

CRANFIELD UNIVERSITY

Ziling Zang

Integrated modelling of the clogging processes of plastic grid
permeable pavement

School of Water, Energy, Environment and Agri-food
Water

PhD

Academic Year: 2017 - 2021

Supervisor: Professor Tim Hess

Associate Supervisor: Dr Imma Bortone and Dr Robert Grabowski
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This thesis is submitted in partial fulfilment of the requirements for
the degree of PhD

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ABSTRACT

Because of rapid urban expansion increasing the area of impervious land surfaces and climate change, flood risks and extreme precipitation events are projected to become severe challenges in the future. Sustainable Urban Drainage Systems (SUDS) are designed to increase urban surface permeability and reduce stormwater runoff. As a commonly used SUDS technique, plastic grid permeable (PGP) pavement effectively reduces urban surface runoff. However, clogging is a severe problem that reduces the operational performance and lifespan of PGP. Further research is needed on the clogging process, but it is challenging to conduct field experiments to predict permeable pavement's long-term performance for engineers and researchers. This study aims to evaluate the suspended particle clogging process and mechanisms of PGP using a modelling approach. A 1-D model was developed with COMSOL Multiphysics to understand the clogging process, based on the spatially and temporally mathematical expression. The new integrated hydraulic and clogging model for the PGP system consists of three parts: 1) Hydraulic model, 2) Rainfall-infiltration boundary (RIB) condition and 3) Clogging model, which named as the PGP-HRC model.

The PGP-HRC model focuses on the interaction, feedbacks, and parameter changes between the three parts. The hydraulic model and RIB provide water flow driven force for the clogging model. Over time, clogging reduces the media porosity and further changes the soil properties, affecting the hydraulic model and RIB. By testing the model input parameters with different rainfall intensity, initial soil water content, suspended particle concentration, particle size and duration, the clogging time, depth and mechanisms of the PGP system can be understood.

PGP-HRC model enables detailed study of hydraulic and clogging processes within porous media, and can be adapted for a range of applications. For the mitigation of surface water flooding, it provides a platform to test the design and maintenance of PGP, which will help for climate adaptation strategy and extreme precipitation in the urban area.

Keywords:

Sustainable Drainage Systems; pervious pavement; surface water flooding;
suspended sediment transport; COMSOL Multiphysics

ACKNOWLEDGEMENTS

I would like to acknowledge and give my warmest thanks to my supervisors (Prof Tim Hess, Dr Imma Bortone and Dr Robert Grabowski) who made this work possible. Although they provided supports from different aspects, their guidance and advice carried me through all the stages of my project. I enjoyed every moment of discussion during meetings, and their suggestions and project managing strategy have become the most valuable asset for my career and life.

Also, I appreciate the help from my review panel, and they made every review meeting a friendly discussion. My Reviewer Prof Jane Rickson provided precious ideas, always believed me, and encouraged me in each review meeting. My Chair, Prof Nigel Simms focusing on my progress and made me realised that time was ticking.

Thanks for the financial and mental support from my parents. They undoubtedly support me in pursuing my dreams, comforting me when stress overwhelmed me. I love my parents and my grandparents so much! And their support is definitely the driven force through my PhD.

I also appreciate my boyfriend, Aqeel Shamsul, always standing by my side, cooking good food for me, giving me hug and love. Sometimes that is all I need after a long tiring day.

Thanks to every friend in my life, especially Rossetti Xu and Yanjun Gao, who listened to me and accompanied me through every happy or sad moment in my life.

I am fortunate to have 2 cats, Momo and Dundun. They are such adorable creatures; thanks for them keeping me company and loving me. Holding them every day is my best way to decompress.

Also, I would like to thank every colleague for creating a warm office atmosphere for discussing academic issues and Cranfield University providing a tremendous academic environment. I also thank every friend of Cranfield and the UK CSSA for enriching my life.

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LIST OF ABBREVIATIONS

BDF	Back Differential Formulation
CGP	Concrete Grid Paver
FEH	Flood Estimation Handbook
FEM	Finite element method
GWCC	Geotextile Water Characteristic Curve
LID	Low Impact Development
LIUDD	Low Impact Urban Design and Development
LVF	Liquid volume fraction
MUSIC	Urban Stormwater Improvement Conceptualisation
PA	Porous Asphalt
PC	Porous Concrete
PDE	Partial Differential Equations
PGP	Plastic Grid Paver
PGP-HRC	Developed integrated hydraulic and clogging model for the PGP system
PICP	Permeable Interlocking Concrete Paver
PTF	Pedotransfer function
RIB	Rainfall-infiltration boundary condition
RMSE	Root mean square error
RPC	Relative suspended particle concentration
SPS	Suspended particle size
SS	Suspended sediment
SUDS	Sustainable Urban Drainage System
SUDSLOC	SuDS Selection and Location tool
SWCC	Soil Water Characteristic Curve
SWMM	Storm Water Management Model
TN	Total nitrogen
TP	Total phosphorous
TSS	Total suspended sediment
WSUD	Water Sensitive Urban Design

LIST OF SYMBOLS

<i>Symbol</i>	<i>Description</i>	<i>unit</i>
t	Time	[s]
K	Hydraulic conductivity	[m s ⁻¹]
$K_{sat,t}$	Real-time saturated hydraulic conductivity	[m s ⁻¹]
$K_{sat,0}$	Initial saturated hydraulic conductivity	[m s ⁻¹]
K_s	Saturated hydraulic conductivity	[m s ⁻¹]
K_{sat_s}	Saturated hydraulic conductivity for soil	[m s ⁻¹]
K_{sat_g}	Saturated hydraulic conductivity for geotextile	[m s ⁻¹]
K_{unsat_g}	Unsaturated hydraulic conductivity for soil	[m s ⁻¹]
K_{unsat_s}	Unsaturated hydraulic conductivity for geotextile	[m s ⁻¹]
k_s	Soil permeability	[m ²]
k_r	Relative permeability of the soil	[-]
H	Hydraulic head	[m]
H_p	Pressure head	[m]
p	pressure	[Pa]
α	van-Genuchten parameter	[m ⁻¹]
m, n, l	van-Genuchten parameter	[-]
z	Length or depth along the direction, for a 1D vertical model, z equals to the vertical elevation.	[m]
ψ	Soil water potential	[cm] of water
$ \psi $	Suction pressure	[cm] of water
q	Flux	[m ² s ⁻¹]
g	Acceleration of gravity	[m s ⁻²]
∇p	Pressure drop from point a to b / pressure gradient	[Pa]
W	Specific moisture capacity	[-]
Se	Effective saturation of porous media	[-]
S	Storage coefficient	[Pa ⁻¹]
κ_s	Permeability of soil	[s ⁻¹]
κ_g	Permittivity of geotextile	[s ⁻¹]
θ	Liquid volume fraction	[1]
θ_0	Initial porosity	[1]

θ_s	Porosity/Saturated liquid volume fraction	[1]
θ_e	Effective liquid volume fraction	[1]
θ_r	Residual liquid volume fraction	[1]
θ_i	The initial soil moisture content throughout the whole soil column	[1]
μ	Fluid dynamic viscosity	[Pa·s]
a	Thickness of nonwoven geotextile (under a certain confined pressure)	[m]
a_0	Initial thickness of geotextile without confined pressure	[m]
a_f	Final thickness geotextile after unloaded	[m]
a_s	thickness of the soil layer above the geotextile	[m]
σ	Confined pressure applied on the geotextile	[kPa]
σ_0	Initial confined pressure	[kPa]
σ_f	Final confined pressure	[kPa]
σ_v	Vertical confined pressure	[kPa]
α_a, β_a	Fitting parameters for calculating geotextile thickness	[-]
γ	Soil unit weight	[kN m ⁻³]
γ_{sat}	Saturated soil unit weight	[kN m ⁻³]
γ_d	Dry soil unit weight	[kN m ⁻³]
Q_m	A mass source term	[kg m ⁻³ ·s ⁻¹]
t_p	Ponding time	[s]
q	Darcy flux	[m ² s ⁻¹]
f	Green-Ampt infiltration rate	[m s ⁻¹]
h	head	[m]
h_0	depth of ponded water on surface	[m]
ψ_f	capillary pressure of wetting front	[m]
F	The change of cumulative infiltration	[m]
F_p	Cumulate infiltration	[m]
w	Water input rate at time t	[m s ⁻¹]
ψ_b, r	Constants can be obtained by the fitting curve of S_e and ψ_f from experimental data	[-]
θ_{sat}	Initially saturated moisture content	[1]

θ_{fc}	Initially field capacity moisture content	[1]
θ_{pwp}	Initially permanent wilting point moisture content	[1]
u_{in}	Real-time inlet velocity	[m s ⁻¹]
v_{rain}	rainfall intensity	[m s ⁻¹]
C	Suspended particle concentration	[kg m ⁻³]
C_s	Sedimentary particle concentration	[kg m ⁻³]
L	Column length/depth	[m]
C_0	Suspended particle concentration at a certain depth	[kg m ⁻³]
D	The coordinate components of the hydrodynamic dispersion coefficient tensor	[m ² s ⁻¹]
R	Reaction field	-
ρ_p	Density of Suspended particle	[kg m ⁻³]
ρ_f	Density of fluid (water)	[kg m ⁻³]
ρ_b	Bulk density	[kg m ⁻³]
ε_p	Inside porosity of packed suspended particles	[1]
v	Darcy velocity	[m s ⁻¹]
λ	Filtration coefficient	[m ⁻¹]
λ_g	Filtration coefficient for geotextile	[m ⁻¹]
λ_s	Filtration coefficient for soil/sand	[m ⁻¹]
$\alpha_{collision}$	Collision efficiency factor	[-]
η	Retention ratio/collector efficiency	[-]
η_g	Retention ratio/collector efficiency for geotextile	[-]
η_s	Retention ratio/collector efficiency for soil/sand	[-]
d	Real-time porous media grain diameter	[m]
d_p	Suspended particle size	[m]
d_0	Initial porous media particle diameter	[m]
η_D	Single collector contact efficiency, transport by diffusion	[-]
η_I	Single collector contact efficiency, transport by interception	[-]
η_G	Single collector contact efficiency, gravitational transport	[-]
γ	Single collector contact efficiency variable	[-]

A_s	Porosity-depended parameter	[-]
N_{Pe}	Pelect number	[-]
$D_{\infty}, \mathcal{D}_{AB}$	Diffusion coefficient in an infinite medium	[m ² s ⁻¹]
N_{vdW}	London-van der Waals number	[-]
N_A	attraction number	[-]
N_G	gravity number	[-]
N_R	aspect ratio	[-]
B	Boltzmann's constant	[J K ⁻¹]
T	Absolute temperature	[K]
r_p	Suspended particle radius	[m]
Ham	Hamaker constant	[J]
D_m	mechanical dispersion	[m ² s ⁻¹]
$\mathcal{D}_{bm}$	effective molecular diffusion	[m ² s ⁻¹]
α_L	Dispersivity	[cm]
τ	Tortuosity coefficient	[-]
D_{FEH}	Rainfall duration for FEH	[h]
R_{FEH}	Maximum D _{FEH} -hour rainfall depth for FEH	[mm]
T_{FEH}	Return period of event for FEH	[a]
$c, e, f_{FEH}, d_1, d_2, d_3$	Parameters that have been mapped for the UK for FEH	[-]

1 Chapter One: Introduction

The background, techniques and summary of previous studies of the Sustainable urban Drainage system (SUDS) have been introduced in this Chapter. The overall aim, objectives and thesis structure are also indicated.

1.1 Background

Flood risks are predicted to increase in the future due to climate and land-use change (Evans *et al.*, 2004). Surface water flooding is a significant hazard that is predicted to increase in severity and frequency due to urbanisation and climate change.

1.1.1 Effects of urbanisation

The global population has increased rapidly in urban areas worldwide since the start of the 20th century (Marshall, 2007). Urban expansion leads to increased urban impervious surface areas (e.g. roofs, roads and other surface cover). The natural balance of impermeable-permeable surface area has changed by human activities. Traditional urban drainage systems are getting less effective because of the large amount of increasingly polluted stormwater runoff in a short time (Arnold and Gibbons, 1996; Finkenbine, Atwater and Mavinic, 2001; Brabec, Schulte and Richards, 2002). Traditionally, stormwater in urban areas is rapidly transported away via a pipe network. When surface runoff is generated, the system allows water to drain immediately and freely and discharge to natural water bodies. However, due to the limitation of pipeline peak flow and city sewage treatment capacity, the conventional drainage system is not a flexible solution for adapting to the future increase in large precipitation events.

1.1.2 Effects of climate change

Climate change is a severe challenge for the whole world. The IPCC (2014) indicated a significant increase in risks of storms, extreme precipitation and flooding. These changes will be projected to negatively impact ecosystems, economies, and people's health (IPCC, 2014). Because of the regional variability, it is hard to predict local future climate change in precipitation.

In the United Kingdom, winter rainfall showed an increasing trend in intense long-duration events over the last 50 years; a similar increase in rainfall intensity was also found in long summer precipitation events (Watts *et al.*, 2015). Watts *et al.* (2015) also concluded that future climate change, including higher water temperature, more algae in summer, and less river flow, will significantly decline water quality for rivers and lakes. Total annual precipitation has increased over the last 20 years, with an annual average of 1689.2 mm (2011-2020) compared to 1349 mm (2001-2010) (Jaganmohan, 2021; Figure 1-1). It is also worth noting the changing monthly rainfall. The monthly rainfall in February 2020 is much higher than in previous years. The average monthly rainfall may only increase slightly, and the dramatically raised monthly rainfall indicates increasing extreme weather.

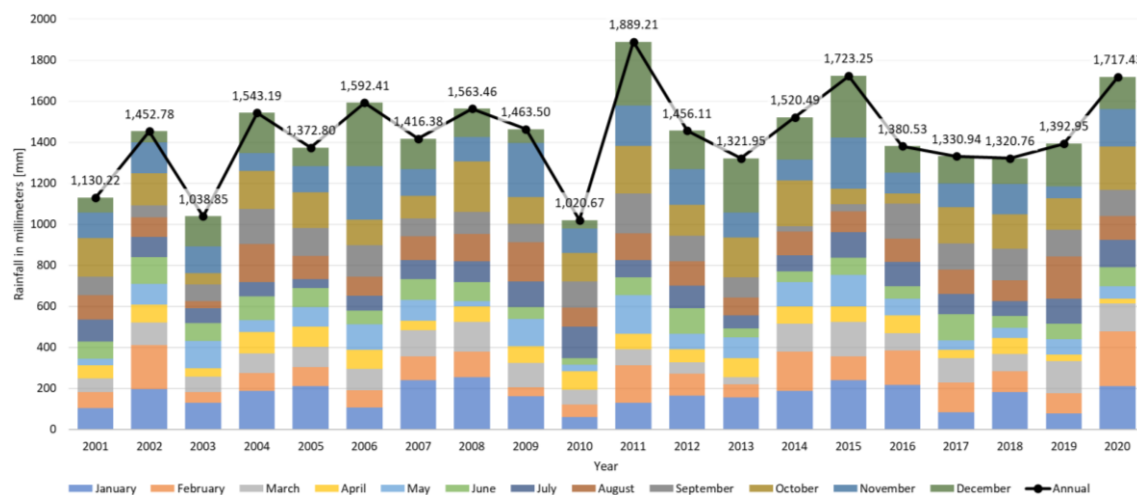


Figure 1-1 Average monthly and annual rainfall in the UK, including rainfall data for 20 years from 2001 to 2020. (Source: Jaganmohan, 2021)

1.1.3 Effects on amenity and biodiversity

The growing global urban development has challenged the amenity and biodiversity of urban landscapes (Vlahov, 2002; Seto, Guneralp and Hutyra, 2012).

The expansion of cities has directly led to biodiversity loss and artificially transformed and homogenised the most biologically abundant landscape on earth (McKinney, 2006; Grimm *et al.*, 2008). Nowadays, people have started to

realise the importance of biodiversity, metropolitan areas are beginning to support biodiversity, and many plants and animals are also actively responding to the urban environments (Schwartz, Jurjavcic and O'Brien, 2002; Aronson *et al.*, 2014).

The impact of urbanisation on human health and well-being is complicated (Ives and Kelly, 2016). On the one hand, numerous social and economic opportunities have been provided by cities; on the other hand, public health and well-being are also challenged by cities, especially in densely populated areas and heavy industrial areas (Vlahov, 2002; Ives and Kelly, 2016).

Facing the above challenges, Sustainable Urban Drainage Systems aim to reduce the water flooding stress in cities and improve amenity and biodiversity by artificial landscape (Ballard *et al.*, 2015).

1.2 Sustainable Urban Drainage System (SUDS)

To support the urban from adapting to future challenges, urban development and drainage systems for new construction must be planned effectively; drainage in existing systems must be well managed and maintained (Evans *et al.*, 2004).

