ($dh/dz=1$). A unit slope suction profile means that the increase in suction is equal to the increase in elevation. For a fast infiltration, the pore-water pressure profile behaves as a unit of slope until an elevation of b is reached, where b can be calculated by the following relationship:

$$q = k(b + h_w) \quad (3.19)$$

Where:

q	=	infiltration rate
k	=	coefficient of permeability
b	=	height of fine soil above interface
h_w	=	water-entry head of the coarse-grained layer.

After the height of b is reached, the suction profile becomes constant up to the surface of the capillary barrier. An illustration of the pore-water pressure profile obtained from this method is shown in Figure 3.8.

The estimated pore-water pressure profile at breakthrough is then used to construct a volumetric water content profile. For a slow infiltration case, the total storage capacity can be calculated as:

$$SC = \int_0^z \theta_w (z + h_w) dz \quad (3.20)$$

If the pore-water pressure at the initial condition is in a hydrostatic condition where the ground water table is at a specified elevation below the interface, then the net storage capacity can be calculated as:

$$NSC = \int_0^z \theta_w (z + h_w) dz - \int_0^z \theta_w (z + h_{gwl}) dz \quad (3.21)$$

Where:

NSC = net storage capacity

h_{gwl} = height of ground water table from the interface

Figure 3.8 Pore-water pressure profile of a capillary barrier at breakthrough.

3.4.2. Lateral diversion

The lateral diversion of a capillary barrier can be calculated using the formula developed by Ross (1990). The assumptions used in the Ross (1990) formulations are: steady-state infiltration, a semi-infinitely thick layer, and a quasi linear approximation for the relationship between the relative permeability and pressure head. Ross defined the diversion capacity as the maximum water volume flowing laterally per unit area of the fine-grained layer above the interface of the fine and coarse-grained layers before entering the coarse-grained layer. The diversion length is defined as the distance in the down-dip direction from the crest of the capillary barrier to the point where the rate of water percolation into the coarse-grained layer is similar to the rate of rainfall. Figure 3.9 shows an illustration of the diversion length and diversion capacity.

The total horizontal flow in the fine layer $Q_{\max}$ is equal to the total flux of water diverted until breakthrough occurs. It can be calculated using an equation as follows (Ross, 1990):

$$Q_{\max} = k_s \tan \phi \int k_r d\psi = \tan \phi \int k_w d\psi \quad (3.22)$$

where,

$Q_{\max}$	=	maximum total horizontal flux at breakthrough
k_s	=	saturated permeability
ϕ	=	angle of the fine-coarse interface with respect to the horizontal
k_r	=	wetting phase relative permeability
k_w	=	unsaturated permeability
ψ	=	moisture potential

The relative permeability function used by Ross is the quasi-linear function, or

$$k_r = e^{\alpha\psi} \quad (3.23)$$

where,

α	=	sorptive number
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Figure 3.9 Illustration of diversion length and diversion capacity of a capillary barrier
(Tami, 2004c)

The integration limits are the matric suction at the top and bottom of the fine-grained layer. A steady-state, one-dimensional vertical flow of the infiltrating water is assumed. At the top surface of the fine-grained layer, the relative permeability is simply the infiltration rate divided by the saturated permeability of the fine layer. At the top surface of the coarse-grained layer, the moisture potential at the top of the coarse-grained layer is similar to the moisture potential at the bottom of the fine-grained layer. With these integration limits, Equation 3.22 can be reduced to a closed form solution:

$$Q_{\max} = k_s \frac{\tan \phi}{\alpha} \left[\left(\frac{q}{k_s^*} \right)^{\frac{\alpha}{\alpha^*}} - \frac{q}{k_s} \right] \quad (3.24)$$

where the unstarred and starred values are the fine-grained and coarse-grained layer parameters, respectively.

For a constant infiltration, the capillary diversion length, L , is the total water volume diverted by the capillary barrier divided by the infiltration rate:

$$L = k_s \frac{\tan \phi}{q \alpha} \left[\left(\frac{q}{k_s^*} \right)^{\frac{\alpha}{\alpha^*}} - \frac{q}{k_s} \right] \quad (3.25)$$

Several analytical equations have been proposed to calculate the diversion length of a capillary barrier (Bussiere et al., 2007). Steenhuis et al. (1991) used the water-entry value of coarse-grained material instead of the pressure that corresponds to the infiltration rate in the SWCC of the coarse-grained layer used by Ross (1990). Steenhuis et al. (1991) used the permeability function developed by Ritjema (1965) to overcome the weakness of Ross's (1990) permeability quasi-linear approximation. Another researcher, Webb (1997), revised the Ross (1990) formulation to accommodate the van Genuchten and Mualem (1980) permeability function curve. Warrick et al. (1997) extended Ross's (1990) equation to include finite thickness of the fine-grained layer. The finite thickness of the fine-grained layer is accounted for by changing the integration limit at the top to equal the maximum matric suction attained by the fine-grained layer instead of the matric suction corresponding to the infiltration rate of Ross's (1990) equation.

3.5. PROPOSED MODIFIED CAPILLARY BARRIER SYSTEM

The proposed modified capillary barrier is called the dual capillary barrier (DCB). An illustration of the modified capillary barrier is shown in Figure 3.10. The arrangement of the modified capillary barrier consists of two stacks of single capillary barriers (SCB). The DCB has two fine-grained layers and two coarse-grained layers as opposed to the one fine-grained layer and one coarse-grained layer of the SCB. The advantages of a DCB as compared to a SCB of the same thickness are the increase in storage capacity that leads to the increase in lateral diversion length. The increase in storage capacity and diversion length are explained in following sections. A cross-section of the DCB and SCB that explains the increase in storage capacity and lateral diversion length is shown in Figure 3.11.

Figure 3.10 Proposed double capillary barrier (DCB).

Figure 3.11 Cross-sections of SCB and DCB as an explanation of the increase in storage capacity and lateral diversion length.

3.5.1. Increase in water storage capacity

The increase in water storage can be illustrated using an example illustrated in Figure 3.12, Figure 3.13 and Figure 3.14. Figure 3.12 shows the pore-water pressure profile at initial and breakthrough conditions for SCB and DCB. The initial condition for both SCB and DCB is a hydrostatic condition with an assumed water table at the bottom of the cover ($z=0\text{m}$). The idealized pore-water pressure profile at breakthrough condition for a capillary barrier has been reported by Stormont and Morris (1998). For SCB, at breakthrough condition, the value of pore-water pressure at the bottom of fine grained layer is the water-entry value of the coarse-grained layer. The pore-water pressure decreases linearly (45°) with increasing

height of the fine-grained layer (unit slope profile). For DCB, at breakthrough condition, similar principle to that of SCB applies to the pore-water pressure profile at the fine-grained layers. The pore-water pressure profile at the upper coarse-grained layer is assumed to be constant as represented by a vertical line with a value of the water-entry value of the coarse-grained layer.

In order to obtain the highest storage capacity, the volumetric water content at breakthrough should be at the highest volumetric water content condition. Figure 3.12 shows that at breakthrough condition, the range of pore-water pressure that affects the water storage of the SCB is between the water-entry value of the coarse-grained layer (assumed to be 0.5 kPa) and the limiting pore-water pressure that corresponds to the thickness of the fine-grained layer and the water-entry value of the coarse-grained layer (55 cm or 5.5 kPa). Similarly, the range of PWP that affects the water storage of DCB is from 0.5 kPa to 3 kPa. These PWP ranges can be defined as the important PWP ranges that affect the water storage of SCB and DCB, respectively.

From the pore-water pressure profiles shown in Figure 3.12 and the water-characteristic curve (WCC) of the fine- and coarse-grained materials shown in Figure 3.13, the volumetric water content profiles of SCB and DCB can be drawn. The storage capacity of SCB and DCB is calculated by measuring the area bounded by the volumetric water content profile at the initial condition and the volumetric water content profile at breakthrough condition. The typical storage capacity of DCB can be seen to be higher than the storage capacity of SCB as shown in Figure 3.14. The mass of water stored in the fine-grained layer of SCB or DCB can be calculated by integrating the WCC of the fine-grained layer material with respect to suction. The bottom integration limit is the water-entry value of coarse-grained layer and the top integration limit is the respective limiting suction. The integrations show that the storage for a SCB is $0.035 \text{ m}^3/\text{m}^2$ while for a DCB is $0.038 \text{ m}^3/\text{m}^2$. An increase in water storage of around 9% is achieved. The increase in water storage of DCB can be attributed to the fact that the smaller thickness of each fine-grained layer of DCB provides a range of suction where volumetric water content at breakthrough of the material is higher compared to the volumetric water content at

breakthrough of SCB. Detail explanations on the increase in water storage capacity of a DCB are presented in Harnas et al. (2014) and Rahardjo et al. (2013a).

Figure 3.12 Suction profile at initial and at breakthrough conditions for a SCB and DCB.

Figure 3.13 Typical SWCC for fine-grained soil and coarse-grained soil (after Harnas et al., 2013^{*2})

² *The data for SWCC was taken from Harnas et al. (2013) FRCA for fine-grained soil and CRAP for coarse-grained soil, respectively

Figure 3.14 Volumetric water content profiles at initial and breakthrough conditions for a SCB and DCB.

3.5.2. Increase in lateral diversion length

By having two fine-grained layers, water is expected to flow in the two fine-grained layers as compared to the one layer in the SCB. As a result, the lateral diversion length of the system is increased. Figure 3.15 shows an illustration of a lateral diversion of a DCB.

Several analytical formulations have been developed to calculate the lateral diversion length of a capillary barrier system. Most of the formulations are based on Ross's (1990) equation with different applied boundary integration limits applied. Ross's (1990) equation is an integration equation with top and bottom integration limits (Equation 3.22). The bottom and top integration limits for Ross's (1990) equation are dependent on the infiltration rate applied. The bottom integration limit for Ross's (1990) equation is the matric suction that corresponds to the precipitation rate obtained from the permeability function curve of the coarse-grained layer. The top integration limit for Ross's (1990) equation is the matric suction that

corresponds to the precipitation rate obtained from the permeability function curve of the fine-grained layer.

Figure 3.15 Lateral diversion of a DCB.

An illustration of the top and bottom integration limits of Ross's (1990) equation for an infiltration rate of 1×10^{-5} m/s is shown in Figure 3.16. The permeability function data for the illustration of the increase in lateral diversion length were taken from Harnas et al. (2013). The fine grained soil is FRCA and the coarse-grained soil is CRA obtained from the publication, respectively.

Figure 3.16 Illustration of Ross's (1990) equation boundary conditions.

The integration limits of Ross's (1990) equation were modified by several researchers. Steenhuis (1991) proposed the use of the water-entry pressure of coarse-grained soil as the bottom integration limit. The Steenhuis's (1991) bottom integration limit is widely accepted by other researchers (Bussiere et al., 2007). On the other hand, the top integration limit of Ross's (1990) equation was modified by Warrick et al. (1997) to account for finite thickness. Warrick et al.'s (1997) top integration limit is the maximum matric suction attained at the fine-grained layer.

The integration limits for capillary barrier lateral diversion capacity calculations can be illustrated by Figure 3.17 for an example of an infiltration rate of 5×10^{-5} m/s and a 50 cm thick fine-grained layer. The bottom integration limit from Ross's (1990) equation is 0.5 kPa. This is similar to Steenhuis's (1991) bottom integration limit of the water-entry value of the coarse-grained layer obtained from Figure 3.14. For the top integration limit, Ross's (1990) integration limit is 9 kPa, while Warrick et al.'s (1997) top integration limit is 5.5 kPa. Warrick et al.'s (1997) top integration limit is calculated as the sum of the bottom boundary condition (0.5 kPa) and the pressure corresponds to the 50 cm thickness of the fine-grained layer (5 kPa).

By utilizing Steenhuis et al.'s (1991) bottom integration limit and Warrick et al. (1997)'s top integration limit for Ross's (1990) equation, the lateral diversion capacity of a DCB and SCB (cross-section shown in Figure 3.11) can be evaluated. The lateral diversion capacity of a SCB is shown as the highlighted area (area I) in Figure 3.18. The area is limited by a bottom integration limit of 0.5 kPa (water-entry value of the coarse-grained layer in Figure 3.14) and a top integration limit of 5.5 kPa. The lateral diversion capacity of a DCB is provided by two fine-grained layers. The top fine-grained layer has a bottom integration limit of 0.5 kPa and a top integration limit of 3 kPa, and the bottom fine-grained layer has similar integration limits as the top fine-grained layer. The lateral diversion capacity produced by the top and bottom fine-grained layers of a DCB are shown as the highlighted area in Figure 3.19 (area II) and Figure 3.20 (area III), respectively. The total lateral diversion capacity of a DCB is the sum of the lateral diversion capacity produced by the top and bottom fine-grained layers (sum of area II and area III). Figure 3.18, Figure 3.19 and Figure 3.20 indicate that the lateral diversion capacity of a DCB (sum of area II and III) is higher than the lateral diversion capacity of a SCB (area

I). The lateral diversion length can be calculated as the lateral diversion capacity divided by the infiltration rate. Therefore, higher lateral diversion capacity of a DCB yields a higher lateral diversion length of a DCB compared to that of SCB.

Figure 3.17 Lateral flux diverted by a capillary barrier with a finite thickness.

Figure 3.18 Lateral diversion capacity of a SCB.

Figure 3.19 Lateral diversion capacity provided by the top fine-grained layer of a DCB.

Figure 3.20 Lateral diversion capacity provided by the bottom fine-grained layer of a DCB.

3.6. CONCLUDING REMARKS

The theories applicable to this study are presented in this chapter. Unsaturated soil characteristics are the basis of capillary barrier cover system and has been explained in this chapter. Principles of water flow in unsaturated soil and the concept of capillary barrier are also described in this chapter. Subsequently, based on these principles, a modified capillary barrier system, dual capillary barrier (DCB) system is proposed. The DCB system has two advantages; increase in water storage capacity and increase in lateral diversion capacity. The advantages of the DCB are described in details in this chapter.

CHAPTER 4

RESEARCH PROGRAM

4.1. INTRODUCTION

The proposed landfill cover system to be investigated is a modified capillary barrier system using construction and demolition waste materials of RC and RA. In order to investigate the proposed landfill cover system, two research methods were used: laboratory experiments and numerical simulations. The study was divided into four parts as follows. The first part was a laboratory investigation of RC and RA materials. The materials were tested for index properties, hydraulic properties, unsaturated soil properties, cementation potential and water hydrophobicity properties. The experimental procedures for the laboratory investigation of RC and RA are explained in Section 4.2.

The second part of the study was a laboratory column study. The objectives of the column study were to investigate the barrier effect produced by the different combinations of RC and RA, to investigate the increase in storage of the modified capillary barrier system and to investigate the effect of evaporation on the performance of the capillary barrier system. Preliminary numerical modeling was performed prior to the soil column study to select material combinations in order to obtain meaningful results from the column study. The procedures for the laboratory column study experiments are explained in Section 4.3.

The third part of the study was the laboratory box study. The objectives of the laboratory box study were to investigate the lateral diversion of the SCB and DCB at a mild slope of 2 to 7 degrees. Three physical models were constructed from recycled asphalt (RA) material. The first and second physical models were capillary barriers with different fine-grained layer thicknesses, whereas the third physical model was the dual capillary barrier model. The procedures for the laboratory box study are explained in Section 4.4.

The last part of the study was the numerical modeling. The numerical modeling results were verified by comparing the results of the numerical model with the

laboratory column and box study data. The verified numerical model was then used to observe the factors affecting the performance of SCB and DCB. The experimental procedures for the numerical modeling are presented in Section 4.5.

4.2. INVESTIGATION OF RECYCLED MATERIAL PROPERTIES

The recycled materials used in this study were fine recycled concrete aggregate (FRC), coarse recycled concrete aggregate (CRC), fine recycled asphalt pavement (FRA) and coarse recycled asphalt pavement (CRA). The test methods and equipment used in the laboratory tests are described in this section. The laboratory tests conducted in this study involved investigations of material's basic properties, hydraulic properties, cementation and hydrophobicity properties.

4.2.1. Basic properties

Investigations of the basic properties of the recycled materials included grain-size distribution analyses, measurements of soil specific gravity and relative density tests. The grain-size analyses were performed in accordance with ASTM D422 (2002a). The measurements of soil specific gravity were performed following ASTM D854 (2002b). The soils were classified based on the unified soil classification system (USCS) following ASTM D2487 (2002c).

4.2.2. Hydraulic properties

4.2.2.1. Constant head permeameter

The saturated permeability of the recycled materials was measured using a constant head method in a flexible wall permeameter (ASTM D5084, 2002d). In the flexible wall permeameter, a soil specimen of 70 mm diameter and 50 mm height was saturated by applying an effective stress of 10 kPa. Once the soil specimen was saturated, a hydraulic gradient was applied by creating a difference in pressure on the top and the bottom of the specimen. The pressure gradient initiated water flow from the bottom to the top of the specimen. The water was allowed to drain until a

steady-state water flow was achieved, after which the outflow was measured over an interval of time using a volume change indicator. The test was conducted under at least three different hydraulic gradients so that a relationship between the water flow rate and the hydraulic gradient could be established and the saturated coefficient of permeability of the soil could be determined.

4.2.2.2. Wet-dry cycle falling head permeameter test

In order to investigate the effect of cementation on the saturated coefficient permeability of RC, falling head permeability tests were conducted under different wet-dry cycles. The wet-dry cycles were used to induce cementation. The falling head permeability tests were conducted in accordance with ASTM D5084 (ASTM, 2002d). The falling head permeability tests were initially conducted on a saturated specimen. After the initial tests, the samples were dried by applying air pressure to the top of the falling head cell. An air pressure of 5 kPa was used to dry the sample. In order to check if the specimen was sufficiently dried, the falling head cell was weighed and checked against its initial (saturated) weight. The drying process was stopped when the volumetric water content change was less than 0.05. Following that, the specimen was re-wetted. The wetting process was conducted by inserting the falling head cell into a pail of water. Similarly, the weight of the cell was also recorded in order to check its saturation state. A similar method was used for the subsequent wet-dry cycle. Three (3) and six (6) wet-dry cycles were used to study the effect of cementation on the permeability of RC.

4.2.2.3. Tempe-cell test

Tempe-cell tests were performed to obtain the drying soil-water characteristic curve (SWCC). Tempe-cell tests were conducted using a Tempe pressure cell (Soilmoisture Equipment Corporation, USA). The Tempe-cell consisted of three main components, namely the top and base caps, a 60 mm high and 85 mm diameter brass cylinder and a 1 bar high-flow high-air entry ceramic disk. The

Tempe-cell tests were performed in accordance with ASTM D2325 (ASTM, 2002e).

Since the minimum air pressure supplied through the pressure gauge was only accurate to 5 kPa, the air pressure control system was modified. The modification used a burette that was filled with water and an additional air pressure releaser. By applying a small pressure through the air pressure regulator (Figure 4.1) and releasing air pressure by the additional air pressure releaser, a small air pressure could be applied and monitored by the water height inside the burette.

Figure 4.1 Schematic diagram of a Tempe-cell with air pressure modification (from Indrawan et al., 2006)

4.2.2.4 Capillary rise tube

The wetting SWCC was determined using the capillary rise tube experiment (Fredlund and Rahardjo, 1993). The capillary rise tube experiment was performed to obtain the wetting SWCC for FRC and FRA materials. A PVC tube with a 100 mm diameter and a 500 mm height was used. The soil was compacted to the specified density inside the tube. The tube was then placed in a tray with a constant water table. The bottom of the tube was covered with plastic mesh and filter cloth to

prevent the soil from falling off of the tube. The top of the tube was covered with plastic wrap and aluminum foil to prevent evaporation. After a sufficient time, equilibrium of the capillary water in the tube was reached. Soil specimens were taken from the sampler ports and the water contents were determined. The magnitude of the negative pore-water pressure of the sample was equal to the height of the tube where the soil specimen was taken. The SWCC was constructed by plotting the volumetric water content and the negative pore-water pressure of the soil specimens. The capillary rise tube is shown in Figure 4.2.

Figure 4.2 Capillary rise tube for wetting SWCC determination

4.2.2.5. Fitting curves for SWCC and permeability functions

Tempe-cell tests can only provide a limited set of SWCC data. In order to obtain a continuous and wide range of data, the experimental data were fitted with Fredlund & Xing's (1994) equation. The permeability function of each soil was predicted

using the method proposed by Fredlund et al. (1994). The fitting method and equation are explained in Chapter 3.

4.2.3. X-Ray Diffraction (XRD) testing for measuring cementation of RC

X-ray diffraction (XRD) is the most widely used method for the identification of soil minerals and the study of their crystal structure (Mitchell, 1995). Analysis of crystal structure is possible because X-ray wavelengths are in the same order as the spacing of atomic planes in crystalline material. Bragg's Law forms the basis for the identification of crystals using X-ray diffraction. Bragg's Law is as follows:

$$n\lambda = 2d \sin \Theta \quad (4.1)$$

where: n is the order reflection, λ is the wavelength (1.54 Å), Θ is the X-ray wave angle and d is the atomic plane spacing. Since each mineral has a different spacing of inter-atomic planes in three dimensions, the atomic plane spacing (d) can be used to identify the minerals in a soil. The theory on X-ray diffraction is explained in further detail in Mitchell (1993).

In this study, the XRD test was performed using a Bruker AXS D8 Advanced machine that strikes the material at a 2θ angular range of 5 to 80°. The samples were prepared in powder form (passed through a #200 sieve). Four samples were used for the XRD test: 1. Original RC without treatment, 2. RC after 28 days of saturation, 3. RC after three wet-dry cycles and 4. RC after six wet-dry cycles. The specific minerals observed as markers of cementation were calcite (CaCO_3), calcium disilicate (C_2S) and calcium trisilicate (C_3S).