Sustainable urban Drainage Systems (SUDS) are designed to minimise the risks of raising quantity and more polluted urban surface runoff and maximise the amenity and biodiversity opportunities (Ballard *et al.*, 2015). SUDS are designed to combine artificial facilities and natural landscapes while decreasing the probability of urban flooding with a large quantity of highly polluted urban surface runoff (Ballard *et al.*, 2015). SUDS is similar to Low Impact Development (LID) in the USA and Japan, Water Sensitive Urban Design (WSUD) in Australia, Low Impact Urban Design and Development (LIUDD) in New Zealand and “Sponge City” in China.

SUDS includes different techniques for three processes (Table 1-1). SUDS aims to control surface runoff, increase water quality, and improve amenity and biodiversity through source control, permeable conveyance and passive treatment (Ballard *et al.*, 2015; Scholz, 2015). Typically, several SUDS

techniques would be applied and connected and form a runoff treatment chain (Scholz, 2015).

Table 1-1 SUDS techniques categories and the process of surface runoff and water quality control

<i>Process</i>	<i>Source control</i>	<i>Permeable conveyance</i>	<i>Passive treatment</i>
<i>Techniques</i>	Green roofs	Filter drains	Filter strips
	Infiltration basins	Swales	Detention basins
	Infiltration trenches	Bioretention	Retention ponds
	Permeable pavements	-	Wetlands
	Rainwater harvesting	-	-
	Soakaways	-	-

1.3 Permeable pavement

Due to the rapid urbanisation and climate change in recent years, impermeable pavements (including roads and roofs) in urban areas have increased significantly (Ellis, Revitt and Lundy, 2012; Al-Rawas *et al.*, 2015). Pavements typically occupy around 25% of the impervious areas in an urban environment (Ferguson, 2009; Shackel, 2010). A significant challenge for urban flood management is that two-thirds of surface runoff flow in urban areas is generated from impervious pavements (Ferguson, 2009; Shackel, 2010).

1.3.1 The function of permeable pavement

Permeable pavements (i.e. pervious pavements) are a common approach for SUDS source control. Permeable pavement improves urban permeability by changing the traditional pavement structure and materials to increase infiltration into the ground and decrease surface runoff. The hydraulic performance of permeable paving system includes the reduction of peak flow, surface runoff and infiltration capacity. At the same time, a simple underground coarse material filtering structure is used to prevent polluting the groundwater from pollutants

carried in the surface runoff, as well as facilitates drainage (Baladès, Legret and Madiec, 1995; Hunt and Collins, 2007).

A permeable paving system could be used for pedestrian and low vehicular traffic pavement, allowing water to penetrate the surface and infiltrate the underlying layers (Ballard *et al.*, 2015). With careful design, sufficient water quality improvement and carrying load could be provided by aggregate underground layers. Inflow water will be finally infiltrated into the groundwater or collected by the underground pipeline system after travelling through the capillary structure of permeable pavement (Ballard *et al.*, 2015). Permeable pavement can be designed in an urban flood treatment chain with a green roof, infiltration trenches, and other SUDS techniques.

As a source control method in SUDS, there are several benefits of permeable pavement (Barnes, Morgan III and Roberge, 2000; Frazer, 2005; Rowe, Borst and O'Connor, 2008; Sansalone, Kuang and Ranieri, 2008; Beecham *et al.*, 2010; Shackel, 2010; Li, 2012).

- Permeable pavement can reduce the volume of stormwater entering the drainage system, releasing the risk of surface water flooding in an urban area.
- Increase the pervious surface area, reinstate the city's permeable capability, and helps to restore the natural hydrological cycle.
- Sediment and contaminants are trapped in the pavement structure, providing water treatment solutions and preventing stormwater polluted groundwater and other natural water bodies.
- It can be designed as a part of rainwater harvesting and contribute to other sustainable urban water management systems.
- Permeable pavement might reduce the heat island effects due to evaporative cooling.

The interception rate of pollutants by the permeable pavement system can be as high as 100% (Pezzaniti, Beecham and Kandasamy, 2009). However, sediment accumulation and clogging could become a significant issue for the long-term

application of permeable pavement systems, since clogging could reduce the infiltration capacity of the permeable pavement.

1.3.2 Construction of permeable pavement

Conventional pavements are specially designed for vehicular traffic or footpath, including one or more base-course with close-knit materials that overlie the sub-grade and an impermeable surface (Mullaney and Lucke, 2014). The structure is designed to increase the loading capacity of the pavements, which allows large and heavy vehicles to pass through safely. The permeable paving system is not designed for high traffic volume and heavy load vehicular traffic because of the high potential of structure damage and reduction of pavement performance (Mullaney and Lucke, 2014). The permeable paving system is usually used in low traffic volume areas, such as footpaths and car parks.

Some permeable pavement research has indicated that the system could not work well on a clay soil sub-base because of the infiltration reduction caused by fine particle clogging (Wilson *et al.*, 2008). Also, the system could not be used in a chilly climate, as permeable pavement could be damaged easily by freeze and frosting (Wilson *et al.*, 2008). Further research addressing these issues, such as clogging, freeze/frosting damage, maintenance and dry/wet climate shift, has taken place (Duin *et al.*, 2008; Pezzaniti, Beecham and Kandasamy, 2009; Lucke and Beecham, 2011; Mullaney and Lucke, 2014).

1.3.3 The general structure of permeable pavement system

A permeable pavement structure includes a hard surface and several underlying layers (Figure 1-2). A permeable pavement surface is usually covered with a hard surface (such as concrete, aggregate, grids) that allows light vehicular traffic and footpath (Sansalone, Kuang and Ranieri, 2008; Shackel *et al.*, 2008). The underlying structure consists of a bedding layer (i.e. laying course), two geotextile layers (optional) and an aggregate sub-base. The general structure for different types of permeable pavement is similar, and the design and materials can be adapted based on the regional condition for construction.

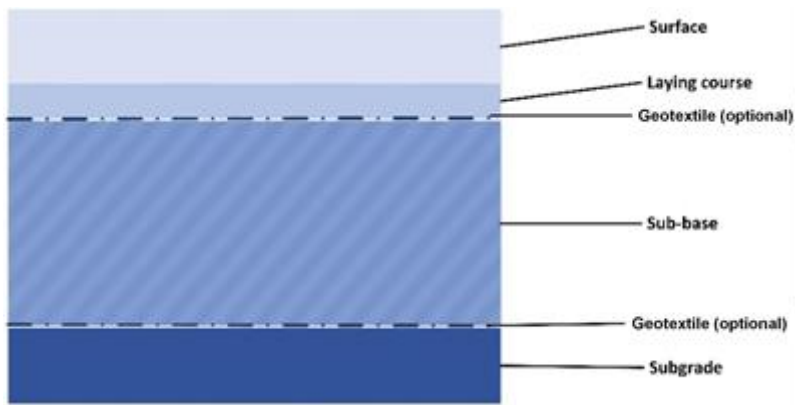


Figure 1-2 Cross-section of the basic structure of the permeable paving system.

Geotextile has been widely applied in urban road and pavement construction for reinforcement, separation, filtration and drainage. Geotextile functions as a drainage layer in permeable pavement structures to separate two layers with significant porosity differences. Geotextiles are considered beneficial for increasing water quality performance, especially in hydrocarbon treatment. Pratt, Wilson and Cooper (2002) also suggested that geotextile helps prevent lateral movement between layers if the geotextile has been carefully chosen.

Geotextile must be selected carefully for different applications based on the physical and hydraulic properties of the layers to achieve the water drainage requirements. Ho (2000) suggested that the geotextile should be selected to retain the soil layer on top of the geotextile with minimum clogging. Bathurst, Ho and Siemens (2007) also concluded three significant criteria for geotextile application selection:

- The permeability of geotextile is large enough to allow water to flow through horizontal or vertical through;
- The soil particles in the layers cannot be transported across geotextile;
- The geotextile will not easily be clogged with fine soil particles internally.

1.3.4 Design of permeable pavement

According to the current research, the surface of a permeable pavement system can consist of four designs: Permeable Interlocking Concrete Paver (PICP), Concrete or Plastic Grid Paver (CGP or PGP), Porous Asphalt (PA) and Porous

Concrete (PC) (Hunt and Collins, 2007; Mullaney and Lucke, 2014). Each type of permeable pavement has a different percentage of permeable area, and the appropriate localised design should be considered according to the local environment and the sub-base condition (Table 1-2).

Table 1-2 Short descriptions and comparisons of different types of the permeable pavement surface. (Source: Hunt and Collins, 2007; Mullaney and Lucke, 2014)

<i>Surface type</i>	<i>Description</i>	<i>Percentage of permeable area</i>	<i>Joint filling materials</i>
PICP	Unique design shape of paving, the joint between paving blocks allow water to infiltrate into the underlying structure.	8-20%	2-5mm aggregate
CGP & PGP	Concrete or plastic grid with materials infill. The opening spaces are larger than PICP.	CGPs: 20-50% PGPs: 90-98%	Aggregate or soil with vegetation plant
PA	Hot mix asphalt without fine portion.	100%	N/A
PC	A unique mixture of Portland cement binder, fine aggregates were omitted.	100%	N/A

1.4 Plastic Grid Permeable pavement (PGP) system

The general design of PGP comprises a plastic grid structure, infilled with soil, planted with vegetation on top, and underlain with a laying course (bedding layer), geotextile and sub-base (Figure 1-3). The plastic grid provides stable chemical properties and structural integrity, which will not be involved in any reactions in the system. The high percentage of permeable surface (90-98%, Table 1-2) of PGP allows most of the inflow water to infiltrate into the underlying layer smoothly. Grass encourages evapotranspiration, minimises surface erosion from the soils and increases soil water retention ability. The surface infill can be replaced by gravel (same aggregates used for laying course) to maximise infiltration capacity. The material of the top surface infill will strongly affect the degradation and retention of a contaminant such as heavy metal and hydrocarbon (Das and Chandran, 2011; Li *et al.*, 2017).



Figure 1-3 Installed PGP system (left) and cross-section of PGP system design (right)

Considered the possibility of a reduction of infiltration rate due to the mixing of the soil and gravel sub-base, and to increase contaminant removal efficiency, two geotextile layers can be added to the design of PGP for improving the hydrological and water quality performance (Figure 1-3). The thickness of soil infill and sub-base (Table 1-3) varies depending on the pavement load and water treatment requirements (Mullaney and Lucke, 2014; Ballard *et al.*, 2015).

Table 1-3 Characteristics of each layer

Structure layer	Thickness (mm)	Aggregate size (mm in diameter)
Soil infill	30-50	-
Plastic grid	30-50	-
Laying course	30	2-5
Sub-base	200-500	20-63

1.5 Hydrological performance

The hydrological impact of permeable pavement increases the infiltration capacity of the urban area, reduces surface runoff, and decreases peak flow in receiving watercourses.

1.5.1 Hydraulic performance of permeable pavement

Previous empirical research shows that compared with conventional concrete or asphalt road, a permeable paving system can reduce urban surface runoff by 52% (Jayasuriya and Kadurupokune, 2010) to 100% (Collins, Hunt and Hathaway, 2008; Castillo-Rodríguez et al., 2021) and peak reduction by 40% (Jayasuriya and Kadurupokune, 2010) to 100% (Rodríguez-Rojas *et al.*, 2018). By installing permeable pavement, the urban imperviousness has been reduced by around 40%; at the same time, more than 60% of urban surface runoff volume has decreased (Rankin and Ball, 2004; Jayasuriya and Kadurupokune, 2010; Al-Rubaei *et al.*, 2013). The permeable paving system also performs well during a 1:20 year return storm (Rodríguez-Rojas *et al.*, 2018). The main findings from the fieldwork on the hydraulic performance of permeable pavement are summarised in Table 1-4.

Permeable pavement types differ in their hydraulic performance; generally, there are no consistent results. Some research indicated that PICP has a higher infiltration rate compared to the PGP system, but the CGP and PGP systems presented a rather excellent runoff management capacity. For example, the infiltration capacity of the paving blocks surface (PICP) was shown to be higher than the vegetated plastic grid surface (PGP) permeable paving system (Straet, Beckers and Degré, 2008). However, Jayasuriya and Kadurupokune (2010) compared the hydraulic performance of Ecotrihex paver (PICP) and Turf Cells (PGP). According to Jayasuriya and Kadurupokune (2010) research, the PGP system performs better in runoff reduction, 7-9% higher than Ecotrihex paver. The PGP pavement was measured to absorb 5 mm more rainfall before surface runoff was generated compared to Ecotrihex paver (Jayasuriya and Kadurupokune, 2010). Fieldwork in Spain also suggested that CGP and PGP have a better runoff management capacity (Rodríguez-Rojas *et al.*, 2018).

Table 1-4 Field study results of runoff volume and peak flow reduction achieved by permeable pavement installation from literature.

Research study (Location)	Permeable pavement type	Rainfall	Main finding
<i>Pratt, Mantle and Schofield, 1995</i> (Nottingham, UK)	PICP (car park)	9 mm event	22%-51% of rainfall becomes discharge, with 4 different sub-base (gravel, blast furnace slag, granite and limestone)
		10 mm event	51%-100% of rainfall becomes discharge, with 4 different sub-base (gravel, blast furnace slag, granite and limestone)
<i>Gilbert and Clausen, 2006</i> (Waterford, CT, USA)	PICP (driveway)	22 months study, the median rainfall intensity was 9 mm/h, and the median duration was 210 min	72% runoff reduction with PICP compared to conventional asphalt.
<i>Collins, Hunt and Hathaway, 2008</i> (City of Kinston Public Service Complex in eastern North Carolina)	PC, CGP and PICP	56 rainfall events on-site, rainfall depths ranged from 3.1 to 88.9 mm	Surface infiltration rate: PC>PICP>CGP. Compared to conventional asphalt pavement, the average percentage of surface runoff reduction from rainfall depth is 99.9% for PC, 98.2% for CGP and 99.4% for PICP.
<i>Dreelin, Fowler and Ronald Carroll, 2006</i> (Athens, Georgia, USA)	PGP (soil with grass surface infill)	9 rainfall events on-site, rainfall depths ranged between 0.03 and 1.85 cm	The PGP car park surface generates 93% less surface runoff than the conventional asphalt surface.

(continued)

Research study (Location)	Permeable pavement type	Rainfall	Main finding
<i>Brattebo and Booth, 2003</i> (Renton, Washington)	PGP with soil and plant surface infill, PGP with gravel surface infill, CGP with soil and plant surface infill and PICP	During several separate measurement months, total rainfall was 570 mm, with 15 distinct events.	In general, there is very little or no surface runoff generated. Only 1 mm of runoff has been generated for PGP with plant infill under a 42 mm, 14 hours rainfall event. 4 mm runoff was generated under the maximum rainfall events (total of 121 mm of rainfall within 72 hours).
<i>Rodríguez-Rojas et al., 2018</i> (Cartuja university campus, Granada, South of Spain)	PICP, CGP with grass surface infill, and PGP with aggregates surface infill	5.4-36.6 mm rainfall event	Compared to conventional pavement, all three types of permeable pavement significantly reduce generated runoff volume by more than 84%. CGP and PGP presented similar data; however, CGP has the highest value of 95%. All three types of permeable pavements also show a similar peak flow reduction, with a median value close to 100%. Permeable pavement also shows high volumetric efficiencies of over 80% in decreasing rainwater collection in the sewer system.
<i>Castillo-Rodríguez et al., 2021</i> (Benicàssim, Spain)	PICP pavement with ceramic tiles	Rainfall events between 15-25 mm	The system presented a nearly 100% capacity for runoff management.
		Maximum rainfall event 48.8 mm, during 13 months monitor	The hydraulic performance has been reduced to 26% of the original.

Data collected from fieldwork and lab experiments used for consistent comparison between systems is complicated by various factors, such as the dry and wet periods of the year (Fassman and Blackbourn, 2010). Thus, it is difficult to determine which system has a better hydrological performance. However, a modelling method can help with similar problems by simply changing the medium parameters.

1.5.2 Previous modelling practice of hydraulic performance

Some modelling work has considered the hydraulic performance of the permeable pavement. Previous models have simulated the surface (PA, PC and PICP only) and several underground sub-base layers separately, using the Kozeny-Carman equation (Tan, Fwa and Han, 2003; Montes and Haselbach, 2006; Kuang *et al.*, 2011) or experimental data (Deo, Sumanasooriya and Neithalath, 2010; Yong, McCarthy and Deletic, 2013). The Kozeny-Carman equation is a fluid dynamics mathematical relationship used to calculate the pressure drop of fluids flowing through a soil packed bed (Carman, 1956).