4.2.4 Material hydrophobicity measurement

A goniometer (OCA 25, Dataphysics GmbH) is an instrument to measure the contact angle of water on a flat surface. A goniometer is shown in Figure 4.3. In this study, the contact angle was used to measure the degree of hydrophobicity of a material. The specimen used for the hydrophobicity measurement was Fine Recycled asphalt (FRA) that was retained on 0.425 mm sieve. The specimen was first compacted until it achieved a flat surface condition. A suspending drop of

water was then produced under the needle. When the water dropped down onto the specimen the profile was captured by the goniometer's camera for contact angle analysis. The time taken for a drop of water to be fully absorbed by the material was also recorded by the goniometer.

Figure 4.3 Goniometer

4.3. COLUMN STUDY

4.3.1 Column setup

The column study adopted the column setup used by Yang et al. (2004b), as shown in Figure 4.4. The column was made from acrylic with a 5mm thickness. The height and diameter of the acrylic column were 1000mm and 190mm, respectively. Several ports were drilled to accommodate the measurements. The cylinder was fixed on a base plate. The function of the base plate was to regulate the water flow in and out from the bottom of the column; in other words, the function of the base plate was to accommodate the boundary condition. A lamp was added to the column setup in order to accelerate evaporation. The lamp was positioned at the same height as the rainfall nozzles. The potential evaporation produced by the lamp was investigated by periodically measuring a beaker full of water under the same environmental conditions as where the column was located.

Figure 4.4 One-dimensional (1-D) column infiltration-evaporation setup

The water flow system of the column had three components: the inflow control, overflow discharge and percolation discharge. The inflow control consisted of a water storage tank, constant head water tank, flow regulator and a rainfall nozzle. The function of the storage tank was to provide a continuous supply of water to the constant head water tank. The constant head water tank ensured the pressures were similar for different rainfall nozzles. The nozzles were manufactured in-house. Different sizes of nozzle openings were manufactured in order to obtain different rainfall intensities. The nozzles were calibrated by measuring the amount of rainfall captured by a beaker over different times. In order to obtain uniform rainfall throughout the column, filter paper was placed on top of the soil column. The measurements were carried out in the center of the column.

The overflow discharge was provided by two outlets at the same level as the soil surface. The function of the overflow discharge was to discharge overflow water caused by ponding or upward infiltration during the experiments. The percolation discharge was provided by the base plate. The base plate had two valves: one led to a tank for percolation measurements and the other led to a tank filled with water to maintain the water table boundary condition.

In order to obtain a homogenous and uniform soil column, precise placement of the soil was performed. Dry tamping method was used for material compaction in this study. The dry density of the compacted materials was controlled by carefully laying materials in thin layers for a specified weight and volume. The procedure of the dry tamping method is as follow: firstly, the amount of soil for every lift was calculated. The soil was placed in a 100mm lift. For every lift, the soil was compacted using a proctor hammer 25 times from a height of 30 cm above the soil layer. Measuring devices were installed during compaction to ensure good contact. The measuring devices were a tensiometer-transducer system to measure pore-water pressure, a TDR system to measure volumetric water content and an automatic weighing balance. The positions of the tensiometers and TDRs in the column are shown in Figure 4.5. The number of tensiometers in fine-grained layer is more than that in the coarse-grained layer because the changes in pore-water pressure in the column are expected to occur in the fine-grained layer. As the pore-water pressure and the volumetric water content near the interface are critical to the analyses of the column study results, the tensiometer and TDR were positioned as close as possible to the interface. For evaporation analyses, changes in pore-water pressure near the surface of the column were expected. Therefore, it is also necessary to put a tensiometer as near as possible to the surface of the column. Details on the measuring devices (Tensiometers, TDR's and automatic weighing balance) are provided in following sections.

Figure 4.5 Positions of tensiometers and TDR instruments in the 1-D laboratory column study

Tensiometers are equipment used for negative pore-water pressure measurement. In this study, tensiometers model 2100F (Soilmoisture Equipment Corporation, USA) were used. The tensiometer consists of four main parts: high air-entry ceramic tip, tensiometer shaft, the pressure transducer and the jet-fill reservoir cup. The schematic diagram of the tensiometer used in this study is shown in Figure 4.6. The high air-entry ceramic tip used in this study has an air-entry value of 1 bar or 100 kPa. The pressure transducer type was GT3-15 which was manufactured by ICT international. The pressure transducer was capable to measure pressure from the range of -100 to 100 kPa. The pressure transducer was then connected to the ICT data logger and the data are transmitted using a wireless radio system to the computer. Figure 4.7 showed the actual tensiometer with pressure transducer and data logger.

Figure 4.6 Schematic diagram of the jet fill tensiometer type 2100F
(SoilmoistureEquipment Corporation, USA)

Figure 4.7 Tensiometer-transducer system with data logger

The pressure transducer was calibrated using the pressure calibrator (Druck DPI 610) shown in Figure 4.8. The pressure from the applicator and the voltage output were plotted and fitted with a linear regression line in order to obtain the correlation equation. The equation was then used in the data acquisition system. As expected, all pressure transducers had a linear correlation between the output voltage and the pore-water pressure with an R^2 value of 1 or very close to 1. The complete tensiometer-transducer system was calibrated one more time in order to determine the response time. Because the expected monitored pressure was in the range of 0-10 kPa, the calibration was conducted using a 1 m ruler (0-10 kPa). The tensiometer tip was submerged in water inside a container. The distance between the tensiometer tip and the pressure transducer was the amount of pore-water pressure exerted into the tensiometer-transducer system. If the tensiometer tip was positioned at a specified depth above the pressure transducer, a negative pore-water pressure equals to the distance between the tensiometer tip and the pressure transducer was applied. The change in pore-water along with the change in this distance was recorded. A typical graph showing the pore-water pressure readings and response times is shown in Figure 4.9.

Figure 4.8 Pressure transducer calibrator

Figure 4.9 Pore-water pressure against time for different height differentials.

Time Domain Reflectometry (TDR) was used to determine the volumetric water content of the soil. The TDR system used in this study was manufactured by the Soilmoisture Equipment Corporation, USA. The system includes a TRASE box pulse generator BE 6050X2, a 16 channel multiplexer enclosure 6020B05 and 8cm buriable waveguides. The measurements were recorded using Wintrase ver.2.0 software. The TRASE system used in this study is shown in Figure 4.10.

TDR determines the velocity of the electromagnetic pulse of energy transmitted into the soil through the transmission line, which consists of a 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 the dielectric constant, the slower the velocity. In general, soil is composed of air, mineral and organic particles and liquid water which have dielectric constants, K , of 1, between 2 and 4 and 80, respectively. Due to the great difference between the dielectric constant of water and other soil components, the travel velocity of the microwave pulse is highly dependent on the water content of the soil. By knowing the relationship of

the K_a value to the volumetric water percentage, which 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 then be converted to the volumetric water content of the soil.

Figure 4.10 The complete TRASE system: TRASE pulse generator (left), multiplexer enclosure(middle) and buriable waveguide (right)

An electronic weighing balance was used to measure the mass of overflowed and percolated water in the infiltration column tests. The electronic weighing balance manufactured by UWE model PS-12K with a capacity of 12 kg and 1 g resolution shown in Figure 4.11 was used in this study. A digital indicator manufactured by Kubota model KL-D1000 was connected to each weighing balance. The digital indicator was then connected to a personal computer. A program developed by Krisdani (2007) was used to setup the interval and duration of the data acquisition. The weighing balance was calibrated using a standard metric weight model no. TW-5000-1 manufactured by Troemner, PA, USA.

Figure 4.11 Weighing balance and digital indicator for percolation measurements

4.3.2. Experimental program

The experimental program is presented in two section. Section 4.3.2.1 describes the preliminary numerical modeling program followed by the column test experimental program in Section 4.3.2.2.

4.3.2. 1 Preliminary numerical modeling

Several combinations of recycled materials can be used as the materials in a capillary barrier. The objective of the preliminary numerical modeling was to narrow down the combinations in order to search for the best combination of materials to use in the laboratory column study. The preliminary numerical modeling was carried out using SVFLUX (Soilvision Ltd). The geometry of the numerical model was a 1-D model with a total thickness of 1m. The boundary condition at the bottom was a pressure head equal to -4 m with the bottom of the model acting as the datum. The bottom boundary condition of -4 m was chosen to represent the typical groundwater level in the field. The top boundary condition was precipitation of 92mm/hr for 1 hr, which is a 10 year return period rainfall based on PUB design code. There were 11 combinations of soil cover used in the preliminary

numerical modeling, as shown in Figure 4.12. Pore-water pressure, volumetric water content and net percolations were recorded for each time step.

Figure 4.12 Cover combinations for preliminary numerical modeling

4.3.2.2 Experimental plan

Three column models were investigated in this study. The first column was constructed using 0.5 m fine recycled asphalt pavement (FRA) overlying 0.5 m coarse recycled asphalt pavement (CRA) (test name CM_FRA_CRA). The second column was constructed to observe the effect of using different materials. The second column material combination was 0.5m FRC overlying 0.5 m CRC (test name CM_FRC_CRC). The third column was constructed by stacking two capillary barriers constructed using 0.25m FRA overlying 0.25m CRA (test name CM_DCB_FRA_CRA). The schematic diagrams of the soil column models are presented in Figure 4.13.

Four types of tests were performed on the column model, namely an upward infiltration (UI) test, a rapid drawdown (DD) test, an infiltration (RF) test and an evaporation (EV) test. The upward infiltration test was conducted by applying a water pressure of 1.52m to the soil column from the base plate. The objectives of the upward infiltration test were to saturate the soil column (removing the air) and, at the same time, to measure the coefficient of permeability of the soil layers. The rapid drawdown test was conducted by lowering the water table obtained from the

upward infiltration test (at the top of the column) to the base of the column. The drawdown test was conducted to investigate the development of negative pore-water pressure in the soil column. Rainfall tests were started after the pore-water pressure in the soil column stabilized or became as close as possible to the hydrostatic condition. The rainfall intensities and durations were then chosen. The first rainfall intensity and duration was 4 mm/h for 6 h. This intensity and duration of rainfall was selected to simulate small rainfall with long duration. The second rainfall had an intensity of 8 mm/h for 6 h, to simulate a medium intensity rainfall with long duration. The third rainfall was 50 mm/h for 1 hr, to simulate a high intensity rainfall with short durations. The total rainfall amount of the third rainfall was similar to the second rainfall. The response of the soil column under different rainfall intensity and durations are of interest of the different intensities and durations of the rainfall. The rainfall intensities were not derived from the intensity-duration-frequency curve due to limitation on the rainfall simulating system. The evaporation test was carried out using a lamp after the rainfall tests. The experimental plan for the column study is summarized in Table 4.1.

Figure 4.13 Schematic diagram of the 1-D infiltration-evaporation column

Table 4.1 Experimental program for the 1-D column study

Column No.	Test	Infiltration		Evaporation		Column No.	Test	Infiltration		Evaporation	
		Intensity (mm/hr)	duration (hr)	Mode	duration (hr)			Intensity (mm/hr)	duration (hr)	Mode	duration (hr)
1	CM_FRA_CRA_UI ¹	-	-	-	-	2	CM_FRC_CRC_UI	-	-	-	-
	CM_FRA_CRA_DD ²	-	-	-	-		CM_FRC_CRC_DD	-	-	-	-
	CM_FRA_CRA_RF ³ 1	4	6	-	-		CM_FRC_CRC_RF1	4	6	-	-
	CM_FRA_CRA_RF2	8	6	-	-		CM_FRC_CRC_RF2	8	6	-	-
	CM_FRA_CRA_RF3	50	1	-	-		CM_FRC_CRC_RF3	50	1	-	-
	CM_FRA_CRA_RF4	4	6	-	-	3	CM_DCB_FRA_CRA_UI	-	-	-	-
	CM_FRA_CRA_EV ⁴ 1	-	-	Lamp	48		CM_DCB_FRA_CRA_DD	-	-	-	-
	CM_FRA_CRA_RF5	8	6	-	-		CM_DCB_FRA_CRA_RF1	4	6	-	-
	CM_FRA_CRA_EV2	-	-	Lamp	48		CM_DCB_FRA_CRA_RF2	8	6	-	-
	CM_FRA_CRA_RF6	50	1				CM_DCB_FRA_CRA_RF3	50	1	-	-
	CM_FRA_CRA_EV3	-	-	Lamp	48						

¹ UI= Upward Infiltration² DD= Drawdown³ RF= Rainfall⁴ EV= Evaporation

4.4. INFILTRATION BOX STUDY

4.4.1 Description of the infiltration box

The infiltration box was designed to investigate the diversion capacity of the capillary barriers in a controlled environment. The apparatus was instrumented in order to capture the process of water flow in the capillary barrier system. The general setup of the model is presented in Figure 4.14. The main components of the physical model are the infiltration box, the rainfall simulator, the measuring devices and the water collection system. The box dimensions were 2.0 m in length, 0.25 m in width and 1 m in height.

Figure 4.14 General schematic of the infiltration box

The infiltration box was fabricated in two parts: the acrylic box and the steel frame. Acrylic was chosen as the material for the box in order to be able to view the flow of water in the capillary barriers. The acrylic was 15 mm thick. The acrylic box consisted of five acrylic parts mounted together and fixed in place with epoxy glue and a silicon membrane. After the acrylic box was built, it was filled with water in order to test for leakage. The acrylic box was then drilled to create holes for the connection of the measuring devices, water drainage and for soil extraction. The position and the naming of the connections are shown in Figure 4.15.

After the holes for the measuring device connectors were made, the box was hoisted up and positioned inside the stainless steel frame. The stainless steel frame was made to fit the acrylic box. The inner dimensions of the stainless steel frame were the outer dimensions of the acrylic box. In order to avoid friction between the stainless steel frame and the acrylic box, rubber pads were placed inside the stainless steel frame. After the acrylic frame was seated nicely inside the stainless steel frame, 16 bolts were drilled to ensure that the box and the frame were rigid. The stainless steel frame sat on a table measuring 2100 mm x 500 mm x 500 mm made from a hollow 65 x 65 mm stainless steel bar. The table was equipped with a jacking system that could lift one side of the stainless steel frame (high side) up to a height of 550 mm, which is equivalent to a 15° inclination angle. A 3-D drawing of the box and a picture of the box are shown in Figure 4.16 and Figure 4.17, respectively.

Figure 4.15 Position of instrumentation in the infiltration box

Eight 25 mm holes were drilled for water drainage from the lower side of the infiltration box. The water was drained using a 3/8 inch flexible Polyurethane tube to the water collection system. On the other side of the acrylic wall (high side), a

soil discharge door was built with a size of 150 x 150 mm. The soil discharge door was meant to help with the process of soil removal from the infiltration box when the experiment was completed. The soil discharge door was made using stainless steel.

Two types of measuring devices were inserted into the infiltration box: tensiometers for pore-water pressure measurement and moisture sensors for volumetric water content measurement. The details of the measuring devices are provided in the following sections. In order to insert the devices into the infiltration box, custom connections were manufactured. The connection used for the tensiometers was similar to the one described by Tami (2004a), whereas a customized connection for the moisture sensors was designed and built in this study. The connection is shown in Figure 4.18.

Figure 4.16 3-D infiltration box drawings

Figure 4.17 Perspective view of the infiltration box

Figure 4.18 Custom connection for moisture sensor insertion to the infiltration box

A rainfall simulator was placed on top of the infiltration box. The rainfall simulator consisted of the rainfall nozzles, a high pressure tube and a high pressure pump (XM-M08.10N, Annovi Reverberri). The rainfall nozzle had an aperture of 0.15 mm and was capable of spraying up to 127 l/min with 100 bar pressure. The rainfall nozzle produced droplets of 50 to 200 microns, depending on the pressure used. The nozzles were connected to the misting tee or elbow to the high pressure water line. A 3/8 inch high pressure polyethylene tube was used to deliver the water from the high pressure pump to the nozzle. The high pressure pump used in this study could discharge a pressure of up to 100 bar. The high pressure pump was equipped with a pressure dial indicator and a pressure adjuster. The position of the nozzle and the arrangement of the rainfall simulator are shown in Figure 4.19.

The rainfall simulator was calibrated by measuring the amount of water from 20 measuring glasses positioned inside the infiltration box. Each glass had a diameter of 8.6 cm. A various intensities of rainfall were applied for a period of 1 hour by changing the pressure from the pump. In order to measure the uniformity of the simulated rainfall, the Christiansen uniformity coefficient (CUC)(Hall et al., 1989) was calculated for each rainfall intensity. Rainfall intensities from 6-35 mm/hr were obtained from the rainfall simulator. The calculated CUC's were between 57% and 78%. CUC values were higher for higher intensity rainfall than lower intensity rainfall. This was because when the pump pressure increased, the coverage area of each rainfall nozzle increased; therefore, the uniformity of the rainfall was better. The CUC range obtained in this study was similar to the range reported in previous studies that used different rainfall simulator arrangements (e.g., Tami et al., 2004a; Krisdani, 2007). A contour plot for a rainfall intensity of 19.7 mm/hr is shown in Figure 4.20. The contour plot was generated from twenty (20) measurement points. The rainfall intensity captured in the middle of the box was around 25 mm/hr, the rainfall intensity was generally higher in the middle of the box. This could be caused by the fact that the mainline was connected to the rainfall simulator line in the middle of the box; hence, the pressure in the middle of the box was higher. It was also found that, at the corner of the box, the rainfall intensity was higher on one side compared to the other. This could be caused by non-vertical installation of the nozzles at the end of rainfall simulator.

Figure 4.19 Position of rainfall nozzle on the infiltration box

Figure 4.20 Contour of rainfall uniformity for rainfall intensity 27mm/hr

4.4.2 Measuring devices

The majority of the measuring devices used in the infiltration box study were similar to those used in the column study described in Section 4.3.2. However, a different sensor type was used to measure changes in the volumetric water content: standing wave moisture (SWM) sensors (MP306, ICT International Corp). The moisture sensors were cylindrical in shape with a diameter of 32mm and a length of 140 mm. There were three 60 mm needles arranged in a single plane. The moisture sensors were connected to a data logger (Soil Moisture Meter, SMM, ICT International Corp). The SMM was similar to the STM for the pore-water pressure

measurements, except that the SMM could support up to 10 moisture sensors. The data from the SMM were also transmitted via the wireless connection using the ICT MCC to the computer. A picture of the moisture sensor is shown in Figure 4.21.

Time domain reflectometry (TDR) (Topp et al., 1980) is a popular method of measuring the volumetric water content of soil. TDR can measure the propagation velocity of an electromagnetic pulse generated by a pulse generator along a transmission line and convert the measurement to a calibrated water content value. However, a relatively new method, the standing wave method (SWM) (Gaskin and Miller, 1996), is increasingly being used to measure soil moisture content. The principle of SWM is to employ an impedance transform of the transmission line to determine the impedance of the probe. The SWM uses an oscillator to generate an electric field from which the difference in impedance at the probe and the transmission cable can be calculated. This difference is related to the dielectric properties of the material for a specified frequency generated by the oscillator. The dielectric properties of the material are related to the moisture content of the material. A study by Sun et al. (1990) comparing the TDR and SWM methods showed that the difference in moisture content measured by the two methods was very small, whereas the advantage of the SWM method over the TDR method is that it is relatively cheaper and more portable as it does not require a pulse generator.

Figure 4.21 Soil moisture sensor MP 306 (ICT international Pty Ltd)

4.4.3 Material and measuring device installation

The materials used for the infiltration box were fine and coarse recycled asphalt (FRA and CRA). These materials were chosen because they satisfy criteria for capillary barrier materials as obtained from the preliminary numerical modeling (i.e. low-water entry value of coarse-grained layer and high water storage of fine-grained layer). Besides the recycled asphalts, two other materials were used as the base of the capillary barrier: graded filter sand (GFS) and a geosynthetic material (Polyfelt TS, Tencate Geosynthetic). These materials were placed below the capillary barrier to simulate the boundary condition and also to avoid contact between the sharp CRA and the acrylic box glass. The properties of the filter sand and the geosynthetic material can be found in Tami et al. (2004a) and Krisdani et al. (2007), respectively.

A dry-tamping method was used to compact the material inside the infiltration box. Firstly, the amount of dry material required was calculated based on the planned dry density. The material was air-dried (gravimetric moisture content of 1 to 2%) before placement. Afterwards, the materials were placed layer by layer with a thickness of 50 mm. A circular compactor with a 10 cm diameter was used to compact the materials inside the infiltration box. The circular plate was dropped from a 100 mm height using an air pressured compactor. The dry densities for GFS, CRA and FRA were 1.5, 1.47 and 1.53 Mg/m³, respectively. After the compaction of 150 mm of GFS, a layer of woven geotextile was placed above the GFS. This was followed by the compaction of the single capillary barrier (SCB) or the dual capillary barrier (DCB). In order to avoid the formation of preferential flow between the acrylic walls and the materials, a layer of thin grease was applied to the acrylic wall using a cloth. The small tip tensiometers and the moisture sensors were also placed during the placement of the materials in order to ensure good contact between the materials and the measurement sensors.