Several modelling studies have highlighted the benefit of permeable pavement in catchment runoff management. The previous modelling results demonstrated a similar high runoff removal rate, even under intense rainfall. Jato-Espino *et al.* (2016) simulated the permeable pavement in an urban catchment in Espoo, Finland, which concluded a 40%-50% average reduction in runoff and catchment total outlet volume. The modelling tools used by Jato-Espino *et al.* (2016) are ArcGIS and Storm Water Management Model (SWMM). Ellis and Viavattene (2014) found that 57% and 30% of overflow can be decreased under a 1:30 year storm and 1:200 year storm, respectively, with a combined permeable pavement and green roof application. The storm was selected from a storm sequence measured in 2007 in Birmingham and Coventry, UK; the estimated 1:200 year event has a total rainfall of 98 mm within 8 hours, while the 1:30 year event has a total rainfall of 35 mm within 6 hours (Ellis and Viavattene, 2014). The modelling tool used by Ellis and Viavattene (2014) is a decision-making tool SUDS Selection and Location tool (SUDSLOC), developed by Viavattene and Ellis (2013). Moreover, the Urban Stormwater Improvement Conceptualisation

(MUSIC) (Wong *et al.*, 2002) model was used to predict the runoff reduction in Auckland and Scotland (Imteaz *et al.*, 2013). In Auckland, the predicted mean runoff reduction is 89%, with rainfall of 25.89 [mm h⁻¹], only 3% lower than the experimental results under the exact condition (Imteaz *et al.*, 2013). However, the MUSIC predicted runoff reduction is 100% with rainfall of 3.9 [mm h⁻¹], which is much higher than the observed data (82%) (Imteaz *et al.*, 2013).

1.5.3 Design tools

SUDSLOC, MUSIC and SWMM are all catchment-based rainfall-runoff simulation models. SUDSLOC is a SUDS technique selection tool that simulates urban catchment rainfall-runoff-sewer flow processes in 1D/2D dimensions (Viavattene and Ellis, 2013; Ellis and Viavattene, 2014). MUSIC is also a decision support system, and it is not a detailed deterministic design model but a conceptual stochastic model (Wong *et al.*, 2002; Imteaz *et al.*, 2013). MUSIC allows users to evaluate possible conceptual designs of stormwater management facilities to achieve predetermined water quality and hydraulic targets in their catchments (Wong *et al.*, 2002). SWMM is a widely used dynamic rainfall-runoff tool for event-based or long-term hydraulic simulation (Rossman, 2010). SWMM operates as a collection of several sub-catchment that receives rainfall and generates runoff and pollutants (Rossman, 2010). These sub-catchments are connected with a complex system of pipes, channels, pumps, regulators, and different storage/treatment devices; meanwhile, each sub-catchment connection's water flow and quality can be predicted over time (Rossman, 2010).

All modelling tools mentioned above are widely used for permeable pavement hydraulic performance simulation. However, they are not entirely suitable for this study. First, existing model tools focus on surface runoff (horizontal catchment flow) instead of infiltration (vertical flow across several layers of permeable pavement). Infiltration is crucial as it is the driving force for suspended particles to travel deeper into the permeable pavement structure. Second, most of the existing tools contain pre-settings for permeable pavement, in which the structure and materials details cannot be altered. However, as clogging develops in the structure, essential parameters (such as porosity, saturated hydraulic

conductivity and media particle size) will be changed over time. Third, existing tools' preset package for permeable pavement does not allow customised equations and physics. Clogging is a complex process in porous media, including physical, chemical, biological and other reactions. This requires simulation tools that allow multiple physical interfaces (such as water flow, concentration and reaction, and particle tracking) to communicate and insert new formulas. Overall, COMSOL Multiphysics satisfies all the above requirements, so this research uses it as the primary simulation tool.

1.5.4 Modelling the hydraulic performance

When modelling the hydraulic performance, permeable pavements have usually been considered a multi-phase porous medium with several layers (Huang *et al.*, 2016). As the most sensitive property affecting hydraulic performance directly, porosity has been selected as the main factor for hydraulic modelling for permeable pavement.

Previous studies used different equations to simulate the porous medium water flow behaviours, and water flow through porous media is an extensively studied area. Woods-Ballard *et al.* (2007) suggested using Darcy's law to estimate the water flow behaviours in the sub-base for sloping subgrades and non-symmetrical pipeline networks, which is similar to the permeable pavement. Kozeny-Carman (Carman, 1956) and van-Genuchten (van Genuchten, 1980) models have been developed to describe the water retention relationship between porosity and hydraulic conductivity in unsaturated flow, and both models have been widely used. The quantitative relationship between porosity and hydraulic conductivity of a porous concrete surface has been established by Montes and Haselbach (2006). Their method is similar to Tan, Fwa and Han (2003), which used the Kozeny-Carman equation and experimental data to model the porosity and hydraulic conductivity on the base layer of permeable pavement. It has also been suggested that the relationship between hydraulic conductivity and porosity can be successfully simulated by the modified Kozeny-Carman equation (Kuang *et al.*, 2011; Huang *et al.*, 2016).

The parameters required for the water retention model are crucial since the clogging process could affect the soil property parameters. Depending on the parameter's availability and application criteria, the van-Genuchten and Kozeny-Carman models have been used in this study. The equations can be found in the later chapters; the van-Genuchten equations have been indicated in equations (2-11), (2-12), (2-13), and (2-14), and the Kozeny-Carman formulation in equation (5-23).

1.6 Pollutant removal performance

The primary pollutants of permeable pavement come from rainfall and urban runoff. Cities are built as an environment with multiple functions such as social, economic, and industrial. Therefore, stormwater runoff contains a range of pollutants from simple daily life, transportation, and industrial emissions (Barlow, 2015). Escarameia, Todd and Watts (2006) have determined that the city's primary source of pollution is the construction and maintenance of roads and vehicles, especially in traffic areas. Dierkes, Holte and Geiger (1999) suggested that pollution might come from car leakage, atmospheric deposition, traffic activities or pavement degradation.

Since many pollutants tend to attach to the sediments, the total pollutant load of the city correlates positively with the sediment load (Vaze and Chiew, 2004; Björklund, 2011). However, the sediment load in cities varies greatly, affected by many factors, such as climate, topography, physicochemical conditions and land use (Barlow, 2015). Depending on the location, weather condition and surrounding environment, the mass of particulate matter accumulated on the urban surface over 5 days varies from 7.3 to 740 [g m⁻²] in the UK (Vaze and Chiew, 2002; Carraz *et al.*, 2006).

More detailed urban pollutant sources and pollutant types have been concluded by Woods-Ballard *et al.* (2007). The pollutants that frequently appear in the permeable pavement with higher concentrations include total suspended sediment (TSS), total phosphorous (TP), total nitrogen (TN), heavy metal (e.g. Iron, Zinc, Lead, Copper and Cadmium) and petroleum hydrocarbon (Huang *et al.*, 2016).

The water quality performance of permeable pavement could be affected by the system structure design. A previous study found comparable removal efficiencies for contaminants by different types of permeable pavement (Table 1-5). PGP system demonstrated a higher zinc and oil removal rate and a slightly lower removal rate of total phosphorus. The reduction of total suspended sediments of both systems is similar.

Table 1-5 Removal efficiencies of water quality performance of two different types of permeable pavement (Jayasuriya and Kadurupokune, 2010).

<i>Permeable pavement type</i>	<i>Total suspended sediment</i>	<i>Zinc</i>	<i>Total phosphorus</i>	<i>Petroleum oil</i>
PICP (Ecotrihex paver)	87-92%	76-93%	55-62%	89-92%
PGP (Turf Cell)	90-92%	91-92%	50-56%	93-94%

The removal rate of TSS for permeable paving system stay at a high level at around 80-95% (Pratt, 1997; Brown *et al.*, 2009; Fassman and Blackburn, 2010; Jayasuriya and Kadurupokune, 2010). In general, Pezzaniti, Beecham and Kandasamy (2009) found that the permeable paving system can still remove nearly 100% of suspended sediment after 35 years of installation.

The heavy metal removal rate of permeable pavement is not uniform, and it highly depends on the design structure and the chemistry element components. In general, the permeable pavement shows excellent ability in removing heavy metal from surface runoff, with PICP outperforming PA types (Table 1-6). The better removal rate of heavy metals may be caused by the smaller porosity of PICP.

Table 1-6 Heavy metal removal rate of permeable pavement

<i>Source</i>	<i>Permeable pavement type</i>	<i>Pb</i>	<i>Zn</i>	<i>Cd</i>	<i>Cu</i>
Legret, Colandini and Le Marc, 1996	PA	79%	72%	67%	-
Legret and Colandini, 1999	PA	74%	77%	84%	-
Dierkes <i>et al.</i> , 2005	PICP	99%	94-96%	99%	98%
Fassman and Blackburn, 2010	PICP	-	90%	-	70%

The reported performance of removing total phosphorus and total nitrogen has been inconsistent (Table 1-7). The removal rate of TP varies from barely removed to more than 70%, and TN varies from less than 10% to 60%. Due to these differences in experimental environments, the massive difference in nutrient removal rate may be because the removal of this pollutant is greatly affected by the local microorganisms, plants, and chemical environment.

Table 1-7 Observed removal efficiency of TP and TN for permeable pavement

<i>Contaminant type</i>	<i>Removal efficiency</i>	<i>Source</i>
Total Phosphorus (TP)	Higher than 70%	Ball and Rankin, 2010; Tota-Maharaj and Scholz, 2010
	40-60%	Rushton, 2001; Bean, Hunt and Bidelsbach, 2007
	Barely remove	Gilbert and Clausen, 2006
Total Nitrogen (TN)	40-60%	Rushton, 2001; Bean, 2005
	Less than 10%	Dreelin, Fowler and Ronald Carroll, 2006; Collins, Hunt and Hathaway, 2010

Microbial biodegradation in-situ experiments with extra fertiliser were used to determine the petroleum hydrocarbon retention and treatment performance in a permeable pavement structure. (Pratt, Newman and Bond, 1999). It was concluded that petroleum hydrocarbon concentration in urban stormwater could be reduced 97.6% by PICP permeable pavement with a geotextile layer (Pratt, Newman and Bond, 1999). The hydrocarbon removal rate in the permeable

pavement is high, at 99.6%, and it is 50% higher than the conventional asphalt surface and 29.4% higher than the conventional concrete surface (Newman *et al.*, 2002).

Petroleum hydrocarbon retention performance is significantly affected by biodegradation processes. The hydrocarbon is mainly biodegraded by bacteria, yeast, and fungi (Das and Chandran, 2011). The nutrients (nitrogen, phosphorus and iron) will affect the rate of degradation of hydrocarbon (Das and Chandran, 2011). Although the external nutrients will promote the biodegradation rate in the soil, after nine years of degradation, experiments with both conditions (with or without external nutrients) reached the same result (Polyak *et al.*, 2018).

Permeable pavement works well in water quality treatment through a series of physical, chemical and biological processes (Dierkes, Holte and Geiger, 1999). The removal mechanisms vary between pollutant types. In general, sedimentation and filtration are the most common removal mechanisms (Table 1-8). Various biological and chemical processes affect the removal of nutrients and hydrocarbons (Table 1-8). The physical process is the most crucial removal mechanism. The physical mechanism can remove sediments, and also many contaminants that tend to attach to the sediment can also be removed.

Table 1-8 Removal mechanisms of pollutants of nutrients and hydrocarbons in SUDS (Wilson, Bray and Cooper, 2004; Woods-Ballard *et al.*, 2007)

Pollutants	Removal mechanisms in SUDS
<i>Sediments</i>	Sedimentation, filtration.
<i>Nutrients</i> (Phosphorous, Nitrogen)	Sedimentation, biodegradation, precipitation, de-nitrification, uptake by plants.
<i>Hydrocarbons</i>	Biodegradation, photolysis, filtration, adsorption.
<i>Metals</i> (Lead, copper, cadmium, mercury, zinc, chromium, aluminium)	Sedimentation, adsorption, filtration, precipitation, uptake by plants.
<i>Metalloids</i> (Arsenic, antimony)	

1.6.1 Modelling the water quality performance

Whilst numerous empirical studies have been conducted, less work has been done in water quality modelling for permeable pavement systems. As suggested in Table 1-8, the complete physical-chemical-biological model is complex, since the interactions involve various pollutants and environmental factors (Deletic and Orr, 2005; Tuccillo, 2006; Wilson *et al.*, 2008). However, building up the integrated model is essential, and the physical model should be the starting point.

A regression model has been developed by Huang *et al.* (2016) for predicting the overall TSS removal efficiency of permeable pavement based on laboratory data. Huang *et al.* (2016) also developed a practical mathematical model for simulating the TP and TN retention and treatment performance for PA, PC and PICP permeable pavement based on obtained laboratory data.

Huang *et al.* (2016) suggested that suspended solid removal processes can be modelled by the sediment removal model developed by Yao, Habibian and O'Melia (1971). Yao, Habibian and O'Melia (1971) proposed simulating the water and wastewater treatment processes with a sand filter based on the particle transport and force of contact with medium grains.

It is important to note that, although many researchers and city planners are interested in installing permeable pavement, it is difficult to describe and predict the permeable pavement's performance accurately. Therefore, it is worth creating a process-based model to include the pollutant removal mechanisms in Table 1-8.

1.7 Conceptual model of hydraulic and pollutant removal processes in the PGP system

Summarising the relationship between various pollutants and infiltration water in the previous study, Figure 1-4 describes the water flow direction and contaminant reaction in each layer of permeable pavement.

The water flows downwards inside the system driven by gravity (except for the evapotranspiration from the topsoil layer). The inflow water infiltrates through

each layer of the permeable paving system, reaches the subgrade, and discharges to groundwater (or collected by pipeline). Vegetation provides force for water uptake and evapotranspiration in the topsoil layer. The surface runoff will be generated when the inflow rate exceeds the overall infiltration capability in a water-saturated paving system.

Following the water flow, contaminants move downward in the structure. Various chemical and biological reactions may happen inside each layer. The sediments can be trapped in the structure pores and cause clogging problems. Other contaminants (heavy metal, TP, TN and hydrocarbon) show more complicated reactions in the PGP structure. The soil environment on the top layer of the PGP system encourages complex chemical and biological reactions. At the bottom of the structure, contaminants remaining in water will be discharged.

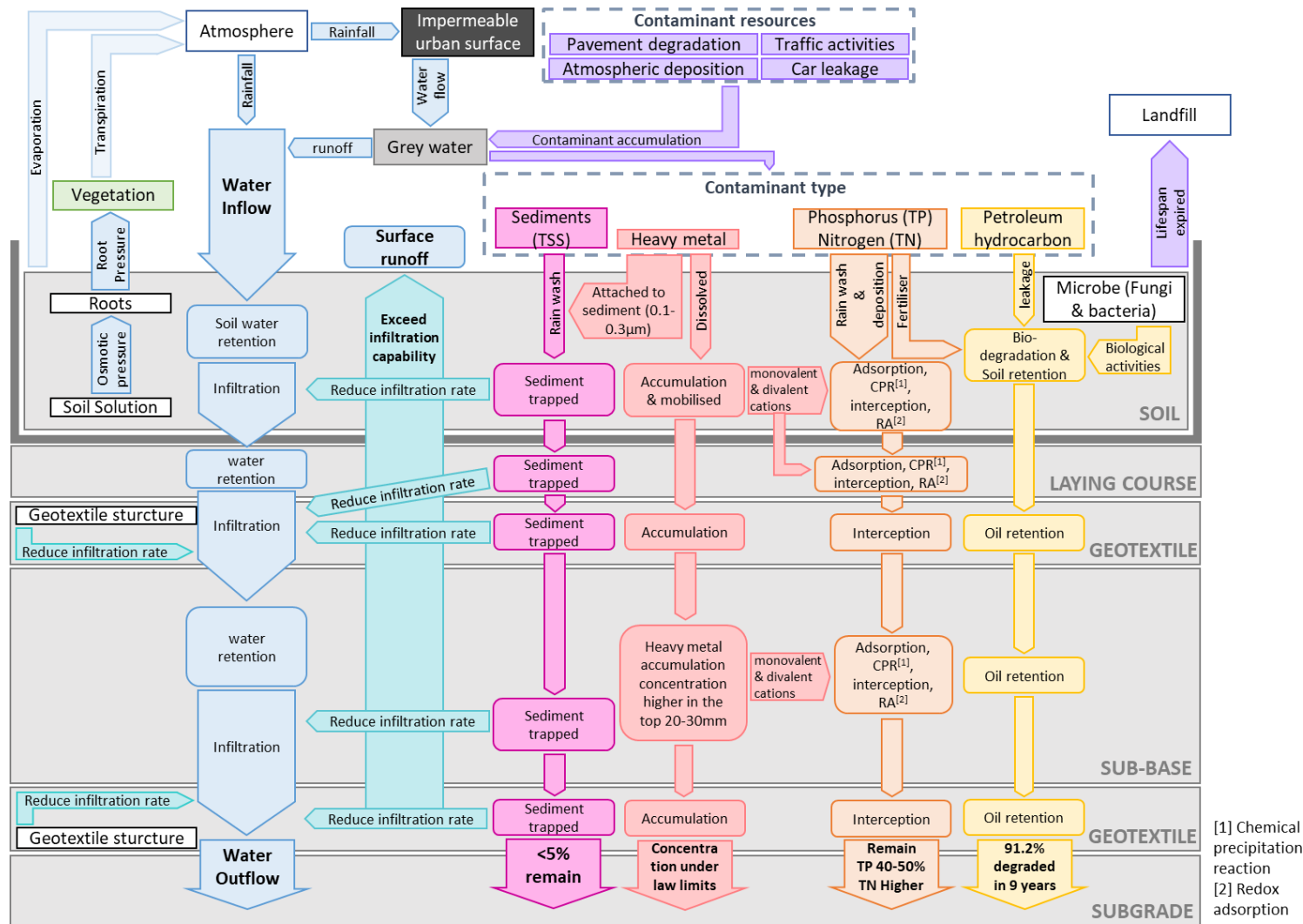


Figure 1-4 Conceptual model of the physical, chemical and biological processes occurring in permeable grid paving.