4.4.3 Experimental program

A series of experiments was conducted to study the diversion capacity of capillary barriers. Three physical models were investigated in this study. The first physical

model was a capillary barrier that was constructed using 0.15 m thick FRA overlying 0.15 m thick CRA (BM_SCB15). The second physical model was a capillary barrier that was constructed using 0.3 m thick FRA overlying 0.15 m thick CRA (BM_SCB30). The last physical model was a dual capillary barrier that was constructed by stacking two capillary barriers constructed of 0.15m thick FRA overlying 0.15 m thick CRA (BM_DCB). The first and second capillary barrier models were constructed to observe the diversion capacity of the capillary barriers and the parameters that affect the performance of capillary barriers constructed using recycled asphalt. The dual capillary barrier model was constructed in order to investigate its lateral diversion capacity performance compared to the single capillary barrier (SCB). Illustrations of these three physical models are presented in Figure 4.22.

Each test was initialized by saturation of the physical model. Saturation was performed by sprinkling the physical model with water and closing all of the drainage valves. When slight ponding was observed, the infiltration box was tilted to a specified angle and the drainage valves were opened. This part of the test was similar to the drawdown test in the laboratory column study. The drainage continued until the pore-water pressures in the infiltration box were stable. The objective of the saturation was to remove any trapped air in the box, whereas the objective of the drainage process was to set an initial condition for the physical model. Once the pore-water pressure readings in the infiltration box were stable, the infiltration test was started. In this study, two rainfall intensities—10 and 20 mm/hr—were selected. The duration for the two rainfalls was one hour.

The slope angle of the infiltration box was varied for each physical model. Physical models BM_SCB15 and BM_DCB were tested under 2 and 5° slope angles, whereas model BM_SCB30 was tested under 2, 5 and 7° slope angles. The mild slope angles were chosen to represent the mild slope found in landfill surface cover. The effects of the slope angle on the performance of the single and double capillary barriers were investigated for each physical model. The performance of each physical model was assessed by measuring the pore-water pressure, volumetric water content and water drainage. The experimental program is summarized in Table 4.2.

Figure 4.22 Three physical models constructed during the study

Table 4.2 Summary of experimental program of the infiltration box

BoxNo.	Test	Slope angle	Infiltration		BoxNo.	Test	Slope angle	Infiltration	
			Intensity (mm/hr)	duration (hr)				Intensity (mm/hr)	duration (hr)
1	BM_SCB15_Sat	0	-	-	2	BM_SCB30_5Deg_RF1	5	10	1
	BM_SCB15_DD1	2	-	-		BM_SCB30_5Deg_RF2	5	20	1
	BM_SCB15_2Deg_RF1	2	10	1		BM_SCB30_DD3	7	-	-
	BM_SCB15_2Deg_RF2	2	20	1		BM_SCB30_7Deg_RF1	7	10	1
	BM_SCB15_DD2	5	-	-		BM_SCB30_7Deg_RF2	7	20	1
	BM_SCB15_5Deg_RF1	5	10	1	3	BM_DCB_Sat	0	-	-
	BM_SCB15_5Deg_RF2	5	20	1		BM_DCB_DD1	2	-	-
2	BM_SCB30_Sat	0	-	-		BM_DCB_2Deg_RF1	2	10	1
	BM_SCB30_DD1	2	-	-		BM_DCB_2Deg_RF2	2	20	1
	BM_SCB30_2Deg_RF1	2	10	1		BM_DCB_DD2	5	-	-
	BM_SCB30_2Deg_RF2	2	20	1		BM_DCB_5Deg_RF1	5	10	1
	BM_SCB30_DD2	5				BM_DCB_5Deg_RF2	5	20	1

4.5 NUMERICAL MODELING

The numerical modeling in this study was performed using SVFlux (Soilvision International Ltd). Two types of numerical modeling were conducted. The first type was the verification of the box and column study results. The study compared the results of the laboratory column and box data to the results of the numerical modeling. The material properties used were similar to the material properties in the laboratory study. The pore-water pressure and the volumetric water content profiles from the laboratory study and numerical modeling were then compared. The second type of numerical modeling was the parametric study. In the parametric study, several governing parameters were varied. The effects of each parameter on the performance of SCB and DCB were then investigated

4.5.1 Numerical modeling of column test

The geometry of the numerical models was similar to the geometry of the laboratory column study. The boundary conditions of the numerical models were selected based on the laboratory column study. During the drawdown test, the water table was initially positioned at the top of the column, thereby creating a hydrostatic initial condition, and the applied bottom boundary condition was no flow. Subsequently, the boundary condition at the bottom was changed to free flow and no boundary condition was applied at the top of the column. During the rainfall and evaporation tests, the boundary condition at the top of the column was a flux boundary (precipitation/evaporation) and the boundary condition at the bottom of the column was a pressure head equal to zero. The geometry and boundary conditions of the numerical model are illustrated in Figure 4.23. The material properties used in the numerical modeling were properties of the material used in the actual infiltration test. The material properties are presented in Chapter 5.

Figure 4.23 Geometry illustration of the 1-D verification numerical model

4.5.2 Numerical modeling for box test

Similar to the column verification, the geometry of the numerical model for the box verification was the same as the geometry of laboratory models. The geometry and boundary conditions of the box numerical modeling are illustrated in Figure 4.24. The sides and the bottom of the numerical model were modeled as zero flux except at the drainage holes locations. The drainage holes locations were modeled as a pressure head equal to zero. During the drawdown test, the initial condition of the model was assumed to be a total head equal to zero. For the rainfall and subsequent tests, the initial condition was the final condition of the previous test based on the experimental plan. The material properties used for the numerical modeling were similar to the material properties used in the actual infiltration box. The material properties are presented in Chapter 5. A more detailed explanation on numerical modeling for box test is described in Appendix A.

Figure 4.24 Geometry and boundary conditions of the infiltration box study

4.5.3 Parametric study

By utilizing the verified numerical model, parametric studies were conducted to investigate the effect of different parameters on the performance of the capillary barrier in both the 1-D and 2-D conditions. The parametric studies focused on the effect of layer thickness in the single capillary barrier and the dual capillary barrier. Parametric studies were conducted for the 1-D and 2-D problems in order to determine the relationship between water storage and lateral diversion of the capillary barrier. The parameters observed during the parametric studies were water storage capacity for the column study and lateral diversion capacity for the box study.

4.5.3.1 Effect of fine and coarse-grained layer thickness on SCB performance

The effect of fine and coarse-grained layer thickness on the performance of SCB was investigated by varying the thickness of each layer. The performance of SCB in the 1-D and 2-D conditions was investigated. Parameters observed during the study were water storage capacity and lateral diversion capacity for the 1-D and

2-D conditions, respectively. For the 1-D condition, the numerical model setup followed the CM_FRA_CRA test of the laboratory column study. The fine-grained layer thickness was varied from 0.25 to 2 m, while the coarse layer thickness was fixed at 0.5 m during the investigation of the effect of fine-grained layer thickness. For the investigation of the effect of the coarse-grained layer thickness, the coarse-grained layer thicknesses were varied from 0.1 to 1 m, while the fine-grained layer thickness was fixed at 0.5 m. The rainfall intensity and water table were fixed at 92 mm/hr and 1 m, respectively. The investigation of the effect of fine and coarse-grained layer thicknesses in the 1-D condition created eight cases which are summarized in Table 4.3.

For the 2-D condition, the numerical setup followed the one used to verify the BM_SCB30_5deg test. The fine-grained layer thicknesses were varied from 0.1 to 1 m, while the coarse-grained layer thickness was fixed at 0.15 m. In order to investigate the effect of coarse-grained layer thickness, the coarse-grained layer thicknesses were varied from 0.1 to 1 m, while the fine-grained layer thickness was fixed at 0.3 m. Similar to the 1-D condition, the rainfall intensity and water table were fixed at 92 mm/hr and 1 m, respectively. The parametric study cases for the 2-D condition are summarized in Table 4.4.

Table 4.3 Parametric study cases on the effect of SCB layer thickness in the 1-D condition

Case No.	Case Name	Cover type	Rainfall Intensity (mm/hr)	Water table (m)	Fine layer thickness (m)	Coarse layer thickness (m)
1	C_FT_1	CM_FRA_CRA	92	1	0.25	0.5
2	C_FT_2	CM_FRA_CRA	92	1	0.5	0.5
3	C_FT_3	CM_FRA_CRA	92	1	1	0.5
4	C_FT_4	CM_FRA_CRA	92	1	2	0.5
5	C_CT_1	CM_FRA_CRA	92	1	0.5	0.1
6	C_CT_2	CM_FRA_CRA	92	1	0.5	0.25
7	C_CT_3	CM_FRA_CRA	92	1	0.5	0.5
8	C_CT_4	CM_FRA_CRA	92	1	0.5	1

Table 4.4 Parametric study cases on the effect of SCB layer thickness in the 2-D condition

Case No.	Case Name	Cover type	Rainfall Intensity (mm/hr)	Water table	Slope angle	Fine layer thickness (m)	Coarse layer thickness (m)
9	B_FT_1	BM_SCB30_5deg	92	1	5	0.1	0.15
10	B_FT_2	BM_SCB30_5deg	92	1	5	0.25	0.15
11	B_FT_3	BM_SCB30_5deg	92	1	5	0.3	0.15
12	B_FT_4	BM_SCB30_5deg	92	1	5	0.5	0.15
13	B_FT_5	BM_SCB30_5deg	92	1	5	1	0.15
14	B_CT_1	BM_SCB30_5deg	92	1	5	0.3	0.1
15	B_CT_2	BM_SCB30_5deg	92	1	5	0.3	0.15
16	B_CT_3	BM_SCB30_5deg	92	1	5	0.3	0.25
17	B_CT_4	BM_SCB30_5deg	92	1	5	0.3	0.5
18	B_CT_5	BM_SCB30_5deg	92	1	5	0.3	1

In addition to the above parametric study, the effect of model length on SCB performance was also investigated. A model of BM_SCB30_5deg with a fine layer thickness of 0.5 m (B_FT_4) was chosen as the base model. The length of this model was then varied to 3, 5 and 10 m.

4.5.3.2 Effect of fine and coarse-grained layer thickness on DCB performance.

The effect of the layers in DCB was investigated by varying the layer thicknesses. Similar to the previous section, the study was performed for the 1-D and 2-D conditions. The parametric study followed the numerical setup from the CM_DCB_FRA_CRA and BM_DCB tests for the 1-D and 2-D conditions, respectively. For the 1-D condition, the thicknesses of the investigated layers were varied from 0.1 to 1 m, while the other layer was fixed at 0.25 m. For the 2-D condition, the thickness of the investigated layer was varied from 0.1 to 1 m, while the other layer was fixed at 0.15 m. The parametric study cases for the effect of fine and coarse-grained layer thicknesses on DCB performance are summarized in Table 4.5 and Table 4.6 for the 1-D and 2-D conditions, respectively.

Table 4.5 Parametric study cases on the effect of DCB layer thickness in the 1-D condition

Case No.	Case Name	Cover type	Rainfall intensity (mm/hr)	Water table (m)	Upper fine layer thickness (m)	Upper coarse layer thickness (m)	Lower fine layer thickness (m)	Lower coarse layer thickness (m)
19	C_UFL_1	DCB_FRA_CRA	92	1	0.1	0.25	0.25	0.25
20	C_UFL_2	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.25
21	C_UFL_3	DCB_FRA_CRA	92	1	0.5	0.25	0.25	0.25
22	C_UFL_4	DCB_FRA_CRA	92	1	1	0.25	0.25	0.25
23	C_UCL_1	DCB_FRA_CRA	92	1	0.25	0.1	0.25	0.25
24	C_UCL_2	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.25
25	C_UCL_3	DCB_FRA_CRA	92	1	0.25	0.5	0.25	0.25
26	C_UCL_4	DCB_FRA_CRA	92	1	0.25	1	0.25	0.25
27	C_BFL_1	DCB_FRA_CRA	92	1	0.25	0.25	0.1	0.25
28	C_BFL_2	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.25
29	C_BFL_3	DCB_FRA_CRA	92	1	0.25	0.25	0.5	0.25
30	C_BFL_4	DCB_FRA_CRA	92	1	0.25	0.25	1	0.25
31	C_BCL_1	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.1
32	C_BCL_2	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.25
33	C_BCL_3	DCB_FRA_CRA	92	1	0.25	0.25	0.25	0.5
34	C_BCL_4	DCB_FRA_CRA	92	1	0.25	0.25	0.25	1

Table 4.6 Parametric study cases on the effect of DCB layer thickness in the 2-D condition

Case No.	Case Name	Cover type	Rainfall Intensity (mm/hr)	Water table (m)	Slope angle	Upper Fine layer thickness (m)	Upper Coarse layer thickness (m)	Bottom Fine layer thickness (m)	Bottom Coarse layer thickness (m)
35	B_UFL_1	DCB_FRA_CRA	92	1	5	0.1	0.15	0.15	0.15
36	B_UFL_2	DCB_FRA_CRA	92	1	5	0.15	0.15	0.15	0.15
37	B_UFL_3	DCB_FRA_CRA	92	1	5	0.25	0.15	0.15	0.15
38	B_UFL_4	DCB_FRA_CRA	92	1	5	0.5	0.15	0.15	0.15
39	B_UFL_5	DCB_FRA_CRA	92	1	5	1	0.15	0.15	0.15
40	B_UCL_1	DCB_FRA_CRA	92	1	5	0.15	0.1	0.15	0.15
41	B_UCL_2	DCB_FRA_CRA	92	1	5	0.15	0.25	0.15	0.15
42	B_UCL_3	DCB_FRA_CRA	92	1	5	0.15	0.5	0.15	0.15
43	B_UCL_4	DCB_FRA_CRA	92	1	5	0.15	1	0.15	0.15
44	B_BFL_1	DCB_FRA_CRA	92	1	5	0.15	0.15	0.1	0.15
45	B_BFL_2	DCB_FRA_CRA	92	1	5	0.15	0.15	0.25	0.15
46	B_BFL_3	DCB_FRA_CRA	92	1	5	0.15	0.15	0.5	0.15
47	B_BFL_4	DCB_FRA_CRA	92	1	5	0.15	0.15	1	0.15
48	B_BCL_1	DCB_FRA_CRA	92	1	5	0.15	0.15	0.15	0.1
49	B_BCL_2	DCB_FRA_CRA	92	1	5	0.15	0.15	0.15	0.25
50	C_BCL_3	DCB_FRA_CRA	92	1	5	0.15	0.15	0.15	0.5
51	C_BCL_4	DCB_FRA_CRA	92	1	5	0.15	0.15	0.15	1

4.6 CONCLUDING REMARKS

The research program of this study is explained in details in this chapter. It started with the methods of investigation of recycled material properties. The recycled material properties especially those related to water flow in unsaturated soil are important in this study. The performance of a capillary barrier in 1-D conditions is investigated using 1-D column tests. The setup, measuring devices and experimental plan of the 1-D column test are described in this chapter. Subsequently, the methods used for the 2-D infiltration box are presented. In order to verify and expand the result of the laboratory test results, numerical modeling was performed. The methods used in the numerical modeling is demonstrated in this chapter.

CHAPTER 5

PRESENTATION OF RESULTS

5.1. INTRODUCTION

The experimental and numerical modeling results are presented in this chapter. The properties of the materials used in this study are presented in Section 5.2. In Section 5.3, the results obtained from the column study are presented. These results include the preliminary numerical modeling and the column study experimental results. Next, the results of the 2-D box study are presented in Section 5.4. Lastly, the results of the numerical modeling and parametric study are presented in Section 5.5.

5.2. MATERIAL PROPERTIES

5.2.1. Basic properties

There were four materials used in this study: fine and coarse recycled concrete aggregates and fine and coarse recycled asphalt pavement. The materials were obtained from a recycling plant operated by Samwoh Pte Ltd. Grain size distributions of the materials used in this study are presented in Figure 5.1. The grain size distribution results showed that fine recycled concrete (FRC) and fine recycled asphalt (FRA) can be categorized as well-graded sand. FRC and FRA have fines content of around 4% and gravel content of around 10%. Coarse recycled concrete (CRC) and coarse recycled asphalt (CRA) can be categorized as poorly (uniform) graded gravel. CRA does not have any sand content, whereas CRC has around 5% sand or fines. The grain size distribution variables, along with the basic properties of the materials, are summarized in Table 5.1.

Figure 5.1 Grain-size distributions of the materials used in this study

Table 5.1 Basic properties of the materials used in this study

Properties	Soils			
	FRC	CRC	FRA	CRA
1. Unified Soil Classification System				
Group symbol	SW	GP	SW	GP
Group name	Well-graded sands	Poorly graded gravel	Well-graded sands	Poorly graded gravel
2. Specific gravity, G_s				
	2.56	2.62	2.43	2.41
3. Grain-size distribution				
D_{60} (mm)	1.66	13.62	1.98	13.02
D_{30} (mm)	0.63	10.24	0.95	10.77
D_{10} (mm)	0.10	6.93	0.32	8.56
Coefficient of uniformity, C_u	16.60	1.97	6.19	1.52
Coefficient of curvature, C_c	2.39	1.11	1.42	1.04
Gravel content ($> 4.75\text{mm}$) (%)	10.60	90.39	9.50	95.90
Fines content ($< 0.075\text{mm}$) (%)	4.65	4.50	4.70	0.00
5. Relative density test results				
Max. dry density, $r_{d\max}$ (Mg/m^3)	1.99	1.89	1.92	1.84
Min. dry density, $\rho_{d\min}$ (Mg/m^3)	1.26	1.25	1.3	1.46
Maximum void ratio, $e_{\max}$	1.03	1.10	0.87	0.77
Minimum void ratio, $e_{\min}$	0.29	0.39	0.26	0.44
6. Saturated permeability, k_s (m/s)				
	1×10^{-4}	1×10^{-1}	5×10^{-3}	5×10^{-1}
7. Dry density for test ($0.8 \times \text{max dry density}$)				
	1.592	1.512	1.536	1.472

The effect of cementation on the saturated permeability of FRC was investigated using a falling head saturated permeability test. Three tests were conducted in order to obtain the coefficient of permeability after 0, 3 and 6 wet-dry cycles. The results of the falling head permeability test are shown in Table 5.2. The saturated permeability of the RCA was considered to be constant at a value of around 1×10^{-4} m/s.

Table 5.2 Results of wet-dry cycle permeability test on FRC

Wet-Dry cycle	FRC Saturated Coefficient of Permeability (m/s)
0	9.91×10^{-5}
3	1.66×10^{-4}
6	1.36×10^{-4}

5.2.2. Water characteristic curve

Drying water characteristic curves (WCC) of the materials obtained from Tempe-cell tests and fitted using Fredlund and Xing's (1994) equation are presented in Figure 5.2. Fitting parameters of the drying water characteristic curves are summarized in Figure 5.3. The air-entry values of FRC and FRA were higher than those of CRC and CRA, while the residual volumetric water contents of FRC and CRC were higher than those of FRA and CRA.

The wetting and drying water characteristic curves of FRC and FRA as obtained from the Capillary rise tube tests and Tempe-cell tests, along with Fredlund and Xing's (1994) best fitting curves, are presented in Figure 5.3. The FRC and FRA drying and wetting WCC best fit parameters are presented in Table 5.4. The results showed that the saturated volumetric water contents obtained from the drying and wetting processes were quite similar. The results also showed that the air-entry values obtained from the wetting curves were lower than the values obtained from the drying curves.

Figure 5.2 Water characteristic curve of RC and RA

Table 5.3 Parameters of the drying water characteristic curve

No	Description	Symbol	Soil			
			FRC	FRA	CRC	CRA
1	Saturated volumetric water content	θ_s	0.38	0.34	0.45	0.38
2	Residual volumetric water content	θ_r	0.146	0.106	0.126	0.101
3	Air-entry value	ψ_a	4.35	0.79	0.25	0.02
4	Fitting parameter	a	15.36	2.62	0.33	0.04
5		n	0.95	1.01	4.71	3.16
6		m	0.83	0.95	0.69	0.82
7		ψ_r (kPa)	331	55	0.89	0.19
8	Saturated permeability (m/s)	k_s	1×10^{-4}	5×10^{-3}	1×10^{-1}	5×10^{-1}

Figure 5.3 Drying and wetting characteristic curves of FRC and FRA

Table 5.4 Wetting water characteristic curve properties of FRC and FRA

No	Description	Symbol	Soil	
			FRC	FRA
1	Saturated volumetric water content	θ_s	0.38	0.34
2	Residual volumetric water content	θ_r	0.24	0.02
3	Air-entry value	ψ_a	0.52	0.29
4	Fitting parameter	a	0.64	2.61
5		n	12.49	1.79
6		m	0.16	19.99
7		ψ_r	1.28	1.57
8	Saturated coefficient of permeability (m/s)	k_s	1×10^{-4}	5×10^{-3}

5.2.3. Permeability function

The permeability functions were predicted using the integration procedure reported in Fredlund et al. (1994). The Fredlund et al. (1994) integration method is explained in detail in Chapter 3. The drying permeability functions for the materials used in this study are shown in Figure 5.4. The wetting and drying permeability functions for fine RC and RA are shown in Figure 5.5. The permeability functions can be fitted using Leong and Rahardjo (1997) equations for direct input into the numerical modeling. The Leong and Rahardjo (1997) fitting parameters are shown in Table 5.5.