1.8 Suspended particle clogging phenomenon of permeable pavement

As shown in sections 1.5 and 1.6, the hydraulic and water quality performance is strongly affected by the properties and structure of the permeable paving system. However, the structure and properties change over time. One of the main factors responsible for this change is the clogging by suspended particles. Clogging is a physical, biological or chemical process caused by fine granular or organic material blocking the flow of water down through the permeable pavement (Siriwardene, Deletic and Fletcher, 2007a).

The clogging processes will reduce porosity and permeability, resulting in a decrease in infiltration rate (Gerrits and James, 2002; Borgwardt, 2006). During dry days, fine particles carried by vehicles landed or wind landed on the surface of permeable pavement. When the rain starts, fine particles are wash into the pore spaces and infiltrated through layers (Tan, Fwa and Han, 2003).

Particles accumulate in the pore spaces, thus decreasing the porosity, weaken permeable pavement storage and drainage abilities. Fine particles may also be produced during construction (Tan, Fwa and Han, 2003). The lower porosity of the top 25 mm of the structure has been found in PC permeable pavement, which might indicate a fine particle clogging process (Kayhanian *et al.*, 2012). On the other hand, clogging processes will increase the suspended sediment removal rate, and it is beneficial to improve the water quality.

While some studies have found only a modest decrease of 8% in hydraulic capacity over 6 years (Kadurupokune and Jayasuriya, 2009), over a longer time of 10 years, this increased to 20-25% (Borgwardt, 2006). Whilst this may still provide stormwater attenuation after 20 years of application (Kadurupokune and Jayasuriya, 2009), and the Infiltration rate of permeable pavement may still be satisfactory after 8 years of operation (Lucke and Beecham, 2011) or 18 years of operation (Yong *et al.*, 2008, 2010). However, the decrease in hydraulic capacity will become more of an issue with climate change and extreme precipitation (Section 1.1.2).

Clogging happens in different positions of the permeable pavement due to the design difference. For PA and PC surface permeable paving systems, clogging usually develops into 25-75 mm in depth (Ballard *et al.*, 2015). Mineral and fine organic sediments were found to be trapped in the upper 20 mm gaps between blocks or pores for PICP permeable pavement (Borgwardt, 2006). Lucke and Beecham (2011) found that most of the sediments (around 92% by mass) accumulated in the surface and bedding layers of the PICP permeable paving system. Meanwhile, clogging could penetrate 50 mm or 6-12 mm deep for PICP or CGPs with grass surface infill, respectively (Hunt, 2011).

Clogging can also be affected by particle characteristics carried by the stormwater, such as particle size. Suspended particle size plays a vital role in the changes in hydraulic performance during clogging development (Wang *et al.*, 2012). Wang *et al.* (2012) experimented with the aquifer recharge using a fine quartz sand column with three groups of inflow suspended particles (0.075-0.05 mm, 0.05-0.0385 mm and <0.0385 mm). The experiment results indicated that the clogging deposited on the surface with suspended particle size (SPS) range 0.075-0.0385 mm, and the clogging layer infiltrated to 2 cm depth with SPS less than 0.0385 mm (Wang *et al.*, 2012). Also, small-diameter suspended particles clog faster than larger-diameter particles (Wang *et al.*, 2012). A series of 1D laboratory experiments indicated that physical clogging is mainly caused by transporting suspended particles sized less than 6 μm (Siriwardene, Deletic and Fletcher, 2007a). Kovenya, Melnikova and Frid (1972) suggested that particles with SPS between 1 μm to 10 μm tend to concentrate on the infiltration medium surface. The infiltration medium can adsorb particles with a minimum diameter of 0.1 μm , while the suspended particles with a diameter of 0.5 μm can migrate the longest distance (Kovenya, Melnikova and Frid, 1972). At the same time, Banihashem and Karrabi (2022) performed a sand filter (medium average grain-size: 3.55 mm) experiment to determine the hydraulic performance differences caused by suspended particle clogging (with SPS at 4, 20 and 40 μm and concentration at 1 $[\text{g L}^{-1}]$). The hydraulic conductivity and porosity are significantly affected by the entering suspended particle size, the larger the particle size, the faster the reduction of the medium hydraulic conductivity and porosity

(Banihashem and Karrabi, 2022). With inflow SPS at 4, 20 and 40 μm , the porosity reduced by 41%, 57% and 53%, and hydraulic conductivity reduced by 85%, 95% and 93%, respectively (Banihashem and Karrabi, 2022).

Furthermore, clogging processes are potentially affected by weather factors (rainfall patterns and temperature), environmental conditions (air quality, catchment soil characteristics and surrounding water bodies) and infrastructures (contributing drainage area, traffic volume) (Razzaghmanesh and Beecham, 2018). The clogging process may also be affected by the condition of the geotextile, a clogged non-woven geotextile (i.e., fine particle or hydrophobic surfactant coating on the surface) has a smaller pore size, and it causes a higher water-entry suction pressure in wetting path (Stormont, Henry and Evans, 1997; Ho, 2000; Bouazza *et al.*, 2006). Meanwhile, Stormont and Morris (2000) suggested that soil penetration will also affect the hydraulic properties of geotextiles.

1.8.1 Clogging caused damage and permeable pavement lifespan

Several pieces of research have investigated the clogging phenomenon in the permeable pavement structure. However, most of the previous studies focus on PICP, PA and PC systems.

Razzaghmanesh and Beecham (2018) concluded 13 studies of PC, PA and PICP infiltration rates. It shows that the infiltration rate starts to reduce in the second year of installation. After four years of operation without maintenance, the rate will continuously decrease to below 1 [m h^{-1}] (from around 50 [m h^{-1}]), which is an acceptable permeable pavement infiltration rate in some countries like Australia. The infiltration data from Razzaghmanesh and Beecham (2018) has covered many cities in Europe, North America, Australia and China. Still, there is a lack of infiltration rate data after five years of operation.

On the other hand, some papers concluded a much quicker system failure. For example, Borgwardt (1997) introduced a field study in which the infiltration rate decreased by 50% after five years of application.

Pratt, Mantle and Schofield (1995) investigated a field study that estimated the permeable pavement system would remain effective for 15-20 years. Suarman, Argue and Pezzaniti (1999) concluded that a 25 (or more) years of continuous sediment inflow at 80 [mg L⁻¹] is required for the permeable pavement to fail (hydraulic conductivity of the upper 50 mm structure decreased to 1/3 of its initial). Another experimental field study in Germany suggested that the infiltration rate reduction has stabilised at 20-25% after around 10 years (Borgwardt, 2006). Pezzaniti, Beecham and Kandasamy (2009) compared three different pavement systems (Formpave, Rocla Ecoloc and Grasspave) through experimental and field trials over 35 years of simulation. The Grasspave system shows the best pollutants retention rate (97%) and the most massive conductivity reduction rate (75%).

Only a few papers describe their findings on PGP systems, which further illustrates the importance of developing a clogging model for PGP systems.

1.8.2 Maintenance

The risks of clogging can be reduced by regular maintenance procedures and improving the permeable pavement system's infiltration rate (Dierkes *et al.*, 2002). Previous studies concluded that severe clogging mainly occurred on a thin surface layer of the pavement structure with porous coating materials, permeable concrete, and aggregates (Baladès, Legret and Madiec, 1995; Haselbach, 2010). Therefore, cleaning the surface of permeable pavement is an effective method to restore drainage to an acceptable level.

Maintenance methods include sweeping, vacuuming, and high-pressure washing to remove fine particles from pore spaces and restore infiltration rate. (Dierkes *et al.*, 2005; Fassman and Blackbourn, 2010) These methods are not verified to be effective for all pavement materials, and sometimes it is necessary to consider a combination of multiple methods (Razzaghmanesh and Beecham, 2018). A replacement needs to be considered after 15-20 years of operation since maintenance procedures can no longer restore infiltration.

In intense rainfall areas, maintenance can be conducted two to four times per year (Razzaghmanesh and Beecham, 2018). The location of the permeable pavement system also affects the maintenance frequency. Pezzaniti, Beecham and Kandasamy (2009) concluded that the structure clogged quicker when the permeable pavement was installed for heavy stormwater source control and shortened the usage lifespan.

A comprehensive model can help to evaluate the clogging process, and provide maintenance suggestions (maintenance method and cleaning depth) and the improvement level after maintenance.

1.9 Conclusion of pollutants, materials and influence factors interactions of permeable pavement

The factors that might affect the PGP system's water quantity and quality performance have been identified in Figure 1-5. The diagram demonstrated the influence factors, including different contaminants, natural factors, and material properties. The consequences after their inter-reaction are also shown in Figure 1-5. In general, water quality can be improved, but clogging processes might reduce the infiltration capacity of the PGP system. The infiltration and clogging processes are the results of the combination of several factors.

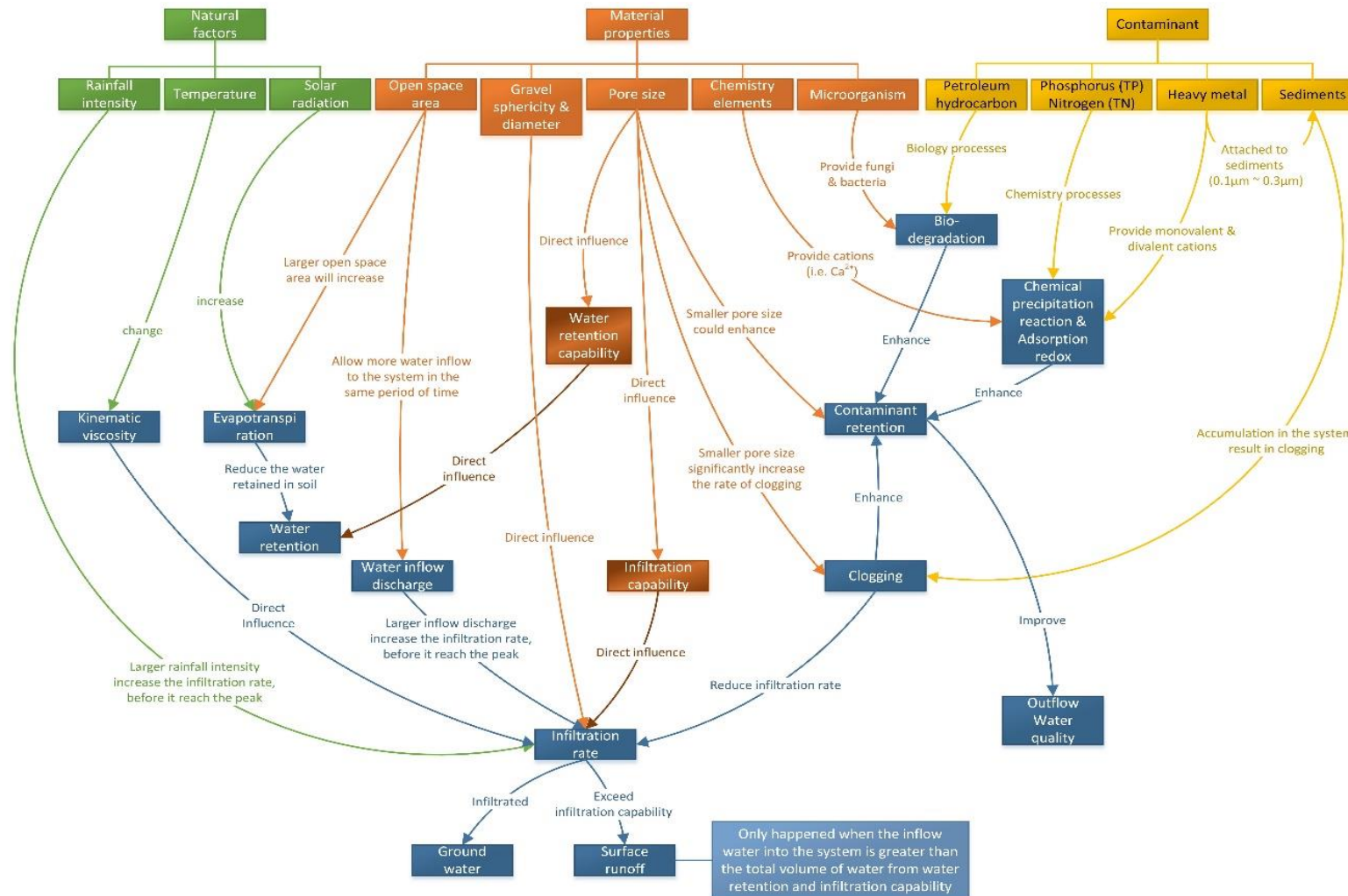


Figure 1-5 Factors that impact the hydraulic performance and pollutant transport happen in PGP systems.

1.10 Modelling the sediment clogging process for the PGP system

Currently, there is no mathematical model sufficient to simulate the clogging process in the PGP system. Clogging is mainly caused by sediment retention; sedimentation and filtration are the key processes for pollutant retention (Table 1-8). The clogging process in the PGP system can be simulated using sediment transport models. The degree of fine particle accumulation in the PGP structure profoundly depends on the pore structure (porosity, shape of media particle and tortuosity), soil permeability, and pollutant particle size distribution (Deo, Sumanasooriya and Neithalath, 2010).

As presented in Chapter 1.8, the influence factors affecting the clogging process are complicated. The influence factors have not been fully considered with the empirical model. Although geotextile clogging has been studied independently, the possible effect of geotextile on the hydraulic and clogging process has not been included in the previous models.

There are a considerable amount of sediment transport models which have been developed previously for filtration systems. Huang *et al.* (2016) developed a model which involved both hydraulic and sediment removal processes for PICP structure; the model can simulate the hydraulic performance with an R^2 (coefficient of determination) at around 0.762-0.907. Clogging processes also can be modelled by porosity reduction with experimental data. Yong, McCarthy and Deletic (2013) developed a Black-Box regression model to simulate porosity reduction for a porous asphalt surface. A simple conceptual pollutant removal model (k - C^* model) for gravel filter has been presented by Kadlec and Wallace (2009). The k - C^* model depends on a first-order dynamic decay theory, assuming a steady-state condition (Kadlec and Wallace, 2009); k and C^* are two of the most important parameters for the model.

A trajectory method has been developed by Yao, Habibian and O'Melia (1971) to estimate the suspended particle trajectory and evaluate the collision opportunities between the suspended particle and the gravel media grain. Later, the Yao model

was further modified and developed by Tufenkji and Elimelech (2004) and Siriwardene, Deletic and Fletcher (2007b).

The previous models mainly were semi-empirical or empirical models (Yong's model and $k-C^*$ model), which severely limited the model application to a specific environment, system type and material. Also, the previous simulation lacked a comprehensive understanding of the suspended particle transport mechanism in the PGP system, especially the difference between fine and coarse particles.

Because of the clogging processes, the long-term water quantity and quality performance are difficult to predict (Borgwardt, 2006). Clogging may also occur at different structure depths (Section 1.8); fine sediment may cause clogging quickly in the geotextile layer, reducing the infiltration rate (Ballard *et al.*, 2015).

Overall, an integrated physically-based model that includes both hydraulic and clogging models should be proposed to understand the clogging process of the PGP system (Ballard *et al.*, 2015).

1.11 Aim

The aim of this research is to evaluate the mechanisms of the hydraulic and suspended particle clogging processes in the PGP system using a modelling approach.

1.11.1 Objectives

Three objectives have been carried out through the project and listed below to achieve the aim.

1. Develop the theory-based mathematical approach for the hydrological and clogging process of the PGP system with reasonable assumptions.
2. To apply the new model to different input parameters (rainfall intensity, initial soil water content, suspended particle concentration and particle size), compare the model performance, and explore the mechanisms behind it.

3. Discuss the further understanding of suspended particle clogging in the PGP system and the implication of the modelling method for design and maintenance for the PGP system in real life.

1.12 Methodology overview

According to the overall aim and objectives, the new modelling method has focused on the hydraulic and clogging process of the PGP system. The integrated performance model of the PGP system includes three parts 1) **H**ydraulic model, 2) **R**ainfall-infiltration boundary, and 3) **C**logging model; therefore, the integrated model will be named the *PGP-HRC model*.

This section presents a summary of model geometry and modelling tools for the PGP-HRC model. The model geometry has been applied to all three parts of the PGP-HRC model tests.