Figure 5.4 Drying permeability functions

Figure 5.5 Drying and wetting permeability functions of FRC and FRA

Table 5.5 Leong and Rahardjo (1997) fitting parameters for the permeability functions used in this study

Description	Symbol	Soil					
		FRC	FRA	FRC Wetting	FRA Wetting	CRC	CRA
Fitting parameter	a	18.27	4.33	0.63	1.79	0.45	0.06
	n	1.26	0.99	1.18	1.95	1.33	1.22
	m	0.84	1.03	2.76	20.08	0.82	1
	p	8.55	9.43	2.76	1.47	11.58	10.36
Saturated permeability (m/s)	k_s	1×10^{-4}	5×10^{-3}	1×10^{-4}	5×10^{-3}	1×10^{-1}	5×10^{-1}

5.2.4. Results of X-ray diffraction test

Four specimens were prepared for testing. The specimens were: dry specimen, a specimen that had undergone 28 days saturation, a specimen that had undergone 3 wet-dry cycles and a specimen that had undergone 6 wet-dry cycles. The results of the XRD test for the tested samples are presented in Figure 5.6a-d. The results indicated the formation of calcite for the specimens undergone 28 days saturation and 6 wet-dry cycles. Traces of calcium disilicate were also present for the specimens after 3 wet-dry cycles. The fact that calcite and calcium disilicate were

observed in the experiments showed that the FRC was cemented after a certain number of wet-dry cycles or after saturation.

Figure 5.6 Results of X-ray diffraction for the different specimens

5.2.5. Results of goniometer test

The typical contact angle obtained from the goniometer for an RA specimen is shown in Figure 5.7. The image processing software embedded in the goniometer calculated the contact angle of the water drop. Three tests were performed. The mean contact angle was found to be 113° . The goniometer also recorded the amount of time required for a drop of water to infiltrate the surface. This test is commonly called the water drop penetration time (WDPT) test. When a drop of water was dropped on the surface of RA, the water was not immediately absorbed at the

moment of contact. The water drop was retained on the surface of FRA for 50 seconds.

Figure 5.7. Water drop on RA Surface

5.3. RESULTS OF COLUMN STUDY

5.3.1. Preliminary numerical modeling results

The preliminary numerical modeling results are presented in Figure 5.8 as a graph of percent percolation against cover types. Percent percolation was defined as the amount of water flux going out from the bottom boundary of the model divided by the amount of water flux going into the model from the top boundary. The results show that a dual layer capillary barrier (DCB) with FRA as the fine-grained layer and CRA as the coarse-grained layer (2 layer FRA_CRA) was the best material combination because it had the lowest percent percolation. The preliminary numerical modeling results also show that when CRA was used as the coarse-grained layer, the percolation was lower regardless of the fine-grained layer material used. If FRA was used as the fine-grained layer, the percolation was also always lower regardless of the coarse-grained layer material used.

Figure 5.8 Percent percolation for different cover types obtained from 1-D preliminary numerical modeling

5.3.2. Laboratory column results

The objectives of the column study were to investigate the barrier effect produced by different combinations of RC and RA, investigate the increased storage of the modified capillary barrier system and investigate the effect of evaporation on the performance of the capillary barrier system

5.3.2.1 Column model consisting of 0.5 m thick FRA overlying 0.5 m thick CRA (CM_FRA_CRA)

The pore-water pressure head and total head profiles from the CM_FRA_CRA_UI test are presented in Figure 5.9. The upward infiltration test results can be used to obtain the saturated permeability of the materials in addition to saturating the soil column. The calculation of the saturated permeability from the upward infiltration test is provided in Appendix B.

Figure 5.9 Head profiles from the CM_FRA_CRA_UI1 test

The drawdown test was started after the upward infiltration test. The objective of the drawdown test was to position the water table at the bottom of the column. Pore-water pressure head profiles for the CM_FRA_CRA_DD1 test are presented in Figure 5.10. The pore-water pressure head profiles show that a negative pore-water pressure head developed after the test started and stabilized after approximately 24 hours.

Figure 5.10 Pore-water pressure head profiles from the CM_FRA_CRA_DD1 test

Rainfall tests were started after the pore-water pressures stabilized in the drawdown test. The first rainfall test (CM_FRA_CRA_RF1) had an intensity and duration of 4 mm/hr and 6 hours, respectively. Pore-water pressure head and volumetric water content profiles for CM_FRA_CRA_RF1 are presented in Figure 5.11 and Figure 5.12, respectively. The results show that the rainfall only affected the top portion of the column. The pore-water pressures became more positive during the wetting process, and returned to being negative during the drying process. Similarly, the volumetric water content increased during the wetting process and decreased during the drying process. There was no drainage observed during the CM_FRA_CRA_RF1 test. The water balance curves for the CM_FRA_CRA_RF1 test are presented in Figure 5.13.

Figure 5.11 Pore-water pressure head profiles from the CM_FRA_CRA_RF1 (4mm/hr for 6 hrs) test

Figure 5.12 Volumetric water content profiles from the CM_FRA_CRA_RF1 (4mm/hr for 6 hrs) test

Figure 5.13 Water balance curves from the CM_FRA_CRA_RF1 (4mm/hr for 6 hrs) test

The rainfall intensity was doubled for the CM_FRA_CRA_RF2 test. The pore-water pressure head and volumetric water content profiles for the CM_FRA_CRA_RF2 test are presented in Figure 5.14 and Figure 5.15,

respectively. The results show that the pore-water pressure moved towards positive values in the fine-grained layer, while the pore-water pressure head in the coarse-grained layer remained constant (capillary barrier effect). Around four (4) hours into the CM_FRA_CRA_RF2 test, water drainage was observed. The observation of drainage indicated that breakthrough had occurred and the storage capacity had been exceeded. The water balance curves for the CM_FRA_CRA_RF2 test are presented in Figure 5.16

For the CM_FRA_CRA_RF3 test, the rainfall duration was shortened to 1 hour and the rainfall intensity was increased to 50 mm/hr. Pore-water pressure head and volumetric water content profiles are presented in Figure 5.17 and Figure 5.18, respectively. There was a large increase in the pore-water pressure in the top portion of the column from the start of the rainfall until the end of rainfall. The water balance for the CM_FRA_CRA_RF3 test is presented in Figure 5.19. Similar to the previous test, drainage was also observed. Drainage occurred around 30 minutes after the rainfall test started.

Figure 5.14 Pore-water pressure head profiles from the CM_FRA_CRA_RF2 (8mm/hr for 6 hrs) test

Figure 5.15 Volumetric water content profiles from the CM_FRA_CRA_RF2 (8 mm/hr for 6 hrs) test

Figure 5.16 Water balance curves from the CM_FRA_CRA_RF2 (8mm/hr for 6 hrs) test

Figure 5.17 Pore-water pressure head profiles from the CM_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

Figure 5.18 Volumetric water content profiles from the CM_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

Figure 5.19 Water balance curves from the CM_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

The tests after CM_FRA_CRA_RF3 (RF4-RF6) had similar rainfall intensities to the previous tests. However, after the rainfall stopped, evaporation was applied. The typical temperature and relative humidity during the potential evaporation test are shown in Figure 5.20. The average temperature and relative humidity were around 40° Celsius and 40%, respectively. The potential evaporation rate was calculated from the weight of the water loss under these conditions. The average potential evaporation rate was around 5-7 mm/day. The evaporation rate obtained in this study is comparable to the typical potential evaporation rate in Singapore (Gasmo, 1997; Rahardjo et al., 2012).

The pore-water pressure head profiles and volumetric water content profiles from the CM_FRA_CRA_RF4 test are presented in Figure 5.21 and Figure 5.22, respectively. Due to the similar rainfall intensity, the pore water pressure from the wetting portion of the CM_FRA_CRA_RF4 test looks similar to the pore-water pressures obtained from the wetting portion of the CM_FRA_CRA_RF1 test (Figure 5.11). However, during the drying process, the pore-water pressure and volumetric water content profiles obtained from the CM_FRA_CRA_RF4 test were shifted more to the left, especially at the fine-grained layer portion of the column. The shift was more noticeable in the pore-water pressure profiles than in the volumetric water content profiles. The water balance curves for the

CM_FRA_CRA_RF4 test were identical to those for the CM_FRA_CRA_RF1 test (Figure 5.13).

Figure 5.20 Typical temperature, relative humidity and potential evaporation from the CM_FRA_CRA_RF4-6 test

Figure 5.21 Pore-water pressure head profiles from the CM_FRA_CRA_RF4 (4 mm/hr for 6 hrs) test

Figure 5.22 Volumetric water content profiles from the CM_FRA_CRA_RF4 (4 mm/hr for 6 hrs) test

The pore-water pressure head and volumetric water content profiles from the CM_FRA_CRA_RF5 test are presented in Figure 5.23 and Figure 5.24, respectively. The movement trends of the linear lines observed in CM_FRA_CRA_RF5 were similar to those observed in CM_FRA_CRA_RF2 (Figure 5.14); however, the pore-water pressure heads that developed had different values. The results also show that the pore-water pressure head along the FRA in the CM_FRA_CRA_RF5 test was located slightly to the left (i.e., a more negative value). This indicates that evaporation actually affected the pore-water pressure in deeper depth of the soil column and not only the surface of the soil column.

The water balance curves from the CM_FRA_CRA_RF5 test are presented in Figure 5.25. The water balance was not entirely complete since the amount of actual evaporation was not measured. The results show that the water balance curves from CM_FRA_CRA_RF5 follow a similar path to the water balance curves from CM_FRA_CRA_RF2 (Figure 5.16), except that the amount of water stored at the breakthrough for CM_FRA_CRA_RF5 was slightly higher than for CM_FRA_CRA_RF2.

Figure 5.23 Pore-water pressure head profiles from the CM_FRA_CRA_RF5(8 mm/hr for 6 hrs) test

Figure 5.24 Volumetric water content profiles from the CM_FRA_CRA_RF5 (8 mm/hr for 6 hrs) test

Figure 5.25 Water balance curves from the CM_FRA_CRA_RF5 (8 mm/hr for 6 hrs) test

The pore-water pressure head and volumetric water content profiles from the CM_FRA_CRA_RF6 test are presented in Figure 5.26 and Figure 5.27, respectively. Similar to the observations for the CM_FRA_CRA_RF5 test, the pore-water pressure profile shifted to a more negative value when evaporation was applied. This led to a lower volumetric water content value, as shown in the volumetric water content profile. The lower volumetric water content value increased the amount of water that the column could take up. The slight increase in water storage is shown in the water balance curve presented in Figure 5.28.

Figure 5.26 Pore-water pressure head profiles from the CM_FRA_CRA_RF6 (50 mm/hr for 1 hr) test

Figure 5.27 Volumetric water content profiles from the SCM_FRA_CRA_RF6 (50mm/hr for 1hr) test

Figure 5.28 Water balance curves from the SCM_FRA_CRA_RF6 (50mm/hr for 1hr) test

5.3.2.2 Column model consisting of 0.5 m thick FRC overlying 0.5 m thick CRC (CM_FRC_CRC)

The total head and pressure head profiles during the CM_FRC_CRC_UI test are presented in Figure 5.29. During the steady-state upward infiltration, there was little change in the pressure head that occurred in the coarse-grained layer, which indicated a high value of saturated permeability. The pore-water pressure head profile in the fine-grained layer was linear, which indicated a uniform value of the saturated coefficient of permeability across the fine-grained layer.

The pore-water pressure head profiles during the CM_FRC_CRC_DD test are presented in Figure 5.30. The results show that the pore-water pressure stabilized after 2 hours. After the pore-water pressure head from the CM_FRC_CRC_DD test stabilized, the CM_FRC_CRC_RF1 test was started. The pore-water pressure head and volumetric water content profiles from the CM_FRC_CRC_RF1 test are presented in Figure 5.31 and Figure 5.32, respectively. The pore-water pressure head changed considerably during the rainfall; however, the volumetric water content changes were considerably small. The water balance curves from the CM_FRC_CRC_RF1 test are shown in Figure 5.33. The water balance curves show that drainage occurred after around 3 hours of rainfall.

Figure 5.29 Head profiles from the CM_FRC_CRC_UI1 test

Figure 5.30 Pore-water pressure head profiles from the CM_FRC_CRC_DD test

Figure 5.31 Pore-water pressure head profiles from the CM_FRC_CRC_RF1 (4 mm/hr for 6 hrs) test

Figure 5.32 Volumetric water content profiles from the CM_FRC_CRC_RF (4 mm/hr for 6 hrs)test

Figure 5.33 Water balance curves from the CM_FRC_CRC_RF1 (4 mm/hr for 6 hrs) test

The pore-water pressure head and volumetric water content profiles obtained from the CM_FRC_CRC_RF2 test are presented in Figure 5.34 and Figure 5.35, respectively. Similar to the previous tests, the pore-water pressure head shifted to a more positive value during rainfall and returned to its initial condition during drainage. Drainage at the coarse-grained layer occurred around 4 hours after the rainfall started. This is confirmed by the water balance curves shown in Figure 5.36.

Figure 5.34 Pore-water pressure head profiles from the CM_FRC_CRC_RF2 (8 mm/hr for 6 hrs) test

Figure 5.35 Volumetric water content profiles from the CM_FRC_CRC_RF2 (8 mm/hr for 6 hrs) test

Figure 5.36 Water balance curves from the CM_FRC_CRC_RF2 (8 mm/hr for 6 hrs) test

The pore-water pressure head and volumetric water content profiles from the CM_FRC_CRC_RF3 test are presented in Figure 5.37 and Figure 5.38, respectively. The pore-water pressure head profiles showed an increase in pore-water pressure head in the fine-grained layer during rainfall, but no changes in the coarse-grained layer. The drainage in the coarse-grained layer occurred after the rainfall stopped as shown by the water balance curves in Figure 5.39.

Figure 5.37 Pore-water pressure head profiles from the CM_FRC_CRC_RF3 (50 mm/hr for 1 hr) test

Figure 5.38 Volumetric water content profiles from the CM_FRC_CRC_RF3 (50 mm/hr for 1 hr) test

Figure 5.39 Water balance curves from the CM_FRC_CRC_RF3 (50 mm/hr for 1 hr) test

5.3.2.3 Column model consisting of a dual capillary barrier (DCB) that consists of two stacks of 0.25 m FRA overlying 0.25 m CRA.

The pore-water pressure head profiles during the drawdown test (CM_DCB_FRA_CRA_DD) are shown in Figure 5.40. The pore-water pressure head profiles stabilized around 1 hour after drawdown. During the drawdown test, the pore-water pressure head decreased to around -0.2 m in the lower coarse-grained layer, after which the pore-water pressure head decreased further to around -0.45 m in the lower fine-grained layer. The pore-water pressure head in the upper coarse-grained layer then bent back to around -0.2 m, and then the pore-water pressure head in the upper fine-grained layer increased linearly to around -0.5 m. The pore-water pressure head profile has a zig-zag shape.

After the pore-water pressure from the CM_DCB_FRA_CRA_DD test stabilized, the CM_DCB_FRA_CRA_RF1 test was started. The pore-water pressure head and volumetric water content profiles during CM_DCB_FRA_CRA_RF1 test are shown in Figure 5.41 and Figure 5.42, respectively. The figure shows that there were limited changes in the pore-water pressure head in the column during the rainfall. Limited change in the volumetric water content was also observed during the test. The water balance graphs in Figure 5.43 show that no drainage occurred during the CM_DCB_FRA_CRA_RF1 test.

Figure 5.40 Pore-water pressure head from the CM_DCB_FRA_CRA_DD test

Figure 5.41 Pore-water pressure head from the CM_DCB_FRA_CRA_RF1 (4 mm/hr for 6 hrs) test

Figure 5.42 Volumetric water content profiles from the CM_DCB_FRA_CRA_RF1 (4 mm/hr for 6 hrs) test

Figure 5.43 Water balance curves from the CM_DCB_FRA_CRA_RF1 (4 mm/hr for 6 hrs) test

A rainfall of 8 mm/hr was applied during the CM_DCB_FRA_CRA_RF2 test. The pore-water pressure head and volumetric water content profiles during the CM_DCB_FRA_CRA_RF2 test are presented in Figure 5.44 and Figure 5.45, respectively. During the rainfall, the PWP head in the column moved towards a more positive value. The pore-water pressure head in the upper fine-grained layer moved to -0.28 m after one (1) hour of rainfall, while the pore-water pressure heads in other layers were not affected by the rainfall. After 3 hours of rainfall, the PWP head moved to around -0.15 m in the upper coarse-grained layer and -0.25 m in the lower fine-grained layer. However, at this time, the lower coarse-grained layer did not show any increase in the PWP head. The water balance curves presented in Figure 5.46 show that after approximately 6 hours the balance started to register an increase in the weight of the percolated water. At this time, the water had moved through the lower coarse-grained layer.

The pore-water pressure head and volumetric water content profiles from the CM_DCB_FRA_CRA_RF3 test are shown in Figure 5.47 and Figure 5.48, respectively. During the rainfall, the pore-water pressure profiles formed a 45° line in the upper fine-grained layer at around 30 minutes. The 45° profile line indicated that water had broken through the first interface. An increase in the pore-water pressure head was also observed in the upper coarse-grained layer. Around 1 hour

after the rainfall, the pore-water pressure in the lower fine-grained layer also shifted to a more positive value. No change was observed in the pore-water pressure in the lower coarse-grained layer. Similar observations are shown in the volumetric water content profiles. The water balance curves presented in Figure 5.49 show that water was stored in the column during rainfall and, when the rainfall stopped, drainage started to occur.

Figure 5.44 Pore-water pressure head profiles from the CM_DCB_FRA_CRA_RF2 (8 mm/hr for 6 hrs) test

Figure 5.45 Volumetric water content profiles from the CM_DCB_FRA_CRA_RF2 (8 mm/hr for 6 hrs) test

Figure 5.46 Water balance curves from the CM_DCB_FRA_CRA_RF2 (8 mm/hr for 6 hrs) test

Figure 5.47 Pore-water pressure head profiles from the CM_DCB_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

Figure 5.48 Volumetric water content profiles from the CM_DCB_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

Figure 5.49 Water balance curves from the CM_DCB_FRA_CRA_RF3 (50 mm/hr for 1 hr) test

5.4 RESULTS OF THE INFILTRATION BOX STUDY

The results of the infiltration box study are shown using the pore-water pressure against time plots, the volumetric water content against time plots and the water balance curves. The measurement points of the pore-water pressure and volumetric water content and the location of the drains for tests BM_SCB15, BM_SCB30 and BM_DCB are shown in Figure 5.50, Figure 5.51 and Figure 5.52, respectively. The measurement points can be categorized into four (4) cross section; A to D. Cross sections A, B, C and D have a horizontal distance of 1.75, 1.25, 0.75 and 0.25 m, respectively, from the toe of the slope.

Figure 5.50 Measurement points, cross sections and drain locations for test BM_SCB15

Figure 5.51 Measurement points, cross sections and drain locations for test BM_SCB30

Figure 5.52 Measurement points, cross sections and drain locations for test BM_DCB15

The water balance curves for the infiltration box study consist of three components; Precipitation was defined as the amount of rainfall applied to the physical model by the rainfall simulator. Lateral diversion for SCB was defined as the amount of water diverted laterally by the fine-grained layer as shown in the BM_SCB15 and BM_SCB30 models. Lateral diversion for DCB was defined as the amount of water diverted laterally by the upper fine-grained layer, upper coarse-grained layer and lower fine-grained layer as shown in the BM_DCB model. Percolation was defined as the amount of water that breaks through to the coarse-grained layer as shown in the BM_SCB15 and BM_SCB30 models. Percolation for DCB was defined as the

amount of water that breaks through to the lower coarse-grained layer as shown in the BM_DCB model. For all three physical models, the amount of percolation was recorded from the amount of water measured from Drain 1 (D1 in Figure 5.50, Figure 5.51 and Figure 5.52)

5.4.1 Single capillary barrier model consisting of 0.15 m thick FRA overlying 0.15 m thick CRA (BM_SCB15)

At the beginning of the experiment, the materials were in the saturated condition with a PWP head in a hydrostatic condition. The PWP head of the fine-grained layer was around 0.075 m, while the PWP head of the coarse-grained layer was around 0.225 m. The corresponding VWCs were around 38% and 42% for the fine- and coarse-grained materials, respectively. The box was then tilted to achieve a slope of 2°. The outlets were opened to allow water drainage. The PWP heads instantly dropped to negative values corresponding to the distance between the measurement points and the new position of the groundwater table. There was very little variation between the PWPs at different measurement sections. Similarly, the VWC readings during the drawdown test instantly dropped when the test started. The coarse-grained layer materials had a volumetric water content around 4%, whereas the fine-grained layer materials had a VWC around 20%.

The BM_SCB15_2deg_RF1 test was conducted 120 hours after the drawdown test with a rainfall intensity of 10 mm/hr. The changes in the pore-water pressure and volumetric water content during the BM_SCB15_2deg_RF1 test are shown in Figure 5.53. An increase in the PWP head was observed during rainfall. The PWP head quickly recovered to initial conditions when the rainfall stopped. The VWC measurements in the fine-grained layer increased during rainfall and recovered quickly to the initial conditions when the rainfall stopped. However, the coarse-grained layer did not register any change in the volumetric water content. The variation in PWP and VWC for the BM_SCB15_2deg_RF1 test was generally uniform for different sections. The water balance curves obtained from the BM_SCB15_2deg_RF1 test are presented in Figure 5.54.

Seventy two (72) hours after the end of rainfall 1, the BM_SCB15_2deg_RF2 test was started. The rainfall intensity of the BM_SCB15_2deg_RF2 test was 20 mm/hr. The characteristics of the PWP and VWC measurements were similar to those observed for the BM_SCB15_2deg_RF1 tests (Figure 5.53). The PWP head increased again during the test and recovered quickly when the rainfall stopped. The increase in the PWP head during the BM_SCB15_2deg_RF2 test was just slightly higher than for the BM_SCB15_2deg_RF1 test. The water balance curves for the BM_SCB15_2deg_RF2 test are presented in Figure 5.56. The amount of lateral diversion was around 30% of the total rainfall.