1.12.1 Model geometry

The general design of the PGP system includes eight layers (Figure 1-3), with a gravel sub-base. However, a simplified model was developed that includes three layers of soil, sand and geotextile for modelling. The simplification is based on three assumptions: 1) the porosity and hydraulic conductivity of soil, sand and geotextile are orders of magnitude lower than gravel, 2) the soil properties make it easier to be affected by clogging, and 3) literature suggested that clogging tends to accumulate on the soil surface (Section 1.8). These simplified three layers (soil, sand and geotextile) provide the most critical impact on hydraulic performance.

Different material types were chosen for model tests based on the general PGP system design (Figure 1-3). Sandy loam is a suitable soil type for turfgrass growing medium, such as the PGP system's top 5 cm surface. For the laying course, sand provides a stable base and a transition to prevent direct contact between geotextile and sandy loam. Non-woven geotextile hydraulic performance is similar to coarse sand, which provides drainage, separation, and support.

A 1D model geometry includes a 5 cm loamy sand layer, with an underlying layer of a 3 cm sand layer and a 0.43 cm non-woven geotextile layer at the bottom (Figure 1-6). The thickness of each layer is determined according to the data recommended by the SUDS manual (Section 1.4).



Figure 1-6 Cross-section of three layers structure for simulating the hydraulic performance of soil, sand and geotextile

1.12.2 Model tools of COMSOL Multiphysics

COMSOL Multiphysics is a finite element analysis software (Multiphysics and Script, 2019). The finite element method is popular for numerically solving partial differential equations to obtain an approximate solution with the lowest error. During the calculation, the finite element analysis method considered each element in both spatial and time dimensions. The model components and the dependent variable are well structured and modularised in the COMSOL interface. Some of the widely used fluid equations (such as Darcy law) have already been integrated into the Subsurface flow module. COMSOL allows several physics interfaces to communicate and customise differential equations for domain or boundary. In conclusion, COMSOL Multiphysics meets the required degrees of freedom for creating the PGP-HRC model.

1.13 Thesis structure

The thesis is composed of eight chapters (Figure 1-7). Chapter 1 is the general introduction and justifies the background, concept, aim and structure of the project. The remaining chapters are arranged in the order in which the PGP-HRC model was built. The hydraulic model is introduced, and the underlying theory is explained in Chapter 2. The hydraulic model was tested under different rainfall intensities, initial soil water content, and parameter sensitivity analysis to understand the hydraulic process of the PGP system in Chapter 3. The mesh

independence study, time-step and error estimation of the model solver has also been discussed in Chapter 3. In Chapter 4, issues with the representation of the upper boundary layer were resolved through the development of a Rainfall-infiltration boundary (RIB) method. The suspended particle clogging model is presented in Chapter 5, in which the method, physics and mathematical approach are explained. The PGP-HRC model was applied under different rainfall intensities, suspended particle concentration and suspended particle size in Chapter 6 to evaluate the clogging process of the PGP system. In Chapter 7, the application of the PGP-HRC model for PGP systems is discussed, and providing suggestions for PGP system design maintenance, along with a summary of model limitations and ideas for future research. Chapter 8 is the conclusion of the thesis.

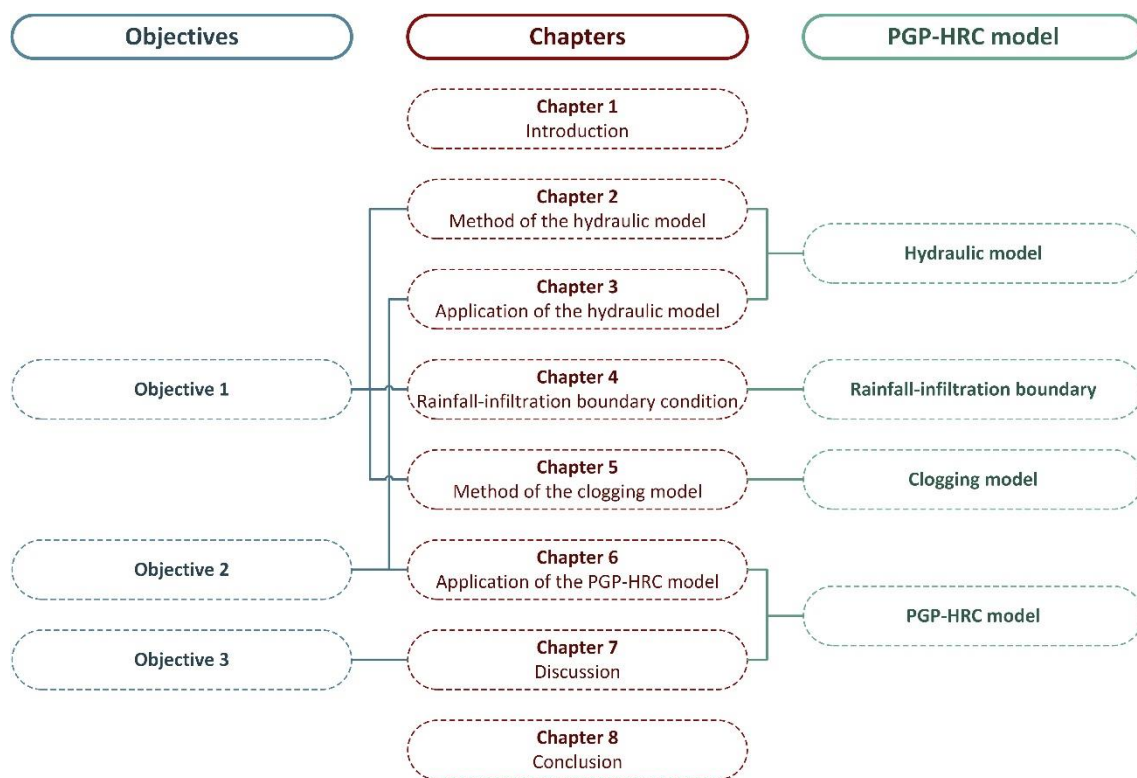


Figure 1-7 Thesis structure and the association between objectives, chapters and model components

2 Chapter Two: Method of the hydraulic model

2.1 Introduction

Typically, a PGP structure contains soil, sand, gravel and geotextile layers in the vertical direction. These materials have similar microstructures, approximate to capillary bundles with different porosities. Driven by pressure differences, water can be infiltrated through these microstructures.

According to the natural wetting-drying patterns (affected by rainfall, solar radiation and other weather conditions) and the characteristic of the materials, the PGP system can be considered an unsaturated multi-phase porous media. Thence, the equations for describing water flow in the aquifer will be sufficient to simulate the hydraulic phenomenon in the PGP system.

Driven by gravitational force, the movement of pollutants through the PGP system follows the water flow. Therefore, the hydraulic performance model is the foundation of the long-term PGP clogging model. This chapter presents the development of a 1D soil-sand-geotextile column in COMSOL to understand the soil behaviours affected by the initial wet or dry condition.

2.1.1 Aim

The velocity profile is the driven force for the movement of suspended particles in the clogging model, and it will dramatically affect the time for fully clogged. The aim of the hydraulic model is to provide the velocity profile for the long-term simulation clogging model, and it is also the first part of the PGP-HRC model.

2.1.2 Overview of approach

A theory-based mathematical model has been built up using COMSOL Multiphysics. The model contains three layers: a soil layer, a sand layer, and a geotextile layer. To represent the water flow behaviours in the PGP system, the model has been tested under different initial conditions and inlet rainfall velocity. Sensitivity analysis has also been done for the simulation, which helps understand how the model responds to various parameters changes.

2.2 Model method

The hydraulic model has considered the widely used hydraulic behaviour equation for saturated and unsaturated subsurface flow. Richards equation has been used for unsaturated flow and Darcy law for saturated flow.

2.2.1 Principles of water movement in soil

Water flows through the pores between soil particles can be assumed as capillary flow through porous media. The motion of liquid flows in unsaturated porous media produced by gravity, and the pressure gradient force follows ordinary hydrodynamic laws.

Richards equation was used to represent the saturated-unsaturated flow behaviours of water in porous media. The air inside porous media is connected to the atmosphere, so pressure changes in the air on the flow can be neglected. Therefore, the unsaturated liquid flow in porous media can be considered as a one-phase flow.

Richards equation is a nonlinear partial differential equation derived from Darcy's law, the 1D vertical flow equation reads (Richards, 1931)

$$\frac{\partial \theta}{\partial t} = \frac{\partial}{\partial z} \left[K(\theta) \left(\frac{\partial H_p}{\partial z} + 1 \right) \right] \quad (2-1)$$

Where θ is the liquid volume fraction; K is the hydraulic conductivity [m s^{-1}], H_p is the pressure head [m], and z equals the vertical elevation [m], t is time [s].

Soil properties (including porosity, residual liquid volume fraction and hydraulic conductivity), inlet velocity, and boundary conditions are required to describe water flow performance.

Soil Water Characteristic Curve (SWCC) shows volume water fraction plotted against suction potential and can be used to determine unsaturated soil mechanics. van-Genuchten equations can estimate SWCC and understand the relationship between suction pressure, soil saturation and hydraulic conductivity (van Genuchten, 1980).

SWCC indicates the relationship between liquid volume fraction (θ) and soil water potential (ψ). When the porous media is saturated, θ equals porosity, and ψ close to zero, water is mainly held in the media by capillary forces. As the water content decreases, the binding force of water gets stronger. At the same time, the potential becomes smaller, water firmly trapped in the smallest pores.

Soil properties, such as field capacity, permanent wilting point, water storage and aggregate stability, can be predicted by SWCC. Field capacity is water content in the soil after the overstock water has drained out. A permanent wilting point is a minimum amount of water in the soil that plants must survive (residual water content); lower than this point, plants will wilt permanently.

van-Genuchten model can characterise the shape of SWCC as follows (van Genuchten, 1980)

$$\theta(\psi) = \theta_r + \frac{\theta_s - \theta_r}{[1 + (\alpha|\psi|^n)]^{1-1/n}} \quad (2-2)$$

$\theta(\psi)$ expresses the SWCC, $|\psi|$ is the suction pressure [cm] of water, θ_s and θ_r is the saturated and residual liquid volume fraction separately, α [L^{-1} or cm^{-1}] and n are van-Genuchten parameters ($\alpha > 0$; $n > 1$).

The van-Genuchten parameters (α and n) can be determined by experiments, such as the instantaneous profile method (Watson, 1966). Previous researchers have measured van-Genuchten parameters highly related to soil texture. Thus different van-Genuchten should be considered for the soil and sand layer in the hydraulic model.

2.2.2 Geotextile selection for the PGP system

Geotextiles are typically made from synthetic materials, such as polypropylene and polyester. These synthetic materials themselves are well known for the properties of hydrophobic (Koerner, 1997). Depending on fabric density, geotextile can be used either as a capillary barrier or drainage. When the geotextile is applied for drainage function, the vicinity soil is saturated with positive pore pressure. Otherwise, when the geotextile is used as a capillary

barrier, the surrounding soil is unsaturated and porewater in a suction state (Iryo and Rowe, 2004).

Woven geotextile can be considered as a regular porous media (Stormont and Morris, 2000). A woven geotextile with a large opening area allows soil contact across the geotextile. Therefore, the effect of the geotextile on unsaturated water movement can be neglected. Based on previous experimental research, for a woven geotextile with a small opening area, water cannot flow through the geotextile until the pressure reaches positive values (Stormont and Morris, 2000).

Typically, the PGP system requires relatively large saturated hydraulic conductivity, and the nonwoven geotextile will be a suitable material (Stormont and Morris, 2000). Unlike woven geotextile, the pore size of nonwoven geotextile is larger than most soil types, resulting in similar hydraulic behaviours to coarse sand (Ho, 2000; Stormont and Morris, 2000).

2.2.3 Principles of water movement in nonwoven geotextile

Some papers suggested that geotextile acting as a capillary barrier that restricts vertical water flow (Richardson, 1997) requires more than 10 mm pressure head to generate an infiltration across geotextile (Dierickx, 1996). Henry (1995) reported that because of the wetting and pore properties, geotextile could block capillaries. Stormont and Morris (2000) demonstrated that nonwoven geotextile tends to restrict water flow in the adjacent unsaturated soil until the suction head reaches a low value. However, other papers suggest differently that nonwoven geotextile can be used as an effective drainage material. Tan *et al.* (2001) noted that nonwoven geotextile could reduce excess pore pressure and produce water flow through geotextile. Water can also drain out from saturated soil during soil consolidation (Giroud, 1983).

Water drainage of nonwoven geotextile is also related to soil saturated hydraulic conductivity (K_{sat_s}) and geotextile (K_{sat_g}). When the nonwoven geotextile and surrounding soil are nearly saturated, it works as a drainage material ($K_{sat_g} > K_{sat_s}$) (Iryo and Rowe, 2003). Otherwise, once the soil and nonwoven geotextile become unsaturated, K_{unsat_g} becomes several orders of magnitude less than

K_{unsat_s} and causes retardation of water flow (Iryo and Rowe, 2003) (K_{unsat_g} and K_{unsat_s} are the unsaturated hydraulic conductivity for geotextile and soil).

Conventionally, for drainage application, the saturated hydraulic conductivity of nonwoven geotextile must be larger or at least equal to soil (i.e. $K_{sat_g}/K_{sat_s} \geq 1$) (Ho, 2000). However, for the crucial requirement of high infiltration, the ratio is recommended to be larger than 10 (Holtz, Christopher and Ryan, 1997). Siemens and Bathurst (2010) modelled the hydraulic performance of a 1D sand-geotextile column. They indicated that no ponding condition requires a ratio of soil and geotextile permittivity to be less than 0.01 ($\kappa_s/\kappa_g \leq 0.01$, permittivity is a term that relates the geotextile saturated hydraulic conductivity with thickness).

The Geotextile Water Characteristic Curve (GWCC) is usually used to describe the relationship between unsaturated suction head and saturation. Previous research described that GWCC is steeper than the typical SWCC for sand, and the nonwoven geotextile behaves approximately to a uniform coarse soil (Stormont, Henry and Evans, 1997; Stormont and Morris, 2000; Iryo and Rowe, 2003; Bathurst, Ho and Siemens, 2007; Siemens and Bathurst, 2010).

Ho (2000) measured GWCC with a sand-geotextile column for nonwoven geotextile for wetting and drying paths. It has been concluded by Ho (2000) that the GWCCs are very steep, which indicates the geotextile saturation change rapidly due to a tiny change in suction pressure.

Previous researchers frequently applied the Richards equation for unsaturated sand-geotextile column hydraulic performance modelling (Richards, 1931) as the governing equation. Siemens and Bathurst (2010) used the most generalised model for soil and geotextile water characteristic curves developed by Fredlund and Xing (1994). The van-Genuchten model (van Genuchten, 1980) was used by Iryo and Rowe (2003) and Stormont and Morris (2000) for modelling the GWCC, and Ho (2000) applied Brooks and Corey equation (Brooks and Corey, 1964).

According to the previous research, it can be assumed that the changes between saturation and no water flow of nonwoven geotextile (sharp changes of the GWCC) can be neglected for long term simulation. Water can only flow through

when the geotextile and surrounding soils are nearly saturated (Iryo and Rowe, 2003). The hydraulic performance through nonwoven geotextile can be assumed as saturated water flow, Darcy's law can be used for the hydraulic simulation for geotextile.

Darcy's law was derived from water flow experiments through the sand column. As one of the fundamental relationships of hydrogeology, Darcy's law can be directly applied to a homogeneously porous media with a constant fluid viscosity. Darcy's law can also be used for slow and viscous flow, such as any flow with a Reynolds number less than one (laminar flow). However, experiments show that Darcy's law will still be valid with Reynolds number up to 10 under a particular circumstance (such as groundwater flow).

Refined by Masoodi *et al.* (2012), the original Darcy's law (Darcy, 1856) can be written in the form:

$$q = -\frac{K}{\rho_f g} \nabla p \quad (2-3)$$

Here q is the flux [$\text{m}^2 \text{s}^{-1}$], ρ_f is liquid density [kg m^{-3}], K is the hydraulic conductivity [m s^{-1}], g is the acceleration of gravity [m s^{-2}], ∇p denotes the pressure drop from point a to b in the distance ($\nabla p = p_b - p_a$).

2.2.4 Modelling tool (COMSOL Multiphysics)

Numerical simulation for the 1D soil-sand-geotextile column was carried out using the finite element program COMSOL. Since Richards equation and Darcy's law are basic fluid mechanics equations for porous media, the model can be set up using the COMSOL model wizard. Furthermore, the "van-Genuchten" (Section 2.2.1) has been chosen for the retention model.

2.2.5 Geometry

As stated in Section 1.12.1, the model contains three layers: a 5 cm topsoil, a 3 cm laying course, and a 0.43 cm geotextile layer.

According to the function and application, sandy loam (suitable for vegetation), sand (usually used as laying course), and Nonwoven geotextile (discussed in section 2.2.2) have been suggested for the PGP system's basic design by PGP design guidance (Ballard *et al.*, 2015).