Figure 5.53 Pore-water pressure head and volumetric water content changes from the BM_SCB15_2deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.54 Water balances from the BM_SCB15_2deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.55 Pore-water pressure head and volumetric water content changes from the BM_SCB15_2deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.56 Water balances from the BM_SCB15_2deg_RF2 (20 mm/hr for 1 hr) test

About 72 hours after the start of the BM_SCB15_2deg_RF2 test, the infiltration box slope angle was increased from 2° to 5° . This increase in slope created a decrease in the initial pore-water pressure head for the next test. The decrease in PWP head was caused by changes in the ground-water table conditions produced by the new slope angle.

The pore-water pressure head and volumetric water content changes during the BM_SCB15_5deg_RF1 test are presented in Figure 5.57. The pore-water pressure head increased during the rainfall in the fine-grained layer, while a limited change was observed in the coarse-grained layer. The corresponding volumetric water content profiles followed a similar trend. The water balance curves for the BM_SCB15_5deg_RF1 test are presented in Figure 5.58. The amount of lateral diversion was around 60% of the total rainfall.

The pore-water pressure head and volumetric water content changes during the BM_SCB15_5deg_RF2 test are presented in Figure 5.59. The increase in the pore-water pressure head occurred mainly in the fine-grained layer during rainfall. When the rainfall stopped, the pore-water pressure heads in the fine-grained layer returned to their initial conditions, whereas the pore-water pressure head in the coarse-grained layer started to increase. The volumetric water content changes followed the

same trend except that no volumetric water change was observed in the coarse-grained layer. The water balance curves for the BM_SCB15_5deg_RF2 test are presented in Figure 5.60. The amount of lateral diversion was around 50% of the total rainfall.

Figure 5.57 Pore-water pressure head and volumetric water content changes from the BM_SCB15_5deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.58 Water balances from the BM_SCB15_5deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.59 Pore-water pressure head and volumetric water content changes from the BM_SCB15_5deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.60 Water balances from the BM_SCB15_5deg_RF2 (20 mm/hr for 1 hr) test

5.4.2 Single capillary barrier model consisting of 0.30 m thick FRA overlying 0.15 m thick CRA (BM_SCB30)

The pore-water pressure head and volumetric water content changes from the BM_SCB30_2deg_RF1 test are presented in Figure 5.61. The increase in the pore-water pressure head in the fine-grained layer occurred instantaneously after the rainfall started. When the rainfall stopped, the pore-water pressure heads in the fine-grained layer returned to their original conditions, whereas the pore-water pressure heads in the coarse-grained layer still changing. The volumetric water content results correspond with the pore-water pressure head profiles. The water balance curves from the BM_SCB30_2deg_RF1 test are presented in Figure 5.62. The amount of lateral diversion was around 50% of the total rainfall. Similar trends were observed for the pore-water pressure head and volumetric water content changes from the BM_SCB30_2deg_RF2 test, as shown in Figure 5.63. The water balance curves presented in Figure 5.64 show lateral diversion of around 40% of the total rainfall for the BM_SCB30_2deg_RF2 test.

After the pore-water pressure heads from the BM_SCB30_2deg_RF2 test stabilized, the box slope angle was increased from 2° to 5°. When the pore-water pressure heads stabilized, the BM_SCB30_5deg_RF1 test was started. The pore-water pressure head and volumetric water content changes from the

BM_SCB_30_5deg_RF1 test are presented in Figure 5.65. The increase in pore-water pressure during rainfall occurred in the fine-grained layer only. For section A and section B, the pore-water pressure head in the coarse-grained layer did not change. This indicated that water did not enter the coarse-grained layer in section A and B. The water balance curves for the BM_SCB30_5deg_RF1 test are presented in Figure 5.66. The water balance curves show that about 80% of the water was diverted by the SCB system.

Figure 5.61 Pore-water pressure head and volumetric water content changes from the BM_SCB30_2deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.62 Water balances from the BM_SCB30_2deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.63 Pore-water pressure head and volumetric water content changes from the BM_SCB30_2deg_RF2(20 mm/hr for 1 hr) test

Figure 5.64 Water balances from the BM_SCB30_2deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.65 Pore-water pressure head and volumetric water content changes from the BM_SCB30_5deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.66 Water balances from the BM_SCB30_5deg_RF1(10 mm/hr for 1 hr) test

When the pore-water pressure head from the BM_SCB30_5deg_RF1 test stabilized, the BM_SCB30_5deg_RF2 test was started. The rainfall intensity of the test was 20 mm/hr. The pore-water pressure head and volumetric water content changes obtained from the BM_SCB30_5deg_RF2 test are presented in Figure 5.67. Similar trends as for the BM_SCB30_5deg_RF1 test were observed. The pore-water pressure heads in the coarse-grained layer at section A and section B did not show any movement. The volumetric water content changes followed the changes in the pore-water pressure head. The water balance curves obtained from the BM_SCB30_5deg_RF2 test are shown in Figure 5.68. The results show that almost 80% of the rainfall was diverted by the capillary barrier system.

The pore-water pressure heads and volumetric water content changes obtained during the BM_SCB30_7deg_RF1 test are presented in Figure 5.69. The pore-water pressure head changes during the rainfall were observed by the tensiometers in the fine-grained layer; however, no changes in the pore-water pressure head were observed in the coarse-grained layer of the SCB. This indicates that the infiltrated water was laterally diverted by the SCB. This is supported by the water balance curves presented in Figure 5.70. The water balance graph shows there was no percolation into the coarse-grained layer.

Figure 5.67 Pore-water pressure head and volumetric water content changes from the BM_SCB30_5deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.68 Water balances from the BM_SCB30_5deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.69 Pore-water pressure head and volumetric water content changes from the BM_SCB30_7deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.70 Water balances from the BM_SCB30_7deg_RF1 (10 mm/hr for 1 hr) test

The pore-water pressure head and volumetric water content changes from the BM_SCB30_7deg_RF2 test are presented in Figure 5.71. Similar trends as from the previous tests were observed. There were no changes in the pore-water pressure head in the coarse-grained layer, which indicates that the SCB diverted the rainfall water laterally. The water balance curves presented in Figure 5.72 confirm that no percolation into the coarse-grained layer occurred in the BM_SCB30_7deg_RF2 test.

Figure 5.71 Pore-water pressure head and volumetric water content changes from the BM_SCB30_7deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.72 Water balances from the BM_SCB30_7deg_RF2 (20 mm/hr for 1 hr) test

5.4.3 Dual capillary barrier model consisting of two stacks of 0.15 m thick FRA overlying 0.15 m thick CRA (BM_DCB)

The pore-water pressure head and volumetric water content changes obtained from the BM_DCB_2deg_RF1 test are presented in Figure 5.73. Changes in the upper and lower fine-grained layer pore-water pressure heads were observed in the test. Changes in the pore-water pressure head in the upper fine-grained layer were also monitored. There were no changes in the pore-water pressure head in the lower coarse-grained layer. This indicates that the water did not infiltrate the lower coarse-grained layer. The volumetric water contents of the upper and lower coarse-grained layer were very similar, which are around the residual volumetric water content of the material. During the test, there were very limited changes in the volumetric water content of these layers. In contrast, the volumetric water contents of the upper and lower fine-grained layers show changes of around 0.02-0.05. The water balance curves shown in Figure 5.74 indicate that almost 90% of the total rainfall was laterally diverted.

Figure 5.73 Pore-water pressure head and volumetric water content changes from the BM_DCB_2deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.74 Water balances from the BM_DCB_2deg_RF1 (10 mm/hr for 1 hr) test

The pore-water pressure head and volumetric water content changes from the BM_DCB_2deg_RF2 test are presented in Figure 5.75. Similar trends as in the previous test were observed. The water balance curves obtained from the BM_DCB_2deg_RF2 test are presented in Figure 5.76. The water balance curves show that almost 80% of the total rainfall was laterally diverted.

Figure 5.75 Pore-water pressure head and volumetric water content changes from the BM_DCB_2deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.76 Water balances from the BM_DCB_2deg_RF2 (20 mm/hr for 1 hr) test

After the pore-water pressure head stabilized from the BM_DCB_2deg_RF2 test, the box slope angle was increased from 2° to 5° . The BM_DCB_5deg_RF1 test was then started. The pore-water pressure head and volumetric water content changes from the BM_DCB_5deg_RF1 test are presented in Figure 5.77. The pore-water pressure head in the upper and lower fine-grained layers increased during the rainfall. The water balance curves for the BM_DCB_5deg_RF1 test are presented in Figure 5.78. The results show that breakthrough into the lower coarse-grained layer was not observed.

When the pore-water pressure heads returned to their initial conditions, the BM_DCB_5deg_RF2 test was started. The pore-water pressure head and volumetric content changes obtained from the BM_DCB_5deg_RF2 test are presented in Figure 5.79. Similar trends as for the previous rainfall test were observed for the BM_SCB_5deg_RF2 test. The water balance curves for the BM_DCB_5deg_RF2 test are presented in Figure 5.80. The water balance curves also show that no breakthrough into the lower coarse-grained layer was observed in BM_DCB_5deg_RF2 test. The lateral diversion on the water balance graphs obtained from BM_DCB test consists of volumes of water that drains out from drains 2,3 and 4 (Figure 5.52). The water balance graphs with details on the amount of water leaving the infiltration box for each drain are shown in Appendix C.

Figure 5.77 Pore-water pressure head and volumetric water content changes from the BM_DCB_5deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.78 Water balances from the BM_DCB_5deg_RF1 (10 mm/hr for 1 hr) test

Figure 5.79 Pore-water pressure head and volumetric water content changes from the BM_DCB_5deg_RF2 (20 mm/hr for 1 hr) test

Figure 5.80 Water balances from the BM_DCB_5deg_RF2 (20 mm/hr for 1 hr) test

5.5 RESULTS OF NUMERICAL MODELING

The results of numerical modeling are presented in this section. The numerical results were compared to the experimental data in terms of pore-water pressure head and volumetric water content profiles. The drying WCCs and drying permeability functions were used for the numerical modeling. The use of wetting WCCs and wetting permeability functions in the numerical modeling is only presented in the numerical modeling of the CM_FRA_CRA test.

5.5.1 Numerical modeling of the column model

5.5.1.1 Numerical modeling of CM_FRA_CRA

The results of numerical modeling of the CM_FRA_CRA test are represented by the CM_FRA_CRA_RF2 and CM_FRA_CRA_RF5 tests. The results of numerical modeling for the other tests are presented in Appendix D. Figure 5.81 shows a comparison of the experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_2 using drying material properties. The results show that numerical modeling can match the experimental data during and after rainfall. Comparisons of the same test using the wetting material properties are presented in Figure 5.82. However, the results with wetting material properties show a poor match between the experimental and numerical results. The corresponding volumetric water content profiles for wetting and drying material properties are shown in Figure 5.83 and Figure 5.84, respectively. Similar to the pore-water pressure results, the drying material properties match the experimental data well. The results of the CM_FRA_CRA_RF2 numerical modeling using wetting and drying results are typical of other test's wetting and drying comparisons. Therefore, the following presentation of results only show the results of numerical modeling using the drying material properties.

Figure 5.81 Comparisons of experimental and numerical pore-water pressures for the CM_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test (drying)

Figure 5.82 Comparisons of experimental and numerical pore-water pressures for the CM_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test (wetting)

Figure 5.83 Comparisons of experimental and numerical volumetric water contents for the CM_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test (drying)

Figure 5.84 Comparisons of experimental and numerical volumetric water contents for the CM_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test (wetting)

The pore-water pressures head and volumetric water contents obtained from experimental and numerical modeling of the CM_FRA_CRA_RF5 test are presented in Figure 5.85 and Figure 5.86, respectively. The results show that the numerical modeling follows the experimental data quite well during the wetting process. However, during the drying process, the experimental data were generally more negative than the numerical modeling. On the other hand, the volumetric water content profiles during the wetting and drying processes obtained from the numerical modeling closely match the experimental data. The comparisons of experimental and numerical water balance graphs for CM_FRA_CRA_RF5 test is shown in Figure 5.87. The starts of drainage obtained from the numerical and experimental were almost similar however, the total amounts of water after 48 hrs show a little discrepancy.

Figure 5.85 Comparisons of experimental and numerical pore-water pressures for the CM_FRA_CRA_RF_5 (8 mm/hr for 6 hrs) test.

Figure 5.86 Comparisons of experimental and numerical volumetric water contents for the CM_FRA_CRA_RF_5 (8 mm/hr for 6 hrs) test

Figure 5.87 Comparisons of experimental and numerical water balance graphs for the CM_FRA_CRA_RF_5 (8 mm/hr for 6 hrs) test

5.5.1.2 Numerical modeling of CM_FRC_CRC

The results of numerical modeling of the CM_FRC_CRC column are represented by the results of the CM_FRC_CRC_RF2 test. The other results of numerical modeling of the CM_FRC_CRC column are presented in Appendix D. The pore-water pressure head and volumetric water content profiles obtained from the experiment and the numerical modeling of test CM_FRC_CRC_RF2 are presented in Figure 5.88 and Figure 5.89, respectively. The pore-water pressure head obtained from the experimental data shifted considerably in the upper portion of the fine-grained layer before changes in the lower portion of the fine-grained layer took place. On the other hand, the pore-water pressure head obtained from the numerical modeling shifted linearly across the fine-grained layer. The volumetric water content obtained from the numerical modeling shows very good agreement with the experimental data. The agreement in the volumetric water content data was also found in the comparisons of water balance graphs shown in Figure 5.90.

Figure 5.88 Comparisons of experimental and numerical pore-water pressures for the CM_FRC_CRC_RF_2 (8 mm/hr for 6 hrs) test

Figure 5.89 Comparisons of experimental and numerical volumetric water contents for the CM_FRC_CRC_RF_2 (8 mm/hr for 6 hrs) test

Figure 5.90 Comparisons of experimental and numerical water balance graphs for the CM_FRC_CRC_RF_2 (8 mm/hr for 6 hrs) test

5.5.1.3 Numerical modeling of CM_DCB_FRA_CRA

The results of numerical modeling of the CM_DCB_FRA_CRA column are represented by the results of the CM_DCB_FRA_CRA_RF2 test. The other results of numerical modeling of the CM_DCB_FRA_CRA column are presented in Appendix D. The pore-water pressure head, volumetric water content profiles, water balance curves obtained from the numerical modeling along with the experimental data are presented in Figure 5.91, Figure 5.92 and Figure 5.93, respectively. Some discrepancies during rainfall were observed from the comparisons of the pore-water pressure heads, especially in the upper fine and coarse-grained layers. Discrepancies in the lower fine-grained layer volumetric water content were also observed from the comparisons of the volumetric water content profiles. Nevertheless, the trend of movement in the numerical modeling was similar to the experimental data. The water balance graphs obtained from numerical modeling also show similar trend of changes and the water volument amount to those obtained from the laboratory experiments.

Figure 5.91 Comparisons of experimental and numerical pore-water pressures for the CM_DCB_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test

Figure 5.92 Comparisons of experimental and numerical volumetric water contents for the CM_DCB_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test

Figure 5.93 Comparisons of experimental and numerical water balance graphs for the CM_DCB_FRA_CRA_RF_2 (8 mm/hr for 6 hrs) test

5.5.2 Numerical modeling of box model

The results of numerical modeling of the box model are presented in the form of pore-water pressure head and volumetric water content against time plots for the models BM_SCB15 and BM_SCB30. For the BM_DCB model, the pore-water pressure head and volumetric water content profiles were used to better visualize the results. Similar to the column model, the drying material properties were used in the numerical modeling as they gave better results than the wetting properties.

5.5.2.1 Numerical modeling of BM_SCB15

The results of numerical modeling of BM_SCB15 are represented by test BM_SCB15_RF2_2deg. The other results of numerical modeling of BM_SCB15 can be found in the Appendix E. The pore-water pressure head and volumetric water content obtained from the numerical modeling, along with the experimental data, are shown in Figure 5.94 and Figure 5.95, respectively.

The comparison of pore-water pressure shows good agreement between the numerical modeling and experimental data. Slight discrepancies during drainage occurred. The numerical modeling pore-water pressure head during drainage decreased instantaneously after the rainfall stopped, whereas the experimental pore-water pressure head decreased with a gentler slope. The volumetric water content comparisons between the laboratory experiment and numerical modeling show a close match.

Figure 5.94 Comparisons of experimental and numerical pore-water pressures for the BM_SCB15_RF2_2deg (20 mm/hr for 1 hr) test

Figure 5.95 Comparisons of experimental and numerical volumetric water content for test BM_SCB15_RF2_2deg (20 mm/hr for 1 hr).

5.5.2.2 Numerical modeling of BM_SCB30

The results of numerical modeling of BM_SCB30 are represented by the BM_SCB3_RF2_5 degree test. The other results for the numerical modeling of the BM_SCB30 model can be found in Appendix E. The pore-water pressure heads and volumetric water contents obtained from the numerical modeling, along with the experimental results, are presented in Figure 5.96 and Figure 5.97, respectively. The pore-water pressure head comparisons show good agreement between the laboratory experiment and numerical modeling results. Slight discrepancies in the pore-water pressure heads in the coarse-grained layer were observed. The volumetric water content comparisons show that the volumetric water content in the

fine-grained layers obtained from the laboratory experiments were generally lower than the results of the numerical modeling. However, the volumetric water content trend obtained from the laboratory experiment was generally in agreement with the numerical modeling.

Figure 5.96 Comparisons of experimental and numerical pore-water pressure heads for the BM_SCB30_RF2_5deg (20 mm/hr for 1 hr) test

Figure 5.97 Comparisons of experimental and numerical volumetric water contents for the BM_SCB15_RF2_5deg (20 mm/hr for 1 hr) test

5.5.2.3 Numerical modeling of BM_DCB

The results of the numerical modeling of BM_DCB are represented by the results of the BM_DCB_RF2_5deg test. The other results of numerical modeling of BM_DCB can be found in the Appendix E. The pore-water pressure and volumetric water content profiles obtained from the numerical modeling, along with the experimental data, are presented in Figure 5.98 and Figure 5.99. The pore-water pressure comparison shows slight discrepancies in the lower fine-grained and lower coarse-grained layers. However, the movement trends of the pore-water pressure heads are similar. The comparisons of the volumetric water contents between the laboratory experiment and numerical modeling are also in close agreement.

Figure 5.98 Comparisons of experimental and numerical pore-water pressure heads for the BM_DCB_RF2_5deg (20 mm/hr for 1 hr) test

Figure 5.99 Comparisons of experimental and numerical volumetric water contents for the BM_DCB_RF2_5deg (20 mm/hr for 1 hr) test

5.5.2.4 Lateral diversion comparisons obtained from numerical modeling and box infiltration test.

The lateral diversion percentages obtained from numerical modeling and box infiltration test is summarized in Figure 5.100. The results shows good agreement between the numerical modeling and the laboratory box infiltration tests.

Figure 5.100 Comparison of lateral diversion percentages obtained from numerical modeling and box infiltration test

5.5.3 Results of the parametric study

The results of the parametric study are represented by the water storage capacity and lateral diversion capacity for the 1-D and 2-D conditions, respectively. In this study, the water storage capacity was normalized by the total amount of rainfall. A normalized water storage of one (1) indicates that the column could store the entire amount of rainfall in the column. Similarly, the normalized lateral diversion is also normalized by the amount of rainfall. When the normalized lateral diversion equals one (1), it means the total amount of rainfall was diverted fully to the drainage port.

5.5.3.1 Effect of layer thickness on SCB performance

The effect of layer thickness on the normalized water storage and the normalized lateral diversion capacity of an SCB is presented in Figure 5.101 and Figure 5.102,

respectively. The results show that an increase in fine-grained layer thickness increases the normalized water storage more than an increase in coarse-grained layer thickness. An increase in coarse-grained layer thickness increases the normalized water storage up to a certain threshold value. Beyond the threshold thickness, there is no increase in normalized water storage. This behavior was also observed for the effect of coarse-grained layer thickness on the lateral diversion capacity of an SCB.

Figure 5.101 Effect of layer thickness on the normalized water storage capacity of an SCB

Figure 5.102 Effect of layer thickness on the normalized lateral diversion capacity of an SCB (Model length $L=2\text{m}$)

The effect of model length on the lateral diversion capacity percentage and lateral diversion length of SCB are shown in Figure 5.103. The figure shows that if the numerical model length is extended from the current 2 m, the lateral diversion length increases. The increase is caused by the increase in negative pore-water pressure in the fine-grained layer at initial condition due to the increase in distance to the groundwater table.

Figure 5.103 Effect of model length on lateral diversion capacity and lateral diversion length

5.5.3.2 Effect of layer thickness on DCB performance

The effect of layer thickness on the normalized water storage and normalized lateral diversion capacity are shown in Figure 5.104 and Figure 5.105, respectively. The results show that an increase in fine-grained layer thickness (upper and lower) significantly increase the water storage and lateral diversion capacity, whereas an increase in coarse-grained layer thickness increases the water storage and lateral diversion capacity up to a certain threshold. After the threshold value, an increase in coarse-grained layer thickness does not increase the water storage or the lateral diversion capacity significantly. The upper fine-grained layer thickness tends to be the most influential layer affecting the water storage and lateral diversion capacity.

Figure 5.104 Effect of layer thickness on the normalized water storage capacity of a DCB

Figure 5.105 Effect of layer thickness on the normalized lateral diversion capacity of a DCB (Model Length $L=2m$)

CHAPTER 6

DISCUSSION

6.1 INTRODUCTION

The results obtained in this study are discussed in this chapter. The use of recycled asphalt and recycled concrete as a capillary barrier construction material is discussed in Section 6.2. The behavior of the capillary barrier in 1-D and 2-D conditions is discussed in Section 6.3 and Section 6.4, respectively. The behavior of DCB in 1-D and 2-D conditions is discussed in Section 6.5. This is followed by a discussion of the results of the numerical modeling and parametric study in Section 6.6. The chapter ends with a discussion of the design of DCB in Section 6.7.

6.2 THE USE OF RECYCLED ASPHALT & RECYCLED CONCRETE AS CAPILLARY BARRIER CONSTRUCTION MATERIAL

6.2.1 *Basic properties*

The grain size distribution curves in Figure 5.1 show that CRC and CRA can be categorized as poorly graded gravel. Both CRC and CRA are designed by the recycling plant to have an average diameter of 14 mm. However, CRC tends to have more fines resulting from the crushing process. FRC and FRA can be categorized as well-graded sand from the grain size distribution curves. Both FRC and FRA have fines content of around 5%. An important consideration related to the suitability of a material for capillary barrier construction is the potential of fines migration from the fine-grained layer to the coarse-grained layer. A theoretical criterion has been proposed to avoid fines migration. The criterion is that the d_{15} (grain size diameter corresponding to the 15% percent finer on the grain-size distribution curve) of the coarse-grained layer over the d_{15} of the fine-grained layer is less than 4 (Terzaghi,

1922). This theoretical criterion is satisfied by the material combinations used as capillary barrier material in this study (FRA over CRA and FRC over CRC).

The specific gravity of RA obtained from this study was around 2.4. This is smaller than the specific gravity of sand or gravel, which was around 2.6-2.7. The lower density can be attributed to the lower density coating of the asphalt binder material. The specific gravity of RC obtained from this study was around 2.5-2.6. This value was only slightly lower than the specific gravity of sand or gravel. The values of the specific gravity of RC and RA obtained from this study were comparable to the values reported by Rahardjo et al. (2010) and Rathje et al. (2001). The lower specific gravity of RC and RA gives an advantage to the use of RC and RA as capillary barrier construction material by having a lower weight for the same amount of material volume as compared to natural aggregates. This could be beneficial as the lower weight could cause less settlement.

6.2.2 Hydraulic properties

The coefficient of saturated permeability (k_s) of the materials used in this study was obtained from a flexible wall permeameter constant head test. CRA had the highest k_s at 5×10^{-1} m/s, while FRC had the lowest saturated permeability (k_s) at 1×10^{-4} m/s. The measured k_s of the materials can be compared to the k_s calculated from empirical equations, such as from Kozeny-Carman (Kozeny, 1927 and Carman, 1956) and Hazen (1892). The Kozeny-Carman (1956) and Hazen (1892) equations are shown as follow:

$$\text{Kozeny-Carman (1956); } k_s (m/s) = \frac{(\rho g)}{\mu} \frac{(n^3)}{1 - n^2} \left(\frac{d_{10}^2}{180} \right) \quad (6.1)$$

where: ρ (Kg/m^3) is density of water, g (m^2/s) is gravitational acceleration, μ is dynamic viscosity (Ns/m^2), n is porosity and d_{10} is the grain size diameter that corresponds to 10% passing in mm).

$$\text{Hazen (1892) } k_s (m/s) = 0.01 \times d_{10}^2 \quad (6.2)$$

These equations are commonly used as a quality control tool for k_s . The comparisons are shown in Table 6.1. The results show that the measured k_s values are close to the calculated k_s values.

Table 6.1 Comparisons of measured k_{sat} to calculated k_{sat}

Materials	Porosity (n)	d_{10} (mm)	k_s Kozeny-Carman (m/s)	k_s Hazen (m/s)	k_s measured (m/s)
CRA	0.39	8.56	0.8	0.7	0.5
CRC	0.45	6.93	1	0.5	0.1
FRA	0.35	0.32	7×10^{-4}	1×10^{-3}	5×10^{-3}
FRC	0.39	0.1	1×10^{-4}	1×10^{-4}	1×10^{-4}

The saturated permeability for FRC was also obtained for several wet-dry cycles using the falling head permeability method. The wet-dry cycles were meant to induce cementation of FRC. The XRD test results showed that cementation occurred after saturation, three (3) wet-dry cycles and 6 wet-dry cycles. However, the saturated coefficients of permeability of FRC for different wet-dry cycles were similar at 10^{-4} m/s. A study by Poon et al. (2006) showed that cementation affects the permeability of FRC when the active portion of FRC is more than 40%. The active portion of FRC was defined as the portion of FRC with a particle size between 0.3-0.6 mm and less than 0.15 mm. An XRD analysis exercise showed that when the particle size was between 0.3-0.6 mm and less than 0.15 mm, cementation would most likely occur (Poon et al., 2006). In this study, FRC had an active portion of around 25%. Therefore, it was still below the limit or threshold studied by Poon et al. (2006). If the active portion is more than 40%, cementation could clog the pores of the FRC, resulting in a decrease in the saturated coefficient of permeability.

The results of the goniometer test showed that the contact angle and WDPT of RAP were 113° and 50 seconds, respectively. Based on the table by Bauters et al. (2000) shown in Table 6.2, the material can be categorized as between slightly and extremely water repellant. The fact that the surface of the FRA repelled water may be the reason that the saturated permeability was higher than the saturated permeability predicted from the grain size

Table 6.2 The water drop penetration time (WDPT) and apparent contact angle for different degrees of water repellency in soil (From Dekker and Ritsema, 1994)

WDPT (s)	Apparent contact angle (°)	Water entry (cm)	Description
<0.5	0	-7.5	Wettable (<5sec)
40	67	-3	Slightly water repellent
2400	90	0	Severely water repellent
>3600	98	1	Extremely water repellent
>3600	122	4	Extremely water repellent

The drying water characteristic curves (WCC) of the materials used in this study were measured using a Tempe Cell, while the wetting water characteristic curves were measured using a capillary rise tube test. The drying WCCs are shown in Figure 5.2. The relative positions and characteristics of the WCC curves were consistent with the grain size distributions of the materials. This showed that the grain size distributions of the materials have a relationship with their WCCs.

WCC can be predicted using grain size distribution analysis, as presented by Fredlund et al. (2002). The predicted WCC from the grain size distribution for RC and RA is presented in Figure 6.1. The air-entry values of FRC and CRC obtained from the prediction were lower than those obtained from the laboratory experiments. The residual volumetric water contents of FRC and CRC obtained from the prediction were also lower compared to those obtained from the laboratory experiment. The underestimation of the predicted WCC may have been caused by the high water adsorption properties of FRC and CRC (FHWA, 2000). The high water adsorption properties of FRC and CRC were caused by the presence of cement (CaCO_3) on the surface of the RC. Therefore, even though the porosity of FRC was higher compared to that of FRA, the cement in RC provided more surface area, which absorbed more water. The high water adsorption caused the material to hold water up to a higher negative pore-water pressure (higher air-entry value) and to have a higher residual water content. In fact, a few studies have pointed out that FRC can have dual porosity properties, as indicated by the two air-entry values. The first air-entry value is commonly less than 1 kPa, which is caused by the loss of water from the pores in the bigger particles, while the second air-entry value at significantly higher pressure is caused by the loss of water from the concrete

portion (Edil et al., 2012). However, in this study, the WCC of FRC did not show dual porosity properties. This may have been because the amount of cement on the surface of the RC was different than the one studied in the literature.

Figure 6.1 Predicted and measured WCC of RC and RA

There are limited data on the water characteristics of RC and RA. Rahardjo et al. (2010) performed Tempe cell tests for different gradations of RC and RA. From the grain size curves, the closest materials to FRC, CRC, FRA and CRA are RCA3, RCA1, RAP3 and RAP1, respectively. The WCCs of recycled materials from Rahardjo et al. (2010) are shown in Figure 6.2. The WCCs obtained by Rahardjo et al. (2010) show similar trends to the WCCs obtained from this study (CRA has the lowest air-entry value, FRC has the highest air-entry value, and so on). However, the details, such as the value of air-entry pressure and fitting parameters, are slightly different. The Fredlund and Xing (1994) parameters from the WCCs obtained from this study and the study by Rahardjo et al. (2010) are summarized in Table 6.3.

The results of the wetting WCC obtained from the capillary rise tube test show considerable differences from the drying WCC. The air and water-entry values of the wetting WCC were smaller than the air and water-entry values of the drying WCC. The slopes of the wetting WCC were also steeper compared to those of the

drying WCC. This shows that the WCC was not unique, but related to the initial condition (wetting or drying) of the soil. However, the saturated volumetric water contents of the wetting and drying WCCs were very close. This supports the study by Pham et al. (2005) that reported the amount of entrapped air during the wetting process to be around 5%.

Figure 6.2 WCCs for different recycled materials (From Rahardjo et al., 2010)

Table 6.3 Parameters for WCCs obtained from this study and from the study by Rahardjo et al. (2010)

No	Description	Symbol	Material							
			FRC	RCA3	FRA	RAP3	CRC	RCA1	CRA	RAP1
1	Saturated volumetric water content	θ_s	0.38	0.33	0.34	0.37	0.45	0.48	0.38	0.39
2	Residual volumetric water content	θ_r	0.15	0.16	0.11	0.04	0.13	0.09	0.10	0.03
3	Air-entry value	ψ_a	4.35	22.00	0.79	0.03	0.25	0.04	0.02	0.01
4	Fitting parameter	a	15.36	69.64	2.62	0.91	0.33	0.05	0.04	0.01
5		n	0.95	1.15	1.01	1.46	4.71	7.97	3.16	31.09
6		m	0.83	1.00	0.95	1.53	0.69	0.47	0.82	0.51
7		ψ_r	331.00	-	55.00	-	0.89	-	0.19	-

The permeability functions of the materials used in this study are shown in Figure 5.4. The shapes of the functions are similar to their WCC. The saturated permeability decreases after the air-entry value of the material, and becomes very small when the material becomes more unsaturated.

6.2.3 Desired material for capillary barrier construction

Preliminary numerical modeling of the 1-D capillary barrier results showed that the water-entry value of the coarse material was the threshold pressure in the interface for water to break through. A comparison of the pore-water pressure profiles at breakthrough for FRA over CRA and FRA over CRC is shown in Figure 6.3. A lower water-entry value for CRA would give an advantage to the capillary barrier system by allowing more water in the fine-grained layer. The results support the study by Tami et al. (2004b) that also reported the importance of the water-entry value of the coarse-grained layer.

Preliminary numerical modeling results also showed that, if FRA was used as the fine-grained layer, the percolation was always smaller regardless of the coarse-grained layer material used. This was because FRA has a higher water storage value than FRC, which was caused by the low initial water content of FRA as shown in Figure 6.4

Figure 6.3 Comparison of pore-water pressure profile at breakthrough for FRA-CRA and FRA-CRC

Two characteristics of the desired recycled materials were identified from the preliminary numerical modeling: 1. The coarse-grained layer should have a low water-entry value; the lower the water-entry value of the coarse-grained layer, the higher the amount of water that can be stored in the fine-grained layer. 2. The fine-grained layer should have a high water storage value. The desired materials can be summarized using their WCC as shown in Figure 6.5. The explanation can also be seen in Harnas et al. (2013).

Figure 6.4 Comparison of the volumetric water content profiles for FRA-CRA and FRC-CRA at initial conditions and at breakthrough

Figure 6.5 Desired characteristics for the fine and coarse-grained layer of a capillary barrier

6.3 1-D BEHAVIOR OF CAPILLARY BARRIER MADE FROM RA & RC

6.3.1 Behavior during drawdown test

The results of the drawdown tests for both the column and box models reached a condition commonly known as the static non-equilibrium condition (Barbour and Yanful, 1994; Choo and Yanful, 2000; Yang et.al., 2004). Static non-equilibrium condition refers to a condition where the maximum negative pore-water pressures that develop during drainage are approximately the same as the residual pore-water pressure head of the material. For example, the pore-water pressure head on column CM_FRA_CRA_DD (Figure 5.10). The decrease in the pore-water pressure head after the start of the drawdown test occurred rapidly and, within 15 minutes, the pore-water pressure had stabilized. This was because the residual volumetric water content of the CRA had been reached and the water phase had become discontinuous. The ultimate negative pore-water pressure head obtained in the coarse layer was around -0.2 m, which was close to the residual matric suction of CRA. When the ultimate negative pore-water pressure in the coarse-grained layer was obtained, the changes in the pore-water pressure in the fine-grained layer slowed down. This was because, in this condition, the permeability of the coarse-grained layer was very low, which impeded the movement of water from the fine to the coarse-grained layer.

This phenomenon was also observed in the box model. After the drawdown test, the pore-water pressure head in the coarse-grained layer decreased to a value corresponding to the distance between the measurement points and the ground water table. However, if the distance was higher than the residual matric suction head of the coarse-grained layer, the pore-water pressure head at the measurement point was at a value close to the residual matric suction head.

The typical pore-water pressure head that developed after a drawdown test is shown in Figure 6.6. The pore-water pressure head in the coarse-grained layer decreased along with the increase in column height. When the residual matric-suction head was reached, the pore-water pressure head became constant, as shown by the vertical line. The pore-water pressure head then decreased again, starting at the

interface up to the surface. The implication of the residual matrix suction limit on the capillary barrier system is that the thickness of the coarse-grained layer can be limited to the residual matrix suction head without decreasing the effectiveness of the capillary barrier.

Figure 6.6 Typical pore-water pressure head developed after a drawdown test.

6.3.2 Behavior during rainfall and drainage

The rainfall tests were started after the pore-water pressures stabilized in the drawdown tests. Generally, the responses of the column during rainfall and drainage for different rainfall intensities were similar, i.e., the pore-water pressure head moved from the initial condition towards a less negative value during the wetting process, then moved towards the initial condition again during the drying process. Due to the fact that the applied rainfall was smaller than the saturated permeability of the fine-grained layer, the pore-water pressure heads in the fine-grained layer never reached a positive value. Ponding of water on the soil surface was not observed during any of the rainfall tests.

There were two typical types of pore-water pressure development during wetting of column CM_FRA_CRA as shown in Figure 6.7 and Figure 6.9. The development of pore-water pressure shown in Figure 6.7 is in the CM_FRA_CRA test for RF1 and

RF2. After the drawdown test finished, line 1 in Figure 6.7 is developed as previously explained in Section 6.3.1. Due to the slow infiltration caused by the long duration and low intensity of the rainfall, the pore-water pressure in the column had time to redistribute within the fine-grained layer. The redistribution caused the profile to show a uniform and linear decrease in the pore-water pressure head throughout the fine-grained layer (line 2 in Figure 6.7). The slow infiltration subsequently shifted the pore-water pressure head to a less negative value until the pore-water pressure head at the interface approached the value of the water-entry head of the coarse-grained layer (Line 3 in Figure 6.7). When the pore-water pressure head at the interface reached the water-entry value of the coarse-grained layer, the capillary barrier started to permit water infiltration into the coarse-grained layer (line 4 in Figure 6.8).

The difference in rainfall intensity for tests CM_FRA_CRA_RF1 and RF2 affected the velocity of pore-water pressure development in the capillary barrier. Figure 6.8 shows the pore-water pressure profiles at the end of the rainfall event for tests CM_FRA_CRA_RF1 and RF2. The pore-water pressure profile at the end of rainfall for test RF1 was similar to line 3 in Figure 6.7 that corresponds to the pore-water pressure head before breakthrough. On the other hand, the pore-water pressure profile at the end of rainfall for test RF2 was similar to line 4 on Figure 6.7 that corresponds to the pore-water pressure heads at breakthrough.

Figure 6.7 Typical pore-water pressure development corresponding to the slow infiltration

Figure 6.8 Comparison of pore-water pressure at the end of rainfall under different rainfall intensities

The other type of pore-water pressure profile observed from the laboratory column tests was shown by tests CM_FRA_CRA_RF3 and CM_FRC_CRC. When the rainfall started, the pore-water pressure developed from the initial condition (line 1 in Figure 6.9) to line 2 in the same figure. A distinct wetting front marked by the pore-water pressure head developed during infiltration. As the wetting front reached a deeper column depth, the change in pore-water pressure also occurred deeper in the column. However, as the wetting front moved towards the interface, the maximum pore-water pressure head never reached a positive value. The increase in the pore-water pressure head only reached a value of h_q . This value corresponded to the rainfall intensity and also the coefficient of permeability of the material involved. As the infiltration continued, the wetting front reached the interface and the pore-water pressure head at the interface increased to the water-entry value of the coarse-grained layer. During this process, redistribution occurred and the pore-water pressure head in the fine-grained layer started to follow a unit slope profile from the interface to the value of h_q .

Figure 6.9 Typical development of the pore-water pressure profile under fast infiltration

The pore-water pressure developed during the 1-D infiltration process followed Figure 6.7 or Figure 6.9 depending on the rainfall intensity, initial condition and coefficient of permeability of the materials inside the column. In this study, the lower permeability of FRC and the high rainfall intensity of RF3 from the CM_FRA_CRA column caused the development of the pore-water pressure to follow Figure 6.9. The pore-water pressure profiles show that the capillary barriers constructed in this study can create a barrier effect. The barrier effect was demonstrated when the pore-water pressure head in the fine-grained layer moved towards a positive value, while the pore-water pressure head in the coarse-grained layer remained constant. The capillary barrier effect can also be proved by observing the volumetric water content changes.

The volumetric water content changes obtained from the capillary barrier tested in this study were compared to the volumetric water content changes obtained from an assumed single layer barrier. The changes in volumetric water content in the assumed single layer barrier due to a steady-state rainfall condition can be calculated by Kisch's (1959) method. The assumed single layer barrier has a similar height (1 m) and boundary conditions as the test RF3 of column CM_FRA_CRA and CM_FRC_CRC. The volumetric water content changes at the initial and steady-state condition for the single layer barriers of FRA and FRC can be seen in Figure

6.10 and Figure 6.11, respectively. The volumetric water contents obtained from the CM_FRA_CRA_RF3 and CM_FRC_CRC_RF3 laboratory experiments were also placed in those figures. The water storage changes shown in Figure 6.10 indicate that the capillary barrier of CM_FRA_CRA can store significantly more water compared to a theoretical value of a single layer FRA. Similarly, as shown in Figure 6.11, the water content changes obtained from the CM_FRC_CRC laboratory experiments were significantly larger than the calculated theoretical values from a single layer FRC.

The volumetric water content changes shown in Figure 6.10 and Figure 6.11 also show that the capillary barrier constructed from FRA_CRA stored slightly more water than FRC_CRC. This is confirmed by the water balance comparisons obtained from columns CM_FRA_CRA and CM_FRC_CRC for RF2 and RF3 shown in Figure 6.12 and Figure 6.13, respectively. The water balance graphs confirm that column CM_FRA_CRA stored more water before breakthrough. The high water storage of column CM_FRA_CRA was caused by the higher water storage capacity of FRA. The high water storage capacity of FRA is also explained in Section 6.2.3 on the desired capillary barrier material.

Figure 6.10 Comparison of water storage of a single layer of FRA and FRA_CRA capillary barrier

Figure 6.11 Comparison of water storage of a single layer of FRC and FRC_CRC capillary barrier

Figure 6.12 Comparison of water balance curves obtained from test RF2 of columns CM_FRA_CRC and CM_FRC_CRC

Figure 6.13 Comparison of water balance curves obtained from test RF3 of columns CM_FRA_CRA and CM_FRC_CRC

6.3.3 Evaporation characteristics of a capillary barrier constructed from RA

The evaporation characteristics of a capillary barrier constructed from RA are discussed by comparing the results of CM_FRA_CRA_RF2 (RF2-D2) and CM_FRA_CRA_RF5 (RF5-EV2). The pore-water pressure heads during the RF2-D2 and RF5-EV2 tests are presented in Figure 6.14. The pore-water pressure head for the RF5 test at initial conditions was slightly more negative due to the previous EV1 test. Similar movement trends of the linear lines in RF2 were noticed in the pore-water pressure development of RF5. However, the pore-water pressure head values that developed in RF5 were slightly different than those of RF2. During drying, the pore-water pressure head readings along the fine-grained layer materials of RF5-EV2 were considerably more negative than those of RF2-D2, especially near the surface. The pore-water pressure head readings near the interface were also shifted to more negative values. For instance, the pore-water pressure head at the interface for R2-D2 at 48 hours after rainfall was -0.35 m, while the pore-water pressure head at a similar time at the interface of R5-EV2 was -0.45 m. This

indicates that evaporation was affecting the soil column at a deeper depth, not only at the surface of the column.

The shift in the pore-water pressure head due to evaporation was greater at the top of the fine-grained layer, but then became lower at the bottom of the fine-grained layer. This could be caused by the different evaporation rates that occurred at different depths of the fine-grained layer corresponding to the different energy levels from the light bulb radiation. It could also be caused by the decrease in permeability in the top portion of the fine-grained layer, which restricted the movement of water from the lower depths of the fine-grained layer.

Very little shift in the pore-water pressure head occurred in the coarse-grained layer of the capillary barrier. This could be because the coarse-grained layer was already at its ultimate negative pore-water pressure. When the pore-water pressure of the coarse-grained layer reached its residual pore-water pressure, the permeability became very low and the movement of water became very small.

Figure 6.14 Comparison of pore-water pressure head from tests RF2-D2 and RF5-EV2 of column CM_FRA_CRA.