2.3 Mathematical approach

As described in sections 2.2.1 and 2.2.3, Richards equation with van-Genuchten retention model has been applied for simulating the water movement in sandy loam and sand layer, and Darcy law for nonwoven geotextile layer.

2.3.1 Water movement in sandy loam and sand layer

Given by Richards (1931), the governing equation (Richards equation) in pressure head can be written as (COMSOL, 2018)

$$(W + SeS) \frac{\partial H_p}{\partial t} + \nabla \cdot (-K \nabla (H_p + z)) = 0 \quad (2-4)$$

Where W is the specific moisture capacity [m^{-1}], Se is the effective saturation of porous media [-], S is the storage coefficient [m^{-1}], z is the length or depth along the direction [m], for a 1D vertical model, z equals to the vertical elevation.

In Equation (2-4), W defines changes in water storage due to changes in liquid volume fraction. Since W , the first term of time-coefficient in equation (2-4) becomes zero at saturation, water storage over time is related to the compression of porous media and water under saturated conditions. As represented in the second term of the time-coefficient equation (2-4), saturated storage occurs with effective saturation.

The relationship between the hydraulic head (H), pressure head (H_p), pressure (p) and vertical elevation (z) given by (COMSOL, 2013)

$$H_p = \frac{p}{\rho_f g}; \quad H = H_p + z \quad (2-5)$$

Also, the permeability k_s [m^2] is related to hydraulic conductivity K [m s^{-1}] and liquid viscosity μ [$\text{Pa}\cdot\text{s}$] by (Richards, 1931)

$$\frac{k_s}{\mu} = \frac{K}{\rho g} \quad (2-6)$$

The saturated liquid volume fraction θ_s can be expressed by an equation of effective liquid volume fraction θ_e and residual liquid volume fraction θ_r , their relationship can be expressed as

$$\theta_s = \theta_e + \theta_r \quad (2-7)$$

In variable saturation flow, fluid can flow through the porous media, but it may or may not thoroughly fill up the pores in the soil depending on the moisture capacity. The coefficients W , Se and K change with H_p and θ_e resulting in a nonlinear Richards equation.

The specific moisture capacity W corresponds to the pressure head with soil moisture changes by (COMSOL, 2013)

$$W = \frac{\partial \theta_e}{\partial H_p} \quad (2-8)$$

For a time depending equation, equation (2-8) can be written as

$$\frac{W \cdot \partial H_p}{\partial t} = \frac{\partial \theta_e}{\partial t} \quad (2-9)$$

Besides, hydraulic conductivity (K) defines the ability to transfer fluid in porous media. The hydraulic conductivity (K) of porous media increases with increasing fluid content by (COMSOL, 2013)

$$K = K_s k_r \quad (2-10)$$

Here K_s is the saturated hydraulic conductivity [m s^{-1}], k_r is the relative permeability of the soil [-].

Varying with the solution of pressure head (H_p), the van-Genuchten model has been used to describe the retention and permeability properties (θ_e , W , Se , k_r) and of porous media as follow (van Genuchten, 1980; COMSOL, 2013)

$$\theta_e \begin{cases} \theta_r + S_e(\theta_s - \theta_r) & H_p < 0 \\ \theta_s & H_p \geq 0 \end{cases} \quad (2-11)$$

$$S_e \begin{cases} \frac{1}{[1 + |\alpha H_p|^n]^m} & H_p < 0 \\ 1 & H_p \geq 0 \end{cases} \quad (2-12)$$

$$W \begin{cases} \frac{\alpha m}{1-m}(\theta_s - \theta_r)S_e^{\frac{1}{m}}\left(1 - S_e^{\frac{1}{m}}\right)^m & H_p < 0 \\ 0 & H_p \geq 0 \end{cases} \quad (2-13)$$

$$k_r \begin{cases} S_e^l \left[1 - \left(1 - S_e^{\frac{1}{m}}\right)^m\right]^2 & H_p < 0 \\ 1 & H_p \geq 0 \end{cases} \quad (2-14)$$

Where,

$$m = 1 - \frac{1}{n} \quad (2-15)$$

α , n , l are the van-Genuchten parameters depending on the porous media properties. The values can be obtained by experiments or estimated by soil texture,

When the fluid pressure reaches atmosphere pressure ($H_p = 0$), the porous media is fully saturated, and the retention and permeability properties (θ_e , W , Se , k_r) become constant values.

In this simulation, the porosity and residual volume fraction of fluid affect the storage. The storage coefficient can be expressed as

$$S = \frac{\theta_s - \theta_r}{1[m]\rho_f g} \quad (2-16)$$

2.3.2 Water movement in nonwoven geotextile

Darcy's law has been used to describe one-dimensional fluid flow as (COMSOL, 2018)

$$\frac{\partial(\theta_s \rho_f)}{\partial t} + \nabla \cdot (\rho_f v) = 0 \quad (2-17)$$

Where ρ_f denotes water density [kg m^{-3}], v is the Darcy velocity, ∇p denotes the pressure gradient [Pa], and it can be expressed as (COMSOL, 2018)

$$v = -\frac{K}{\rho_f g} (\nabla p + \rho_f g \nabla z) \quad (2-18)$$

For a time variable model, equations (2-17) and (2-18) can be inserted into a continuity equation as (COMSOL, 2018)

$$Q_m = \frac{\partial}{\partial t} (\rho_f \theta_s) + \nabla \cdot \rho_f \left[-\frac{K}{\rho_f g} (\nabla p + \rho_f g \nabla z) \right] \quad (2-19)$$

Q_m is a mass source term [$\text{kg m}^{-3} \cdot \text{s}^{-1}$].

The nonwoven geotextile thickness affects both the model geometry and the saturated hydraulic conductivity. The thickness of the geotextile will be reduced with an increasing vertical confined pressure. Ho (2000) measured the new geotextile reduction, and part of the result can be found in Table 2-1.

Table 2-1 Nonwoven geotextile thickness under different vertical confined pressure (Source: Ho, 2000)

<i>Confined pressure [kPa]</i>	<i>Nonwoven geotextile thickness [mm]</i>
0	4.32 (0% reduction)
2	4.28 (1% reduction)
5	4.19 (3% reduction)
10	4.02 (7% reduction)
16	3.80 (12% reduction)
21	3.72 (14% reduction)
26	3.63 (16% reduction)
30	3.54 (18% reduction)
61	3.28 (24% reduction)

Kothari and Das (1992) have reported an equation for modelling the relationship between confined pressure and nonwoven geotextile thickness during loading (Equation (2-20)) and unloading (Equation (2-21)) separately.

$$\text{Loading} \quad \frac{a}{a_0} = 1 - \alpha_a \ln \left(\frac{\sigma}{\sigma_0} \right) \quad (2-20)$$

$$\text{Unloading} \quad \frac{a}{a_f} = \left(\frac{\sigma}{\sigma_f} \right)^{-\beta_a} \quad (2-21)$$

Where a is the nonwoven geotextile thickness under a certain confined pressure [m], a_0 is the initial thickness without any confined pressure [m], a_f is the final geotextile after unloaded [m]. σ is the confined pressure applied on the geotextile [kPa]; σ_0 and σ_f is the initial confined pressure ($\sigma_0 > 0$, [kPa]) and final confined pressure ($\sigma_f > 0$, [kPa]) respectively. α_a and β_a are fitting parameters [-], and the value has been measured by Kothari and Das (1992).

In general, with limited available data, Equation (2-20) and Equation (2-21) can be used to estimate the relationship between confined pressure and nonwoven geotextile thickness. However, the measured geotextile thickness data from Ho (2000) did not perfectly fit with the geotextile thickness loading model (Equation (2-20)) but still followed the general trend. This uncertainty might come from the

gap between the data point and the limitation of available data from Ho (2000). The company's different materials and production methods can also generate unstable data.

For a soil-geotextile column, the vertical confined pressure [kPa] applied on the geotextile (σ_v) can be calculated by the unit weight of soil above the geotextile (γ) and the thickness of the soil layer (a_s) using Equation (2-22).

$$\sigma_v = \gamma \times a_s \quad (2-22)$$

It is crucial to notice that the unit weight of soil will change with the soil saturation. When the soil is saturated with water, the soil saturated unit weight (γ_{sat}) is larger than soil dry unit weight (γ_d). Some of the soil's typical values in their natural state have been determined by Peck, Hanson and Thornburn (1974) and shown in Table 2-2.

Table 2-2 Some of the typical values of the dry and saturated unit weight of soil in their natural state (Source: Peck, Hanson and Thornburn, 1974)

<i>Soil type</i>	<i>Porosity</i>	<i>Dry unit weight (γ_d) [kN m⁻³]</i>	<i>Saturated unit weight (γ_{sat}) [kN m⁻³]</i>
Uniform sand (loose)	0.46	14.1	18.5
Uniform sand (dense)	0.34	17.1	20.4
Well-graded sand (loose)	0.40	15.6	19.5
Well-graded sand (dense)	0.30	18.2	21.2
Windblown silt (loose)	0.50	13.4	18.2
Glacial till	0.20	20.7	22.8
Soft glacial clay	0.55	11.9	17.3
Stiff glacial clay	0.37	16.7	20.3
Soft slightly organic clay	0.66	9.1	15.4
Soft very organic clay	0.75	6.8	14.0
Soft montmorillonitic clay	0.84	4.2	12.6

The nonwoven geotextile thickness of the hydraulic model can be estimated by Equation (2-20), Equation (2-22) and Table 2-2. The pressure difference exerted on the geotextile during the loading process can be obtained from the pressure

increase during soil wetting. σ can be calculated by the dry unit weight and σ_0 equals the saturated unit weight (Table 2-2). Although the sandy loam data is not included in Table 2-2, similar texture or similar porosity can be used as an alternative. The difference caused by this replacement is so tiny that it can be neglected. The soil and sand layer of the PGP system is thin, the change in geotextile thickness caused by the confined pressure change is also minimal. Thus, the nonwoven geotextile thickness has reduced from 4.32 mm to 4.30 mm.

The saturated hydraulic conductivity (K_{sat_g}) of geotextile is related to the geotextile thickness under confined pressure. An increasing confined pressure applied to the geotextile will cause a decrease in geotextile thickness. Palmeira and Gardoni (2000) have reported that for nonwoven geotextile the K_{sat_g} value will reduce with a decreasing geotextile thickness, and the decline trend also supported by Ho (2000).

Depending on the production method and materials, geotextile's saturated hydraulic conductivity varies in an extensive range of values from 8.3×10^{-6} to 0.002 m/s (Koerner, 1997).

Since the saturated hydraulic conductivity is related to the geotextile thickness under confined pressure, another parameter, permittivity (κ_g), is introduced to describe the relationship between K_{sat_g} and geotextile thickness (a). The relationship can be expressed as below (Ho, 2000).

$$\kappa_g = \frac{K_{sat_g}}{a} \quad (2-23)$$

Ho (2000) measured the permittivity (κ_g) value of woven and nonwoven geotextile for water permeability, the test result has shown in Table 2-3.

Table 2-3 Permittivity (κ_g) of woven and nonwoven geotextile (produced by Amco Fabrics and Fibers Company), Source: (Ho, 2000)

<i>Geotextile type (Mode)</i>	<i>Permittivity κ_g (s^{-1})</i>
Polypropylene woven geotextile (Amoco 2044)	0.359
Polypropylene nonwoven geotextile (Amoco 4516)	0.382

Without extra pressure, the thickness under confined pressure (caused by soil and sand layer gravity on the top of geotextile) has been assumed to be 4.30 mm, K_{sat_g} can also be calculated using Table 2-3 and Equation (2-23).

2.3.3 Model summary

The flow chart for the hydraulic model of the PGP-HRC model is shown in Figure 2-1. The dependent variable is pressure (p) for the subsurface flow interface (Darcy's law and Richards equation) in COMSOL Multiphysics. Other output data can also be calculated automatically by COMSOL.

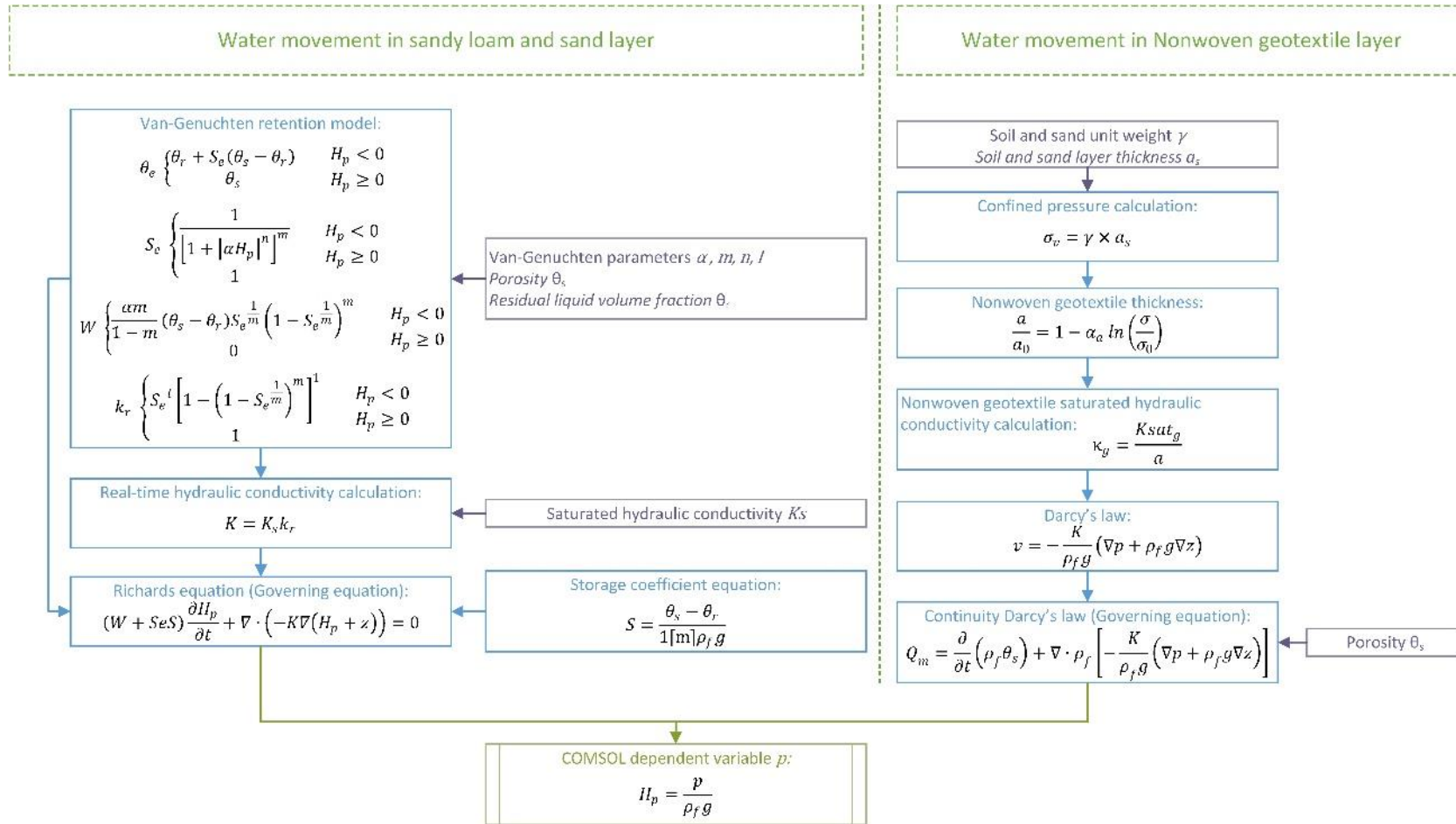


Figure 2-1 Model flow chart for the hydraulic model for the PGP system, as part of the PGP-HRC model.

3 Chapter Three: Application of the hydraulic model

This chapter purposed to apply the hydraulic model with different input parameters, such as initial conditions and rainfall intensity. Chapter 2 has introduced the mathematical approach of the hydraulic part of the PGP-HRC model. However, data, parameters and boundary conditions used for the hydraulic model are described in this chapter. Different tests and sensitivity analysis have also been designed in this chapter for the hydraulic model to understand the water flow performance in the PGP system under different environmental circumstances. The structure of the Richards equation and van-Genuchten model used for the PGP-HRC hydraulic model are embedded within the COMSOL subsurface flow module.

3.1 Model parameterisation

As presented in Chapter 2, the Richards equation and Darcy's law have been used for the soil/sand and geotextile layers, respectively. The geometry and parameters used for the hydraulic model tests are presented here.

3.1.1 Geometry

The model has been applied to a three-layer geometry with sandy loam, sand and nonwoven geotextile. The cross-section structure of the simulated PGP system has shown in Figure 3-1. Water infiltrates downward through the PGP system driven by gravity. Since gravel's porosity and hydraulic conductivity are much higher than the soil, sand and nonwoven geotextile, the water behaviour can be assumed as free-drainage in the gravel layer.