The water balance curves during the RF2-D2 and RF5-EV2 tests are presented in Figure 6.15. The water balance graphs are not entirely complete since the amount of actual evaporation was not measured. The results show that RF5, which was followed by EV2, experienced breakthrough at a relatively similar time as RF2. The amount of water stored at the breakthrough for the RF5-EV2 test was slightly higher than for the RF2-D2 test. The slight increase in water storage might be caused by the fact that evaporation increased the negative pore-water pressure head along the column, as shown by the previous pore-water pressure head graphs. Nevertheless, due to the small rate of evaporation applied to the surface of the column, the effect of evaporative flux in this study was relatively small. An increase in storage of around 7% was observed in the RF5-EV2 and RF6-EV3 tests compared to the RF2-D2 and RF3-D3 tests.

Figure 6.15 Comparison of water balance graphs obtained from tests RF2-D2 and RF5-EV2 of column CM_FRA_CRA

The relatively small effect of evaporative flux could be related to the water availability in the fine-grained layer as explained by the three stages of evaporation (Hillel, 1980). When the precipitation started, the pore-water pressure head in the fine-grained layer moved towards a positive value and then quickly returned to its initial condition due to the drainage process. This caused the pore-water pressure head condition in the column to already be around stage II of the Hillel (1980)

curve (Figure 3.5). Stage II of the evaporation process is when the coefficient of permeability of the soil no longer permits a sufficient flow of water to the surface. The relatively small effect of evaporation was also observed by McCartney and Zornberg (2010). Their study showed that the amount of water in the soil did not decrease significantly during the evaporation stages. The results of this study did not show that evaporation accelerates the removal of water, which contrasts with the results of Krisdani et al. (2005). The different effects of evaporation could be caused by the different materials used in the studies. The use of sandy material (FRA) in this study, as compared to the clayey residual soil used by Krisdani et al. (2005), indicates that water may be lost easily due to drainage in sandy soils, therefore limiting the effects of evaporation. The effects of evaporation on the performance of capillary barrier are also presented in Harnas et al. (2014).

6.4 2D-BEHAVIOR OF CAPILLARY BARRIER MADE FROM RA

The typical pore-water pressure head and volumetric water content changes during rainfall and drainage for a capillary barrier made from RA are shown in Figure 6.16. The initial condition of the pore-water pressure head for a certain test was typically preceded by a drawdown test or a rainfall test. Similar to the 1-D behavior, the drainage from previous tests set the pore-water pressure inside the box to a static non-equilibrium condition. Therefore, the initial conditions of the pore-water pressure head at the measurement points corresponded with the distance from the water table to each measurement point (distance "a" in Figure 6.16). When the box was tilted to a higher slope, the distance between the water table and the measurement points was greater, especially at the measurement points near the crest. Therefore, the initial condition of the pore-water pressure heads with a high slope angle varied more compared to the pore-water pressure heads at the initial condition at a low slope angle. However, if distance "a" was greater than the residual pore-water pressure head of the coarse-grained layer, the pore-water pressure head in the coarse-grained layer above the residual pore-water pressure head remained constant at a value of residual matric suction head. This caused the initial conditions at the different fine-grained layer measurement points (crest to

toe) to be similar if the residual pore-water pressure head of the coarse-grained layer was reached during the drawdown test or the previous rainfall test.

Figure 6.16 Typical pore-water pressure and volumetric water content changes of a capillary barrier during rainfall and drainage

When the rainfall started, the pore-water pressure in the fine-grained layer increased to a more positive value (b in Figure 6.16). The increase corresponded to the rainfall intensity and slope angle. A higher rainfall intensity and slope angle increased the changes in the pore-water pressure head during rainfall. An increase in the slope angle changed the initial condition of the pore-water pressure head to be more negative, which may have caused a larger increase in the pore-water pressure head during the rainfall. After the rainfall stopped, the pore-water pressure heads immediately decreased (c in Figure 6.16). The decrease in the pore-water pressure head was affected by the slope angle and the position of the instrumentation. A higher slope angle increased the velocity of the drainage process compared to a lower slope angle in a capillary barrier, whereas the pore-water pressure head of soil near the toe decreased faster than at the crest of the infiltration box. If the water percolated into the coarse-grained layer, the pore-water pressure head in the coarse-grained layer shifted to a less negative value. Percolation to the coarse-grained layer typically occurred sometime after the rainfall started. Some of the measurements in the coarse-grained layer of the capillary barrier did not capture any changes in the pore-water pressure heads even though water drainage from drain outlet on the coarse-grained layer was observed. This could be because the velocity of the water

flow at the measurement points was too high or because a preferential flow, such as a fingering flow, took place. The changes in volumetric water content followed the changes in pore-water pressure head.

The performance of the capillary barrier constructed from FRA and CRA could be determined from the water balance curves. The effect of thickness, slope angle and rainfall intensity were investigated from the amount of lateral diversion and deep percolation. A summary of the lateral diversion percentages for different tests of the capillary barrier made from RA is presented in Table 6.4. The lateral diversion percentage was defined as the amount of drained water collected from the fine-grained layer of the capillary barrier divided by the total amount of precipitation. The lateral diversion percentage was always higher when the rainfall intensity was lower. This is shown by the higher percentage of lateral diversion on every RF1 test (10 mm/hr) compared to that of the RF2 test (20 mm/hr). This relationship follows the equation proposed by Ross (1990), showing that rainfall intensity has an inverse relationship with the lateral diversion length.

Table 6.4 Summary of lateral diversion percentages from the infiltration box

Test	Lateral diversion (%)
BM_SCB15_2deg_RF1	42
BM_SCB15_2deg_RF2	32
BM_SCB15_5deg_RF1	61
BM_SCB15_5deg_RF2	46
BM_SCB30_2deg_RF1	53
BM_SCB30_2deg_RF2	41
BM_SCB30_5deg_RF1	92
BM_SCB30_5deg_RF2	88
BM_SCB30_7deg_RF1	100
BM_SCB30_7deg_RF2	100

The increase in slope angle from 2° to 5° increased the lateral diversion percentage of the capillary barrier. Increases around 20% and 40% were observed when the slope angle was increased from 2° to 5° for test BM_SCB_15 and BM_SCB30, respectively. The lateral diversion percentage results also show that increasing the

thickness of the capillary barrier increased the lateral diversion percentage. When the thickness of the fine-grained layer was increased from 15 to 30 cm, the lateral diversion increased by 10-12% for an SCB with a 2° slope angle and increased by 35-40% when the slope angle was 5°. There appeared to be a synergistic effect when the slope and thickness of the capillary barrier were increased simultaneously. The increase in lateral diversion when the slope increased from 2° to 5° where the thickness was 30 cm was significantly higher than when the thickness was 15 cm.

6.5 BEHAVIOR OF DCB

6.5.1 1-D behavior and increase in water storage

The results of the drawdown tests for the DCB column reached a similar condition as SCB, which is commonly known as the static non-equilibrium condition (Barbour and Yanful, 1994; Choo and Yanful, 2000; Yang et al., 2004). This condition became the initial condition for the subsequent rainfall tests. The typical behavior of a DCB during infiltration and drainage can be observed from test RF3 (CM_DCB_FRA_CRA_RF3) shown in Figure 5.46. The rainfall initially increased the pore-water pressure head in the upper fine-grained layer. A linear 45° line (unit gradient profile) was observed in the upper fine-grained layer, which was similar to the behavior of SCB. When the water broke through the interface between the upper fine-grained layer and the upper coarse-grained layer, the pore-water pressure head in the upper coarse-grained layer moved to a value close to the water-entry value of the coarse-grained layer. The water then moved to the lower fine-grained layer where a similar profile to the upper and lower coarse-grained layers was observed.

The changes in pore-water pressure head alone cannot show the advantage of DCB compared to SCB. However, the water balance curves can show the effectiveness of DCB in terms of volume of water diverted. For the third rainfall tests (CM_FRA_CRA(SCB)_RF3) and CM_DCB_FRA_CRA(DCB)_RF3, the weighing balance in both the SCB and DCB columns registered a drainage value. The drainage in SCB started about 30 minutes after the start of the rainfall, whereas the

drainage in DCB started about 55 minutes after the start of the rainfall. The comparison of the water balance graphs of SCB and DCB for RF3 is shown in Figure 6.17. The water balance graphs indicate that drainage in DCB was slower than in SCB. The slow drainage in the 1-D condition is an advantage to a cover system as it indicates that water remains in the cover system and can be removed by other mechanisms such as lateral drainage in 2-D condition or evapotranspiration. The slow drainage could be attributed to the change in texture or the effect of layered soil, as previously reported by Zettl et al. (2011) and Huang et al. (2011).

Figure 6.17 Comparison of water balance graphs obtained from the CM_FRA_CRA_RF3 and CM_DCB_FRA_CRA_RF3 tests

Zettl et al. (2011) performed infiltration tests of seven natural coarse-textured sites within Canada and found that the water storage capacity in sites with textural heterogeneity and layering in the soil profile was higher than the estimated water storage capacity. The increase in water storage was caused by the slower drainage occurring in the layered soil. In the case of a fine-grained layer over a coarse-grained layer, a capillary barrier was created. In the case of a coarse-grained layer over a fine-grained layer, the lower coefficient of permeability of the fine-grained layer may result in slower drainage of the overlying coarse-grained layer. Similar to the field results of Zettl et al. (2011), the slower drainage in the DCB column

compared to the SCB column resulted in higher water storage in the DCB column before breakthrough, as indicated by the volumetric water content profile comparison shown in Figure 6.18.

Figure 6.18 Comparison of volumetric water content changes obtained from the CM_FRA_CRA_RF3 and CM_DCB_FRA_CRA_RF3 tests

The higher water storage of DCB compared to SCB could also be attributed to the thinner fine-grained layers in the DCB column. Profiles of the negative pore-water pressure in the fine-grained layer during drainage or breakthrough for both the SCB and DCB columns are shown in Figure 6.19. The pore-water pressure profile during breakthrough in the fine-grained layers of the SCB and DCB columns is shown as a 45° linear line, indicating a unit gradient pore-water pressure profile. The pore-water pressure at the bottom boundary of the fine-grained layers is equal to the water-entry value of the coarse-grained layer, while the pore-water pressure at the top boundary of the fine-grained layer is equal to the sum of the thicknesses of the fine-grained layer and the pore-water pressure at the bottom boundary of the fine-grained layer (Stormont and Morris, 1998). In order to have maximum water storage, the fine-grained layer of a capillary barrier should be at its highest degree of saturation or water content. This can be achieved by having the value of pore-water pressure on the top boundary of the fine-grained layer to be equal or close to the air-entry value of the fine-grained layer. In this study, the FRA air-entry value

(head) was around 0.8 kPa (8 cm). Therefore, the lower thickness of the DCB fine-grained layers (25 cm) compared to the SCB fine-grained layers (50 cm) produced a condition that was more favorable in terms of water storage for DCB, as shown by the typical VWC profile for SCB and DCB during breakthrough as presented in Figure 6.20 This explanation has also been presented in Harnas et al. (2014).

Figure 6.19 Pore-water pressure profile at initial and breakthrough conditions for SCB and DCB

Figure 6.20 Typical volumetric water content profile at initial and breakthrough conditions for SCB and DCB.

6.5.2 2-D behavior and increase in lateral diversion capacity

The typical pore-water pressure and volumetric water content changes of DCB during infiltration and drainage are shown in Figure 6.21. The position of the tensiometers can be identified by the initial conditions of the pore-water pressure heads. The tensiometer with a more negative pore-water pressure head was located in the upper fine-grained layer, while the tensiometer with a less negative pore-water pressure head was located in the bottom coarse-grained layer. However, the arrangement was not the same for the volumetric water content. The volumetric water content of the upper and lower coarse-grained layers was low at around 0.05, while the volumetric water content of the upper and lower fine-grained layers was much higher at around 0.2-0.3. The initial volumetric water content of the upper fine-grained layer was lower than that of the lower fine-grained layer because the pore-water pressure head in the upper fine-grained layer was more negative than that of the lower fine-grained layer.

During the rainfall, the tensiometers in the upper fine-grained and coarse-grained layers instantaneously recorded an increase in the pore-water pressure head. Similar to the behavior of SCB, the increase in the pore-water pressure head was affected by the rainfall intensity. The volumetric water content in the upper fine-grained layer changed after the change in the pore-water pressure head. It was found that changes in the volumetric water content in the upper coarse-grained layer were not recorded by the moisture sensors, which could be caused by the preferential flow occurring in the coarse-grained layer or by the high velocity of water causing the moisture sensors to not record any increase in the volumetric water content. The increase in the pore-water pressure head in the lower fine-grained and coarse-grained layers typically occurred sometime after the rainfall started. If water was laterally diverted by the upper fine-grained and coarse-grained layers, there was no increase in the pore-water pressure head and volumetric water content in the lower fine-grained and coarse-grained layers.

Figure 6.21 Typical pore-water pressure and volumetric content changes of DCB during infiltration and drainage

An illustration of the flow pattern of the DCB test can be seen in Figure 6.22. The flow pattern shows that several paths were taken during the rainfall in the DCB system. The first pattern was that water was laterally diverted in the upper-fine-grained layer. The second pattern was that water was laterally diverted, broke through the upper coarse-grained layer and was subsequently laterally diverted to the drain in the bottom fine-grained layer. The third pattern was the water broke through all the layers and was eventually diverted to the drain in the coarse-grained layer or the underlying foundation layer. From the flow pattern shown in Figure 6.22, the lateral diversion was only recorded from the drain located in the upper and lower fine-grained layers. The weighing balance never recorded any drainage from the upper coarse-grained layer during the infiltration test. This could be due to the boundary condition effect caused by the void and discontinuity of the big particles of the coarse-grained layer materials.

The amount of lateral diversion in DCB with a 2° slope angle was between 75-82% of the rainfall amount, while the amount of lateral diversion in a DCB with a 5° slope angle was 100% of the rainfall amount. The values of lateral diversion in DCB are summarized in Table 6.5. The lateral diversion percentage results show

that DCB was more effective than SCB in diverting water laterally. The reason for DCB to perform better than SCB in diverting water laterally is related to the amount of water that the DCB can store, which is explained in Section 6.5.2.

Figure 6.22 Flow pattern illustration of a DCB

Table 6.5 Lateral diversion percentages from the BM_DCB infiltration box test

Test	Lateral diversion (%)
BM_DCB_2deg_RF1	82
BM_DCB_2deg_RF2	75
BM_DCB_5deg_RF1	100
BM_DCB_5deg_RF1	100

6.6 NUMERICAL MODELING OF SCB AND DCB

A good agreement between the numerical results and experimental data was observed. The pore-water pressure and volumetric water content changes trend of movement obtained from the numerical modeling are generally similar to the experimental results, while the exact value at the measurement points sometimes deviated from the experimental data. This deviation could be caused by inaccuracies in the WCC and the permeability function used. In this study, the drying SWCC and permeability functions were used in the numerical modeling because the drying

properties gave better results than the wetting properties. The poor agreement between the results of numerical modeling and laboratory experiments using the wetting properties may be caused by the different initial conditions of the wetting WCC. The capillary rise tube used for measuring the wetting WCC started from a dry condition, whereas in the infiltration box and column test, the wetting process started from a certain value of water content or pore-water pressure head. This could create different wetting paths that may cause inaccuracies when using the wetting WCC. The current numerical modeling software does not differentiate wetting/drying properties in its input; hence only one set of data - either wetting or drying - can be used. Improvements in accommodating both wetting and drying properties in a flux boundary problem are required.

The water balance curves of the numerical modeling also show agreement with the results of the laboratory experiments. Figure 5.100 shows a comparison of the lateral diversion percentages obtained from the different tests performed. The results show that the maximum error (the difference between the results obtained from the numerical model and the laboratory experimental results) was around 10%. This indicates that the results of the numerical modeling are reliable for the design of a cover system.

An increase in the thickness of the coarse-grained layer increased the normalized water storage and lateral diversion capacity of SCB. This is because the pore-water pressure head in the interface decreased to a more negative value when the thickness of the coarse-grained layer was increased. The increase in thickness decreased the pore-water pressure head in the fine-grained layer, which resulted in more water storage and lateral diversion capacity. However, when the thickness of the coarse-grained layer was increased more than the coarse-grained layer residual matric-suction head, the increase did not decrease the pore-water pressure head any further. Hence, the increase in water storage became very small. This was similar for DCB. Both the upper and lower coarse-grained layers increased the water storage up to a certain value, after which there was no increase in the water storage and lateral diversion capacity with an increase in coarse-grained layer thickness.

Increasing the thickness of the fine-grained layers is one method that can increase the water storage and lateral diversion of a capillary barrier. However, the parametric study results show there is a threshold limit, and increasing the thickness of the fine-grained layers beyond this limit will not significantly increase the water storage and lateral diversion capacity. Stormont and Morris (1998) explained that a unit gradient pore-water pressure head profile was developed in the fine-grained layer of a capillary barrier just before breakthrough. When the thickness of the fine-grained layer was increased more than the air-entry value head of the fine-grained layer, the volumetric water content corresponding to the unit slope pore-water pressure head profile was not at its optimum. The fine-grained layer can store maximum water when it is fully saturated, and this only occurs when the thickness of the fine-grained layer is less than the air-entry value of the fine-grained layer material.

The parametric study results on the effect of layer thickness in DCB show that the fine-grained layer thickness significantly affects the water storage and lateral diversion capacity. The upper fine-grained layer thickness is the most influential layer because, due to the position of the layer relative to the ground water table, the initial volumetric water content for the upper fine-grained layer is lower. Hence, the water storage (volumetric water content at breakthrough - initial volumetric water content) of the upper fine-grained layer is higher than the water storage of the lower fine-grained layer despite having the same volumetric water content profile at breakthrough.

The effect of model length on the lateral diversion capacity percentage and lateral diversion length of SCB are shown in Figure 5.103. The figure shows that if the numerical model length is extended from the current 2 m, the lateral diversion length increases. The increase is caused by the increase in negative pore-water pressure in the fine-grained layer at initial condition due to the increase in distance to the groundwater table. The increase in distance to the groundwater table will also increase the length of lateral diversion or lateral diversion capacity percentage of DCB due to the increase negative pore-water pressure in the fine-grained layers.

6.7 DESIGN OF DCB

The previous sections have shown the important factors that affect the performance of DCB. Those factors are summarized in this section on the design of DCB. Khire et al. (2000) summarized the methods for designing SCB. The first step is determining the critical meteorological period. In this study, the critical meteorological period was represented by the use of “design rainfall” obtained from the rainfall intensity-duration-frequency (IDF) curve developed by PUB (1999). The study on the effect of evaporation on the performance of a capillary barrier (Section 6.3.3) showed that, even though evaporation shifted the pore-water pressure to a more negative position, the change in water balance was minimal. Hence, the use of the rainfall IDF curve as the basis for the design without accounting for evaporation was justified.

The second step of the design of a capillary barrier is layer thickness estimation. The thickness of the coarse-grained layer of DCB can be limited to the residual pore-water pressure head of the coarse-grained layer without reducing the performance, as explained in Section 6.3.1. The material for the coarse-grained layer must have a low water-entry value in order to achieve higher water storage in the fine-grained layer, as explained in Section 6.2.2.

The selected design rainfall from the IDF curve corresponds to the value of the required water storage (s_r). This water storage value needs to be fulfilled by the DCB fine-grained layers. Stormont and Morris (1998) showed that a unit gradient profile occurred just before breakthrough of a capillary barrier. Similarly, this study showed that a unit gradient profile also occurred before breakthrough in DCB. The pore-water pressure profile just before breakthrough in DCB can be seen in Figure 6.23. The soil's maximum water storage in the fine-grained layers can then be calculated as:

$$S_{DCB \max} = \int_0^{b_1} \theta_w(z + h_w) dz + \int_0^{b_2} \theta_w(z + h_w) dz \quad (6.1)$$

where b_1 and b_2 are the thickness of the lower and upper fine-grained layers, respectively. Because the thickness of b_1 and b_2 are the same, the equation becomes:

$$S_{DCB \max} = 2 \int_0^b \theta_w (z + h_w) dz \quad (6.2)$$

$S_{DCB \max}$ is the maximum storage that the fine-grained layers of DCB can accommodate. The water storage of DCB needs to account for the initial volumetric water content. If a hydrostatic condition is assumed as the initial condition, the water storage of DCB can be calculated as:

$$S_{DCB} = 2 \left[\int_0^b \theta_w (z + h_w) dz - \int_0^b \theta_w (z + a) dz \right] \quad (6.3)$$

where a is the thickness of the coarse-grained layer. The water storage of DCB (S_{DCB}) must be equal to or greater than the required water storage (s_r). Hence, the thickness of the fine-grained layer (b) can be estimated from Equation 6.3.

The design of DCB based on water storage is a conservative method. Therefore, water balance simulations should be performed to adjust the thickness of the DCB. The results of the numerical modeling of DCB in this study show that the water balance curves obtained from numerical modeling were comparable to the experimental data (Section 6.6).

Figure 6.23 Pore-water pressure profiles of DCB at breakthrough conditions

CHAPTER 7

CONCLUSIONS & RECOMMENDATIONS

7.1 INTRODUCTION

This chapter provides the conclusions obtained from this study. Investigating the potential use of RA and RC as materials for capillary barrier construction and the modification of capillary barriers to increase lateral diversion capacity were the two main objectives. In this study, a dual capillary barrier (DCB) system constructed using RA and RC was proposed to increase lateral diversion capacity of capillary barriers. The conclusions are presented in two sub-chapters according to the two objectives. Recommendations for future research are presented afterwards.