Evapotranspiration has not been directly reflected in the model. By changing the initial water content, the dry soil after evapotranspiration can be simulated indirectly.

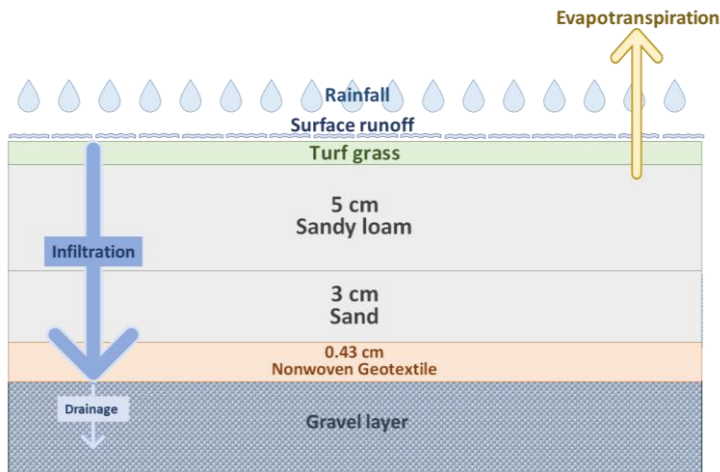


Figure 3-1 Cross-section of three layers structure for simulating the hydraulic performance of soil, sand and geotextile. The water flow behaviours are also shown in the figure.

3.1.2 Table of parameters

Table 3-1 Input parameters for soil, sand and clean nonwoven geotextile

<i>Parameter</i>	<i>symbol</i>	<i>Sandy loam</i>	<i>Sand</i>	<i>Geotextile</i>	<i>Unit</i>
The saturated liquid volume fraction (porosity)	θ_s	0.39	0.375	0.86	-
The residual liquid volume fraction	θ_r	0.0299	0.053	-	-
Saturated hydraulic conductivity	K_s	4.92×10^{-6}	7.47×10^{-5}	1.64×10^{-3}	m s^{-1}
Van-Genuchten parameter	α	4.46	3.55	-	m^{-1}
Van-Genuchten parameter	n	1.3862	3.16	-	-
Thickness	a	50	30	4.30	mm
Permittivity	κ	-	-	0.382	s^{-1}
Source	-	Calculated from ROSETTA model ¹ (Schaap, Leij and van Genuchten, 2001)	(Schaap and Leij, 1998)	(Ho, 2000)	-

¹ ROSETTA model computer program for predicting the water retention parameters, described in Section 5.2.3.1.

3.1.3 Matrix properties for sandy loam and sand layer

The parameters required for Richard's equation include saturated hydraulic conductivity K_s , Saturated liquid volume fraction θ_s (Porosity), and Residual liquid volume fraction θ_r (Residual porosity). Table 3-1 shows the parameters required for the sandy loam and sand hydraulic model. Data were obtained from the average value of UNSODA, AHUJA and RAWLS unsaturated soil hydraulic database, calculated by Schaap and Leij (1998).

3.1.4 Nonwoven geotextile model parameters

The experimental data measured by Ho (2000) was used for the geotextile hydraulic behaviours modelling in this paper. Ho (2000) investigated a 1D sand-geotextile column's hydraulic performance with a single horizontal layer of woven and nonwoven geotextile. The geotextile thickness, permittivity (Table 2-3), porosity, and GWCC data were determined by Ho (2000).

The properties of the nonwoven geotextile specimen that was used by Ho (2000) are listed below:

- Mass per unit area (M.A.): represents the area density, 468 [g m⁻²].
- Apparent opening size (AOS): indicates the size of the holes in geotextile, 0.15 [mm].

As shown in Table 2-1, the original thickness of the nonwoven geotextile specimen is 4.32 mm. The confined pressure under a soil layer and a sand layer can be assumed using Equation (2-22) and the well-graded sand (dense) unit weight data in Table 2-2 (Section 2.3.2).

3.2 Initial and boundary condition

3.2.1 Initial condition

The initial water content of the hydraulic model test has been set to three conditions: saturated, field capacity and permanent wilting point.

When the soil is saturated, the pressure head is close to 0. The initial pressure head value was set to -0.01 cm in this model to prevent non-convergence during

calculation. After the water drains out from the soil freely, the pressure head will drop to -8.43 cm (the depth of the soil) and reach field capacity. The water content will stay the same until drying processes like evaporation or transpiration happen. However, the drying processes have not been included in the PGP-HRC model. Under extreme conditions, the pressure head can reduce to -150 m for a permanent wilting point.

3.2.2 Upper and lower boundary conditions

In the PGP system, a gravel layer will lay underneath the geotextile (Figure 3-1). Compared to the top three layers, gravel has much larger hydraulic conductivity. Therefore, the lower boundary can be assumed as free flow.

Free-drainage boundary conditions barely have resistance to water being excluded from the structure. The pressure head is assumed to be zero pressure as atmospheric pressure (pressure head equals 0), so that flow only occurs when a positive pressure (saturation) occurs in the porous media. A free flow boundary condition was added to the outlet point at the bottom to simulate the free-drainage condition at the lower boundary.

The upper boundary is an input rainfall velocity changing with time. The inlet point of the model is the soil surface.

3.3 Model time steps and mesh

The finite element method (FEM) used by COMSOL to solve Partial Differential Equations (PDE) is required to subdivide a large system into smaller, simpler parts called finite elements. A specific spatial discretization achieves this in the spatial dimension by building a mesh of an object. These meshes constitute the numerical domain of the solution, which has a finite number of points. The FEM formulation of the boundary value problem ultimately yields a system of algebraic equations, and these simple equations are then combined into a larger system of equations to model the entire problem (Logan, 2015). FEM approximates the solution by minimising the relevant error function by the calculus of variations (Logan, 2015). In COMSOL, the geometry of an object can be defined, and mesh can be built manually or automatically.

In COMSOL, the solver can be a Stationary Study or a Time-Dependent Study depending on the problem condition. A steady-state situation uses a Stationary Solver, while a Time-Dependent Solver should be used for a time-dependent or transient PDE solution over time (COMSOL, 2019).

3.3.1 The Time-Dependent Solver

The Time-Dependent Solver was used to compute unsteady flow, pressure field, species composition, and reaction kinetics for the PGP-HRC model (including physics of fluid flow, species transport and reactions).

COMSOL Multiphysics uses the Back Differential Formulation (BDF) for the time-dependent solver, known for its stability and used for a long time (Calvo, Montijano and Rández, 1993). BDF is an implicit solver based on backward differentiation formulas with adaptive integration order of accuracy and step-size selection, first developed by Curtiss and Hirschfelder (1952).

The implicit time-stepping scheme for adaptive time-stepping satisfies specified tolerances and local error estimates (COMSOL, 2019). The time step can be specified for each model run. Failures are recorded based on the associated simulation error estimates related to tolerances (COMSOL, 2019). Additionally, if the algebraic solver does not converge, the requested time step will be reduced (COMSOL, 2019). The nonlinear solver used was controlled by segregated solver sub-nodes, allowing the solution process to be divided into sub-steps to have much control over the nonlinear solution process (COMSOL, 2019).

The adaptive and iterative accuracy requirements are considered for solving the finite element discretization, and it is subjected to relative and absolute tolerances, which are time-step (solver) error and algebraic equations (solver) error (Söderlind and Wang, 2006; Bortone *et al.*, 2020). The adaptive BDF time-stepping scheme adapts to both the order and the time-step size, helping to minimize simulation errors while maintaining the desired accuracy (COMSOL, 2019).

For all tested models of the PGP-HRC model, the relative algebraic error and the relative residual error estimation for each time step are lower than the order of

10⁻¹². After each simulation, no failures of adaptive step size and nonlinear algebraic solvers have been indicated.

3.3.2 Mesh independence study

COMSOL Multiphysics uses the finite element method (FEM) to solve partial differential equations numerically. FEM solves a problem by discretizing space and constructing mesh for a particular geometry. By assembling the algebraic equations of each boundary and element in the domain, FEM approximates the solution by minimizing the correlation error functions using the calculus of variations (Logan, 2015).

In COMSOL Multiphysics, the mesh generator discretizes the domains into minor elements with mesh vertices at the endpoints (COMSOL, 2019). Meanwhile, the Physics-Controlled mesh (default setting) automatically generates the mesh sequence, which adapts to the current model physics interface settings (COMSOL, 2019).

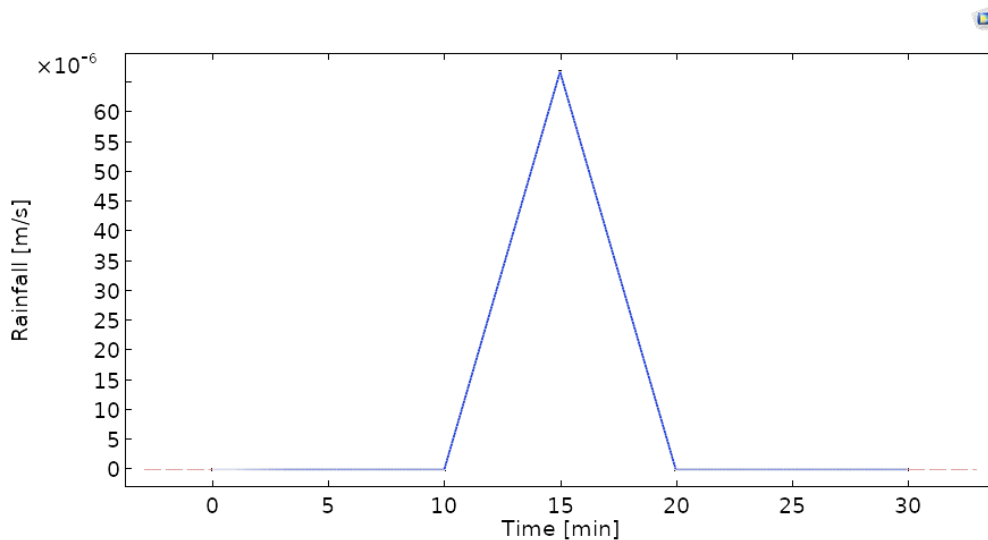
The element size consists of nine size settings for the Physics-Controlled mesh, varying from extremely fine (0.083 cm element size) to extra coarse (0.2579 cm element size). The element size estimation was based on a one-dimensional 8.43 cm geometry. For a 1D model, the domain was divided into roughly equal-sized elements.

Four of the mesh settings have been selected to determine the effects of mesh size on model results. These four settings include two extreme settings (Extremely fine and Extra coarse), a default setting (Normal), and one picked up in a larger interval (Finer). The settings and their statistics are shown in Table 3-2. The Extremely coarse mesh setting was not considered for this study because its element size is too large to effectively cover the variation within the layers.

Table 3-2 Statistics for mesh test settings

<i>Mesh setting</i>	<i>Extremely fine</i>	<i>Finer</i>	<i>Normal</i>	<i>Extra coarse</i>
Mesh vertices	102	29	17	8
Number of elements	101	28	16	7
Maximum element size [cm]	0.0843	0.312	0.565	1.69
Element length ratio	0.969	0.688	0.7739	0.2579
Mesh length [cm]	8.43			

The three-layer hydraulic model was used for the mesh independence study (parameters shown in Section 3.1.2). A 30-minute rainfall event was used as the inlet velocity (Figure 3-2).

**Figure 3-2 Rainfall intensity for mesh independence study**

The mesh independence results have been demonstrated in Figure 3-3. As an important changing variable, the Darcy velocity was chosen to indicate the model's response to the mesh element size. The four curves in Figure 3-3 show no significant differences, except that the Extra coarse setting caused the Darcy velocity results to deviate from the other settings at 15-20 minutes.

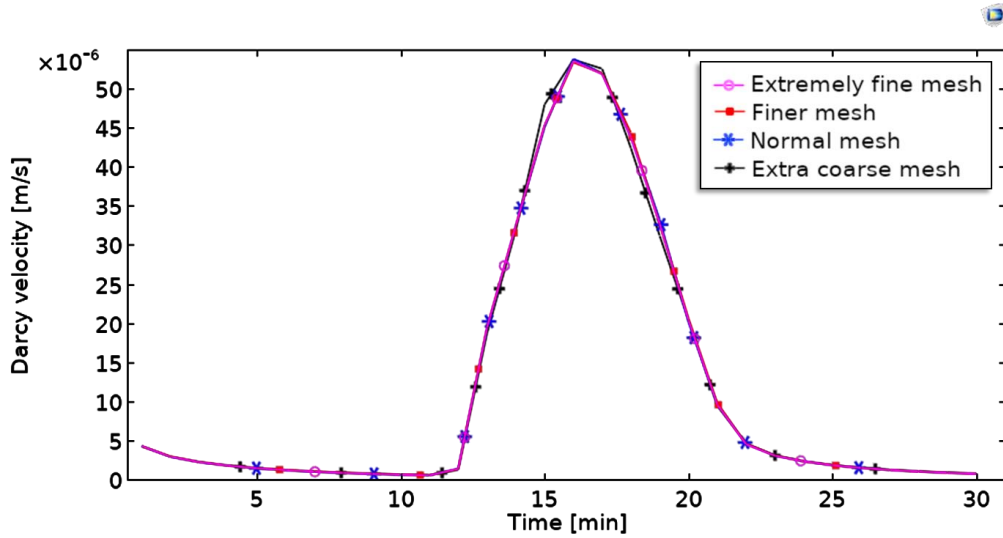


Figure 3-3 Darcy velocity results at outlet point for mesh independence study. Four different physical-control mesh settings have been tested (Extremely fine, Finer, Normal and Extra coarse).

As the mesh independence results indicated (Figure 3-3), the difference in mesh element size does not significantly affect the results for a 1D model. Usually, a smaller mesh element size will increase the computational load and raise the calculation time. However, for a 1D model, the amount of calculation is relatively small. Therefore, the Extremely fine mesh setting will be acceptable for the PGP-HRC model tests for understanding the detailed changes of fluid and particles transported across the media.

3.4 Sensitivity Analysis

The sensitivity analysis aims to understand how Darcy velocity responds to the parameter changes. The sensitivity analysis was carried out for a one-layer model. Parameters analysed are porosity, residual liquid volume fraction, saturated hydraulic conductivity, Van-Genuchten α and Van-Genuchten n .

In the original hydraulic model, the governing equation for the sandy loam and sand layer is the Richards equation, and Darcy's law for the geotextile layer. However, Richards equation is derived from Darcy's law for water movement in unsaturated soil. Therefore, the Richards equation has been tested in the sensitivity analysis for simplification.

3.4.1 Sensitivity analysis parameterisation

A 5 cm sandy loam column has been set up using COMSOL for sensitivity analysis. Table 3-3 presents the values of the tested parameters.

Table 3-3 Input parameters for sensitivity analysis

<i>Parameter</i>	<i>Unit</i>	<i>Sensitivity to θ_r</i>	<i>Sensitivity to θ_s</i>	<i>Sensitivity to α</i>	<i>Sensitivity to n</i>	<i>Sensitivity to K_s</i>
Porosity (θ_s)	-	0.39	0.3 0.4 0.5 0.6 0.9	0.39	0.39	0.39
The residual liquid volume fraction (θ_r)	-	0.040 0.067 0.093 0.120 0.300	0.049	0.049	0.049	0.049
Saturated hydraulic conductivity (K_s)	m s ⁻¹	1.21E ⁻⁵	1.21E ⁻⁵	1.21E ⁻⁵	1.21E ⁻⁵	1.1574E ⁻⁶ 5.787E ⁻⁶ 1.1574E ⁻⁵ 3.7422E ⁻⁵ 7.5231E ⁻⁵
van-Genuchten parameter α	m ⁻¹	3.74	3.74	0.5 1.5 2.5 5 10	3.74	3.74
van-Genuchten parameter n	-	1.74	1.74	1.74	1.2 1.86 2.3 2.8 3.2	1.74

The test range of parameters is decided with the maximum and minimum values obtained from the ROSETTA database (Schaap and Leij, 1998; Schaap, Leij and van Genuchten, 2001). The range in parameter values, some extreme values above the normal range for sand, were selected as end members to increase the visual difference in figures (porosity 0.9, residual liquid volume fraction 0.3 and van-Genuchten parameter α 10 m⁻¹). The tested parameters gradually increase

between the minimum and maximum values (Table 3-3). The results of sensitivity analysis have been generated every 1 minute.

The rainfall for sensitivity analysis is a 30 minutes rainfall event with the highest intensity of 240.48 [mm h⁻¹] at the 15th minute (Figure A-3, Appendix A).

3.5 Tests design

The input data used for model tests design have been described in Table 3-4. Eight test models have been designed with different initial soil pressure head and inlet rainfall intensity (Table 3-4).

The rainfall intensity has been set for three different patterns:

R0: Constant rainfall intensity at 1.08 [mm h⁻¹], a value much lower than the saturated hydraulic conductivity of sandy loam.