7.2 THE USE OF RA AND RC AS MATERIALS FOR CAPILLARY BARRIER CONSTRUCTION

The conclusions from the study on the use of RA and RC as materials for capillary barrier construction are as follows:

1. FRA and FRC can be categorized as well-graded sand, while CRA and CRC can be categorized as poorly-graded gravel. The theoretical criterion of fine migration was satisfied by the combinations of fine and coarse RA and RC used in this study.
2. The recycled asphalt used in this study can be categorized as slightly to severely water-repellent. The implication of the water repellency of RA was that the saturated permeability value from the empirical equations (Hazen, 1892 and Kozeny & Carman, 1956) underestimated the coefficient of the saturated permeability of RA. In contrast to RA, the empirical equations overestimated the saturated coefficient permeability of RC. This may have been caused by the cement coating of RC. Traces of cementation in RC after the samples underwent several wet-dry cycles was observed. However, the saturated coefficient of permeability of RA after several wet-dry cycles remained constant.

3. The results of the laboratory column tests and numerical modeling showed there are two desired characteristics of materials used for capillary barrier construction. The first characteristic is that the coarse-grained layer material should have a low water-entry value. The second characteristic is that the fine-grained layer material should have a high water storage capacity.
4. The 1-D laboratory column test and the 2-D infiltration box test results showed that RA and RC can be used as materials for constructing capillary barriers. A significant increase in water storage was obtained when a capillary barrier was used compared to a single layer cover.
5. The 1-D laboratory column test and numerical modeling results showed that, due to the low residual volumetric water content and low air-entry value of FRA, the fine-grained layer of the capillary barrier constructed from FRA stored more water before breakthrough than the capillary barrier constructed from FRC.

7.3 INCREASING WATER STORAGE AND LATERAL DIVERSION OF A CAPILLARY BARRIER

The conclusions from the investigation of the water storage and lateral diversion of SCB and DCB systems are as follows:

1. The results of the drawdown test of the laboratory column test and infiltration box, along with the parametric study, showed that the thickness of the coarse-grained layer in SCB and DCB systems can be limited to the residual matric suction head of the coarse-grained layer. When the thickness of the coarse-grained layer was increased beyond the residual matric suction head of the coarse-grained layer, only a minimal increase in the performance was recorded.
2. Evaporative flux affected the pore-water pressure head in the capillary barrier. A more negative initial condition of the pore-water pressure head was achieved with the application of evaporative flux. As a result, the initial condition enabled the column to store more water during rainfall before breakthrough occurred. Nevertheless, due to the small rate of evaporation applied to the surface of the column and the low water availability in the

fine-grained layer, the effect of evaporative flux in this study was relatively small.

3. The increase in slope angle and thickness of the fine-grained layer increased the lateral diversion capacity of the single capillary barrier. Increasing the thickness of the fine-grained layer was one method that increased the water storage and lateral diversion of the capillary barrier. However, the parametric study results showed there was a threshold limit beyond which increasing the thickness of the fine-grained layer barrier did not significantly increase the water storage and lateral diversion capacity. The suggested optimum thickness of the fine-grained layer is the thickness that corresponds to the air-entry value of fine-grained layer materials.
4. A comparison of the volumetric water content profiles and water balance graphs of the laboratory column test results showed that DCB stores more water than SCB. The high water storage of DCB was caused by effects of soil layering and the lower thickness of the fine-grained layers. The soil layering and lower thickness of the fine-grained layers created an initial conditions of lower volumetric water content values and a higher volumetric water content value before the occurrence of breakthrough compared to SCB.
5. The laboratory infiltration box test results show that DCB can achieve a significantly higher lateral diversion capacity than SCB for the same slope angle and rainfall intensity.
6. The study on lateral diversion capacity of SCB and DCB was performed using a model with a length of 2 m. An increase in the model length can change the measured performance of SCB and DCB due to the change in the initial pore-water pressure condition caused by the increased in distance between the cover system and groundwater table).

7.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Studies recommended for future research related to the use of RA and RC as materials for capillary barrier construction and the design of DCB are as follows:

1. The RA and RC used in this study were obtained from a recycling plant where processing was conducted to ensure uniformity. However, as the properties of RA and RC are dependent on the source of the waste materials, the variability in RA and RC can be large and needs to be further investigated.
2. Further study on the applications of hysteretic models in numerical seepage analyses needs to be encouraged in order to obtain better numerical modeling results
3. In this study, DCB was found to laterally divert water better than SCB. The performance of DCB in other functions of soil covers, such as restricting gas emissions, requires further study.
4. A full-scale field experiment on DCB with comprehensive instrumentation and monitoring systems is recommended in order to observe the performance of DCB in the field.

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APPENDIX A

Numerical modeling details of infiltration box test

Numerical modeling step details.

1. Settings for all model.

a. Material properties.

The SWCC for all models used the one obtain from laboratory measurement showed in Chapter 5. The SWCC used in the study is shown in Chapter 5 . For input to SVFlux program, Fredlund & Xing (1994) fitting parameters can be entered. The fitting parameters are shown in Table A1. The permeability function can be automatically generated by SVFlux by choosing an option of Fredlund & Xing (1994) estimation in the SVFlux program. No anisotropy in permeability was applied. The permeability functions used in the study are shown in Chapter 5.

Table A.1 Fitting parameters of materials used in the study.

No	Description	Symbol	Materials					
			FRC	FRA	CRC	CRA	Geosynthetic	Concrete sand
1	Saturated volumetric water content	θ_s	0.38	0.34	0.45	0.38	0.843	0.411
2	Residual volumetric water content	θ_r	0.146	0.106	0.126	0.101	0.0034	0.03
3	Air-entry value	ψ_a	4.35	0.79	0.25	0.02	0.421	1.49
4	Fitting parameter	a	15.36	2.62	0.33	0.04	1.504	1.941
5		n	0.95	1.01	4.71	3.16	2.128	6.302
6		m	0.83	0.95	0.69	0.82	5.918	0.868
7		ψ_r (kPa)	331	55	0.89	0.19	1.936	3.94
8	Saturated permeability (m/s)	k_s	1×10^{-4}	5×10^{-3}	1×10^{-1}	5×10^{-1}	1×10^{-1}	1×10^{-2}

b. Model Settings.

General:	System	2D
	Type	Transient
	Units	Metric
	Time	s
	Length	m
Time:	Start time	0

Init. Incr.	1
Max Incr.	1000
End time	172800
Transient Formulation	Mixed (mass conservative)
Other Settings	Default

2. Settings for model BM_SCB15

Geometry

Measurement points

Point	X (2 deg)	Y (2 deg)	X (5 deg)	Y(5 deg)	measurement
a1	0.2	0.082	0.2	0.092	PWP
a2	0.7	0.099	0.7	0.136	PWP
a3	1.2	0.117	1.2	0.18	PWP
a4	1.7	0.134	1.7	0.224	PWP
b1	0.2	0.232	0.2	0.242	PWP
b2	0.7	0.249	0.7	0.286	PWP
b3	1.2	0.267	1.2	0.33	PWP
b4	1.7	0.284	1.7	0.374	PWP
e1	0.3	0.085	0.3	0.101	VWC
e2	0.8	0.103	0.8	0.145	VWC
e3	1.3	0.12	1.3	0.189	VWC
e4	1.8	0.138	1.8	0.232	VWC
f1	0.3	0.235	0.3	0.251	VWC
f2	0.8	0.253	0.8	0.295	VWC
f3	1.3	0.27	1.3	0.339	VWC
f4	1.8	0.288	1.8	0.382	VWC

Flux Sections

Flux 1 x,y (0,0) to (0,1.5) and Flux 2 x,y (0, 0.15) to x,y (0, 0.30)

2a. BM_SCB15_2deg DD

Initial Condition : Water table at the surface of the model

Boundary condition:

2b. BM_SCB15_2 deg RF1

Initial condition: Head condition from BM_SCB15_DD @172800 s

Boundary Condition:

Normal Flux data : 0-3600 s = 2.77×10^{-6} m/s
 3600-172800 s = 0

2c. BM_SCB15 2 deg_RF2

Initial condition: Head condition from BM_SCB15_RF1 @172800 s

Boundary Condition:

Normal Flux data : 0-3600 s = 5.55×10^{-6} m/s
 3600-172800 s = 0

2d. BM_SCB15 5deg

Similar boundary condition are used for 5 degrees models

3. Settings for model BM SCB30

Geometry:

Measurement points;

Point	X (2 deg)	Y (2 deg)	X (5 deg)	Y(5 deg)	measurement
a1	0.2	0.082	0.2	0.092	PWP
a2	0.7	0.099	0.7	0.136	PWP
a3	1.2	0.117	1.2	0.18	PWP
a4	1.7	0.134	1.7	0.224	PWP
b1	0.2	0.232	0.2	0.242	PWP
b2	0.7	0.249	0.7	0.286	PWP
b3	1.2	0.267	1.2	0.33	PWP
b4	1.7	0.284	1.7	0.374	PWP
C1	0.2	0.382	0.2	0.392	PWP
C2	0.7	0.399	0.7	0.436	PWP
C3	1.2	0.417	1.2	0.48	PWP
C4	1.7	0.434	1.7	0.524	PWP
e1	0.3	0.085	0.3	0.101	VWC
e2	0.8	0.103	0.8	0.145	VWC
e3	1.3	0.12	1.3	0.189	VWC
e4	1.8	0.138	1.8	0.232	VWC
f1	0.3	0.235	0.3	0.251	VWC
f2	0.8	0.253	0.8	0.295	VWC
f3	1.3	0.27	1.3	0.339	VWC
f4	1.8	0.288	1.8	0.382	VWC
g1	0.3	0.385	0.3	0.401	VWC
g2	0.8	0.403	0.8	0.445	VWC
g3	1.3	0.420	1.3	0.489	VWC
g4	1.8	0.438	1.8	0.532	VWC

Flux Sections

Flux 1: $x,y (0,0)$ to $(0,1.5)$

Flux 2: $x,y (0, 0.15)$ to $x,y (0, 0.30)$

Flux 3: $x,y (0, 0.30)$ to $x,y (0, 0.45)$.

3a. BM_SCB30_2deg DD

Initial Condition : Water table at the surface of the model

Boundary condition:

3b. BM_SCB30_2 deg RF1

Initial condition: Head condition from BM_SCB30_DD @172800 s

Boundary Condition:

Normal Flux data : 0-3600 s = 2.77×10^{-6} m/s
3600-172800 s = 0

3c. BM_SCB30 2 deg RF2

Initial condition: Head condition from BM_SCB30_RF1 @172800 s

Boundary Condition:

Normal Flux data : 0-3600 s = 5.55×10^{-6} m/s
3600-172800 s = 0

3d. Subsequent models;

Similar boundary conditions to the above models. Just different by slope angle.

4. Settings for model BM DCB

Geometry:

Measurement points;

Point	X (2 deg)	Y (2 deg)	X (5 deg)	Y(5 deg)	measurement
a1	0.2	0.082	0.2	0.092	PWP
a2	0.7	0.099	0.7	0.136	PWP
a3	1.2	0.117	1.2	0.18	PWP
a4	1.7	0.134	1.7	0.224	PWP
b1	0.2	0.232	0.2	0.242	PWP
b2	0.7	0.249	0.7	0.286	PWP
b3	1.2	0.267	1.2	0.33	PWP
b4	1.7	0.284	1.7	0.374	PWP
C1	0.2	0.382	0.2	0.392	PWP
C2	0.7	0.399	0.7	0.436	PWP
C3	1.2	0.417	1.2	0.48	PWP
C4	1.7	0.434	1.7	0.524	PWP
d1	0.2	0.532	0.2	0.55	PWP
d2	0.7	0.549	0.7	0.586	PWP
d3	1.2	0.567	1.2	0.63	PWP
d4	1.7	0.584	1.7	0.67	PWP
e1	0.3	0.085	0.3	0.101	VWC
e2	0.8	0.103	0.8	0.145	VWC
e3	1.3	0.12	1.3	0.189	VWC
e4	1.8	0.138	1.8	0.232	VWC
f1	0.3	0.235	0.3	0.251	VWC

f2	0.8	0.253	0.8	0.295	VWC
f3	1.3	0.27	1.3	0.339	VWC
f4	1.8	0.288	1.8	0.382	VWC
g1	0.3	0.385	0.3	0.401	VWC
g2	0.8	0.403	0.8	0.445	VWC
g3	1.3	0.420	1.3	0.489	VWC
g4	1.8	0.438	1.8	0.532	VWC
h1	0.3	0.535	0.3	0.551	VWC
h2	0.8	0.553	0.8	0.595	VWC
h3	1.3	0.570	1.3	0.639	VWC
h4	1.8	0.588	1.8	0.832	VWC

Flux Sections

Flux 1: x,y (0,0) to (0,1.5)

Flux 2: x,y (0, 0.15) to x,y (0, 0.30)

Flux 3: x,y (0, 0.30) to x,y (0, 0.45).

Flux 4: x,y (0, 0.45) to x,y (0, 0.60).

4a. BM_DCB_2deg DD

Initial Condition : Water table at the surface of the model

Boundary condition:

4b. BM DCB 2 deg RF1

Initial condition: Head condition from BM_DCB_DD @172800 s

Boundary Condition:

4c. BM DCB 2 deg RF2

Initial condition: Head condition from BM_DCB_2deg_RF1 @172800 s

Boundary Condition:

Normal Flux data : 0-3600 s = 5.55×10^{-6} m/s
3600-172800 s = 0

4d. Subsequent models.

Similar boundary conditions to the above models. Just different by slope angle.

APPENDIX B

Calculation of coefficient of saturated permeability
from upward infiltration of 1-D column study

Table B1. Calculation of saturated coefficient of permeability obtained from test CM_FRA_CRA_UI

depth	Pressure head (m)	Elevation head (m)	Total Head (m)	ΔH	ΔL	i	Infiltration rate (m ³ /s)	K (m/s)
0.00	1.52	0.00	1.52	0.000	0.050	0.000	0.000580	
0.05	1.47	0.05	1.52	0.001	0.200	0.004		0.261
0.25	1.27	0.25	1.52	0.000	0.200	0.002		
0.45	1.07	0.45	1.52	0.095	0.100	0.952		0.000777
0.55	0.87	0.55	1.42	0.201	0.175	1.149		
0.73	0.50	0.73	1.22	0.054	0.075	0.726		
0.80	0.37	0.80	1.17	0.135	0.150	0.903		
0.95	0.08	0.95	1.03	0.033	0.050	0.651		
1.00	0.00	1.00	1.00	-	-	-		

Table B2 Calculation of saturated coefficient of permeability obtained from test CM_FRC_CRC_UI

depth	Pressure head (m)	Elevation head (m)	Total Head (m)	ΔH	ΔL	i	Infiltration rate (m ³ /s)	K (m/s)
0	1.52	0.00	1.52	0.000	0.050		0.00058	
0.05	1.47	0.05	1.52	0.000	0.200	0.00		
0.25	1.27	0.25	1.52	0.001	0.200	0.01		0.2368
0.45	1.07	0.45	1.52	0.244	0.100	2.44		
0.55	0.73	0.55	1.28	0.103	0.175	0.59		
0.725	0.45	0.73	1.18	0.135	0.075	1.80		0.0003
0.8	0.24	0.80	1.04	0.006	0.150	0.04		
0.95	0.08	0.95	1.03	0.034	0.050	0.68		
1	0.00	1.00	1.00	-	-	-		

Table B1 Calculation of saturated coefficient of permeability obtained from test CM_DCB_FRA_CRA_UI

depth	Pressure head (m)	Elevation head (m)	Total Head (m)	ΔH	ΔL	i	Infiltration rate (m ³ /s)	K (m/s)
0.00	1.52	0.00	1.52	0.00	0.050	0.00000	0.000640	
0.05	1.47	0.05	1.52	0.01	0.200	0.01937		0.03291
0.25	1.26	0.25	1.51	0.25	0.200	0.03976		0.00050
0.45	0.81	0.45	1.26	0.00	0.100	1.27319		0.11155
0.55	0.71	0.55	1.26	0.00	0.175	0.01320		0.00062
0.73	0.53	0.73	1.25	0.12	0.075	0.00571		
0.80	0.33	0.80	1.13	0.11	0.150	1.65333		
0.95	0.07	0.95	1.02	0.02	0.050	0.73789		
1.00	0.00	1.00	1.00		-	-		

APPENDIX C

Water balance obtained from BM_DCB test

Figure C1 Water balances from the BM_DCB_2eg_RF1

Figure C2 Water balances from the BM_DCB_2eg_RF2

Figure C3 Water balances from the BM_DCB_5deg_RF1

Figure C3 Water balances from the BM_DCB_5deg_RF2

APPENDIX D

Comparisons of experimental and numerical results for
1-D column study

Figure D1. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_1

Figure D2. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_1

Figure D3. Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF1

Figure D4. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_2

Figure B5. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_2

Figure D6 Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF2

Figure D7. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_3

Figure D8. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_3

Figure D9 Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF3

Figure D10. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_4

Figure D.11. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_4

Figure D12 Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF4

Figure D13. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_5

Figure D14. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_5

Figure D15 Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF5

Figure D16. Comparisons of experimental and numerical pore-water pressures for test CM_FRA_CRA_RF_6

Figure D17. Comparisons of experimental and numerical volumetric water contents for test CM_FRA_CRA_RF_6

Figure D18 Comparison of experimental and numerical water balances graph for test CM_FRA_CRA_RF6

Figure D19. Comparisons of experimental and numerical pore-water pressures for test CM_FRC_CRC_RF_1

Figure D20. Comparisons of experimental and numerical volumetric water contents for test CM_FRC_CRC_RF_1

Figure D21. Comparison of experimental and numerical water balances graph for test CM_FRC_CRC_RF1

Figure D22. Comparisons of experimental and numerical pore-water pressures for test CM_FRC_CRC_RF_2

Figure D23. Comparisons of experimental and numerical volumetric water contents for test
CM_FRC_CRC_RF_2

Figure D24. Comparison of experimental and numerical water balances graph for test
CM_FRC_CRC_RF2

Figure D25. Comparisons of experimental and numerical pore-water pressures for test CM_FRC_CRC_RF_3

Figure D26. Comparisons of experimental and numerical volumetric water contents for test CM_FRC_CRC_RF_3

Figure D27 Comparison of experimental and numerical water balances graph for test CM_FRC_CRC_RF3

Figure D28. Comparisons of experimental and numerical pore-water pressures for test CM_DCB_FRA_CRA_RF_1

Figure D29. Comparisons of experimental and numerical volumetric water contents for test CM_DCB_FRA_CRA_RF_1

Figure D30 Comparison of experimental and numerical water balances graph for test CM_DCB_FRA_CRA_RF1

Figure B31. Comparisons of experimental and numerical pore-water pressures for test CM_DCB_FRA_CRA_RF_2

Figure B32. Comparisons of experimental and numerical volumetric water contents for test CM_DCB_FRA_CRA_RF_2

Figure D33 Comparison of experimental and numerical water balances graph for test CM_DCB_FRA_CRA_RF2

Figure B34. Comparisons of experimental and numerical pore-water pressures for test CM_DCB_FRA_CRA_RF_2

Figure B35. Comparisons of experimental and numerical volumetric water contents for test CM_DCB_FRA_CRA_RF_3

Figure D36 Comparison of experimental and numerical water balances graph for test CM_DCB_FRA_CRA_RF3

APPENDIX E

Comparisons of experimental and numerical results for
2-D infiltration box study

Figure E1 Comparisons of experimental and numerical pore-water pressures for the BM_SCB15_RF1_2deg test

Figure E2 Comparisons of experimental and numerical volumetric water content for test BM_SCB15_RF1_2deg

Figure E3 Comparisons of experimental and numerical pore-water pressures for the BM_SCB15_RF2_2deg test

Figure E4 Comparisons of experimental and numerical volumetric water content for test BM_SCB15_RF2_2deg

Figure E5 Comparisons of experimental and numerical pore-water pressures for the BM_SCB15_RF1_5deg test

Figure E6 Comparisons of experimental and numerical volumetric water content for test BM_SCB15_RF1_5deg

Figure E7 Comparisons of experimental and numerical pore-water pressures for the BM_SCB15_RF2_5deg test

Figure E8 Comparisons of experimental and numerical volumetric water content for test BM_SCB15_RF2_5deg

Figure E9 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF1_2deg test

Figure E10 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF1_2deg

Figure E11 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF2_2deg test

Figure E12 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF2_2deg

Figure E13 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF1_5deg test

Figure E14 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF1_5deg

Figure E15 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF2_5deg test

Figure E16 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF2_5deg

Figure E17 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF1_7deg test

Figure E18 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF1_7deg

Figure E19 Comparisons of experimental and numerical pore-water pressures for the BM_SCB30_RF2_7deg test

Figure E20 Comparisons of experimental and numerical volumetric water content for test BM_SCB30_RF2_7deg

Figure E21 Comparisons of experimental and numerical pore-water pressures for the BM_DCB_RF1_2deg test

Figure E22 Comparisons of experimental and numerical volumetric water content for test BM_DCB_RF1_2deg

Figure E23 Comparisons of experimental and numerical pore-water pressures for the BM_DCB_RF2_2deg test

Figure E24 Comparisons of experimental and numerical volumetric water content for test BM_DCB_RF2_2deg

Figure E25 Comparisons of experimental and numerical pore-water pressures for the BM_DCB_RF1_5deg test

Figure E26 Comparisons of experimental and numerical volumetric water content for test BM_DCB_RF1_5deg

Figure E27 Comparisons of experimental and numerical pore-water pressures for the BM_DCB_RF2_5deg test

Figure E28 Comparisons of experimental and numerical volumetric water content for test BM_DCB_RF2_5deg