R1: Rainfall event 1. A rainfall event lasts for 10 minutes, starting at 60 minutes, and gradually increases until it reaches the largest value at 240.48 [mm h⁻¹], and the rainfall event finish at 70 minutes. Rainfall event 1 is a created extreme event.

R2: Data was modified from rainfall event Cranfield 113 (Figure A-2, Appendix A). An actual rainfall event was obtained from Cranfield weather station on 7th June 2016, which was the heaviest rainfall event in the collected rainfall data. 120 minutes of free drainage (inlet velocity equals 0 [mm h⁻¹]) has been added at the end of the rainfall event.

R3: Constant rainfall intensity at 7.2 [mm h⁻¹], this is a value close to the saturated hydraulic conductivity of sandy loam.

The rainfall intensity for each event has shown in Table A-1 (Appendix A). R0 rainfall can be used as a control group. R1 is an invented short-term rainfall, and R2 is the largest rainfall event (with 5 minutes resolution) amount of data obtained from weather stations. R3 represents a constant rainfall applied to the clogging model in Chapter 7.

Notably, only the top 5 cm sandy loam layer has been applied to the permanent wilting point initial soil pressure head (-150 m) for the dry initial condition simulation (PR0 and PR3). Under the natural environment in the U.K., it is unlikely that the deep soil will reach the permanent wilting point. Still, the surface soil (here is the top 5 cm sandy loam layer) could be dried out caused by evapotranspiration (Figure 3-1). Although the hydraulic model has not considered the evapotranspiration process, the initial soil pressure head at the permanent wilting point represents the dry soil results caused by evapotranspiration.

The operation time lasts 120 minutes for tests PR0, PR1, FR0, FR1, PR0 and PR3. The duration of tests SR2 and FR2 is 465 minutes. The model result was recorded every 1 minute.

The letter S, F and P in each test name represents the initial soil pressure head. S expresses saturated, F means field capacity, and P terms permanent wilting point.

Table 3-4 Model tests design and their input parameters

<i>No.</i>	<i>Initial soil pressure head</i>	<i>Rainfall event</i>	<i>Rainfall duration</i>
SR0	-0.01 cm (saturated)	R0	120
SR1	-0.01 cm (saturated)	R1	120
SR2	-0.01 cm (saturated)	R2	465
FR0	-8.43 cm (field capacity)	R0	120
FR1	-8.43 cm (field capacity)	R1	120
FR2	-8.43 cm (field capacity)	R2	465
PR0	-150 m for sandy loam layer (permanent wilting point)	R0	120
	-8.43 cm for sand and geotextile layer		
PR3	-150 m for sandy loam layer (permanent wilting point)	R3	120
	-8.43 cm for sand and geotextile layer		

3.6 Results

Darcy velocity is usually used to describe the flow in porous media, defined as the flow per unit cross-section area. Darcy velocity is an essential input variable and the main driving force for the clogging model as the water flow phenomenon indicator.

Figure 3-4 is the sensitivity analysis results. Figure 3-5 (d) and Figure 3-7 (b) have plotted the model results against the depth [cm], the 0 cm point of the y-axis denotes the soil surface, and at -8.43 cm point of the y-axis is the bottom of the porous media. From 0 to -5 cm is the sandy loam layer on top, with a 3 cm layer of sand and a 0.43 cm geotextile underneath.

Figure 3-5 (a) (b) (c), Figure 3-6 and Figure 3-7 (a) have plotted the model results against time. For rainfall R0, R1, and R3, the model running time is 120 minutes, and 465 minutes for rainfall R2.

Figure A-1 (Appendix A) and Figure A-2 (Appendix A) described the inlet rainfall velocity for R1 (Rainfall event 1) and R2 (Rainfall event Cranfield).

Based on the lower boundary condition assumption, the pressure head will stay at 0 cm. The hydraulic head equals -8.43 cm at the bottom of the geotextile constantly.

3.6.1 Results of sensitivity analysis

The sensitivity analysis results are shown in Figure 3-4. The yellow line in each graph is the inlet rainfall intensity. Peak Darcy value is lower as increasing porosity, decreasing residual liquid volume fraction and reducing saturated hydraulic conductivity (Figure 3-4 a, b, e). And the peak Darcy velocity extends for a longer period as decreasing K_s (Figure 3-4 e).

Most sensitivity analysis tests only affect a small amount (Figure 3-4 a, b, c, d). However, in the results of sensitivity analysis to K_s , the blue line, though had a significant impact (Figure 3-4 e).

3.6.2 Darcy velocity for consistent rainfall R0

The Darcy velocity results with consistent rainfall intensity are expressed in Figure 3-5. It is crucial to notice that the rainfall intensity of R0 (1.08 [mm h⁻¹]) is smaller than the saturated hydraulic conductivity of the top, sandy loam layer (17.712 [mm h⁻¹]). Which is the main reason causing the difference between Figure 3-5 (a) and (b) at the first 40 minutes of simulation.

The Darcy velocity inside and at the outlet point of Test FR0, SR0 and PR0 reaches the rainfall intensity value after 40 minutes (Figure 3-5 a, b, c). The initial soil conditions are different; however, after only 40 minutes, the soil water balance reaches equilibrium, and the inlet velocity equals the outlet velocity (Figure 3-5 a, b, c).

3.6.3 Model results for rainfall events R1 and R2

The Darcy velocity results of tests SR1, FR1, SR2 and FR2 are shown in Figure 3-6. The largest value of Darcy velocity at the outlet point occurs 1-2 minutes after the rainfall event. The highest value of Darcy velocity at the outlet point is smaller than the highest rainfall intensity (Figure 3-6).

3.6.4 Darcy velocity for dry initial water content test PR3 and PR0

During the first 40 minutes of simulation, compared to test PR3 (Figure 3-7), test PR0 shows a slightly higher Darcy velocity at the outlet point than rainfall intensity (Figure 3-5 c). The lower rainfall intensity for PR0 made it slower to reach the water balance equilibrium (Figure 3-5 c). Also, the water content gradually increases with depth, from the surface (dry condition) to deep in the column (wet condition). However, after 40 minutes for PR0 (Figure 3-5 c) and through the whole profile of PR3 (Figure 3-7), the rainfall infiltrates and drains out from the porous media without any restriction.

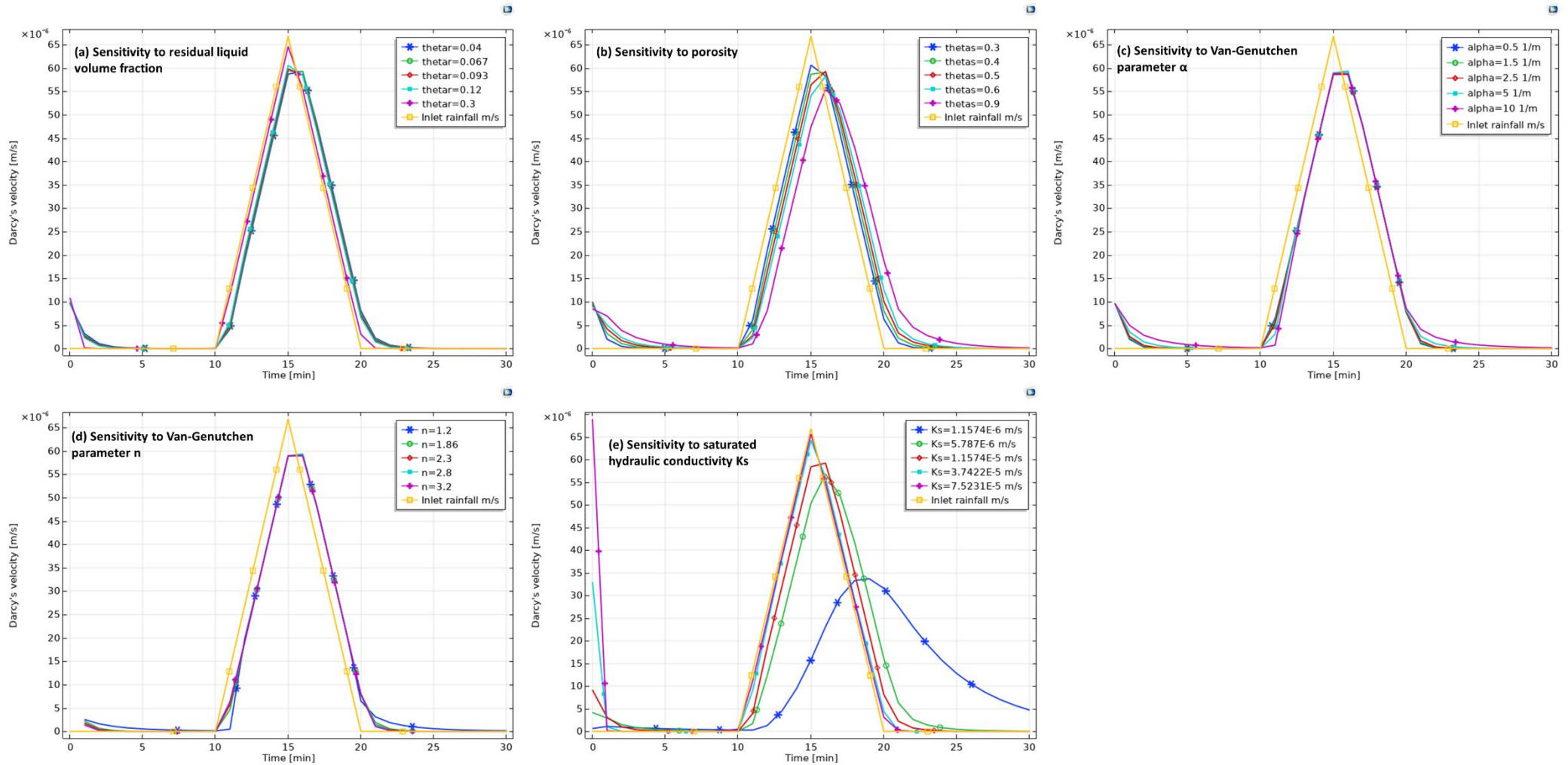
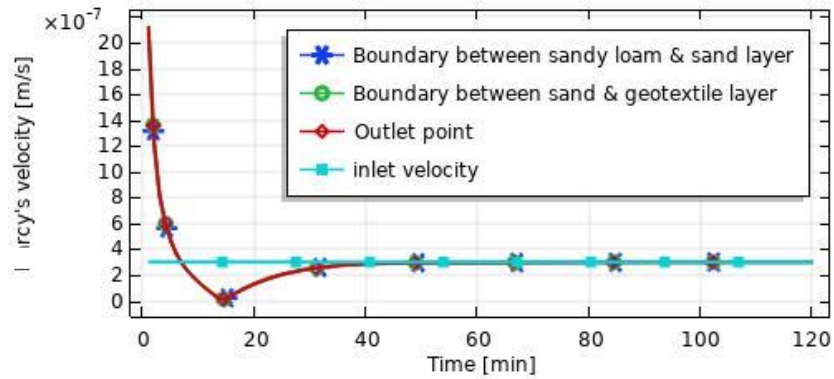
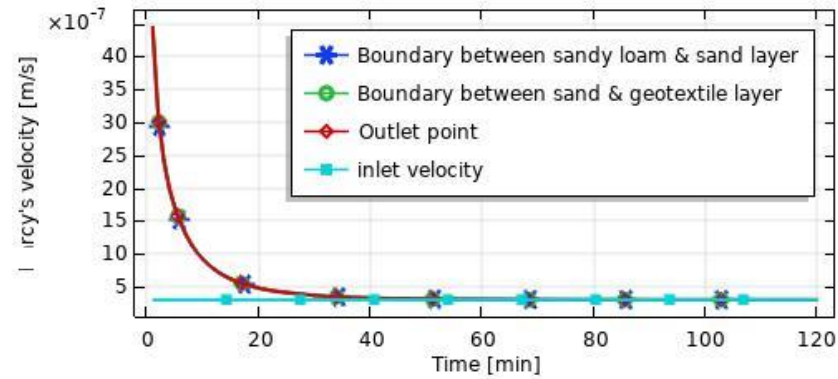


Figure 3-4 Result of modelled Darcy velocity plotted against input rainfall intensity, testing the sensitivity of the model to (a) residual liquid volume fraction, (b) porosity, (c) van-Genuchten parameter α , (d) van-Genuchten parameter n and (d) saturated hydraulic conductivity.

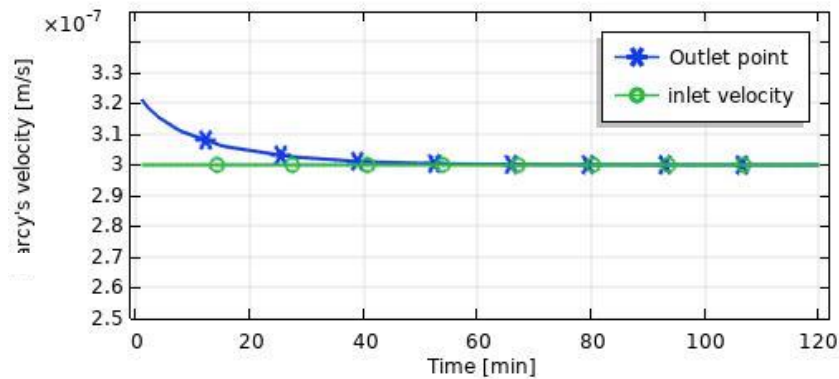
(a) FR0



(b) SR0



(c) PR0



(d) PR0

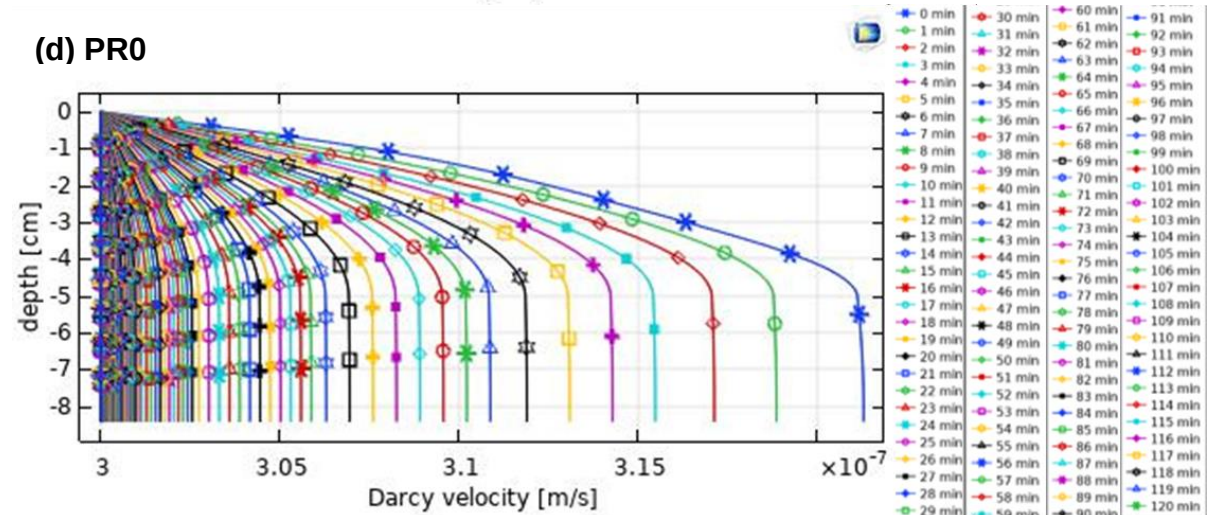
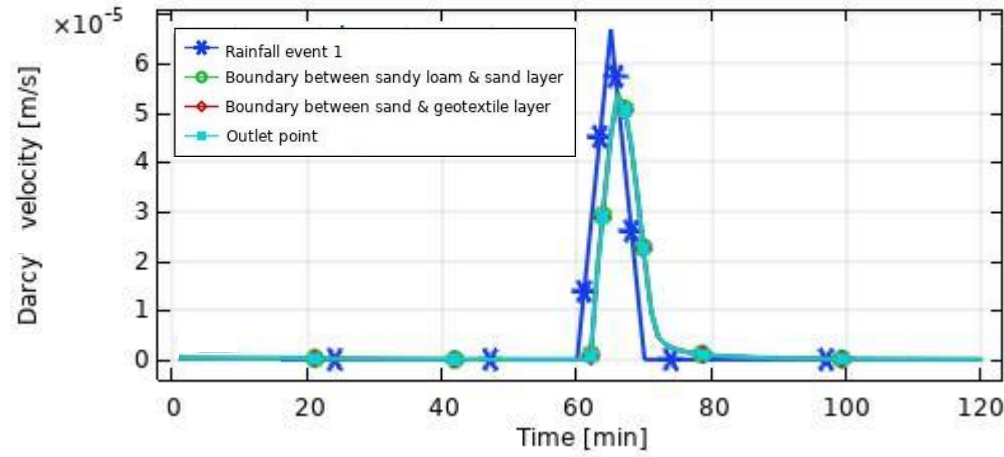
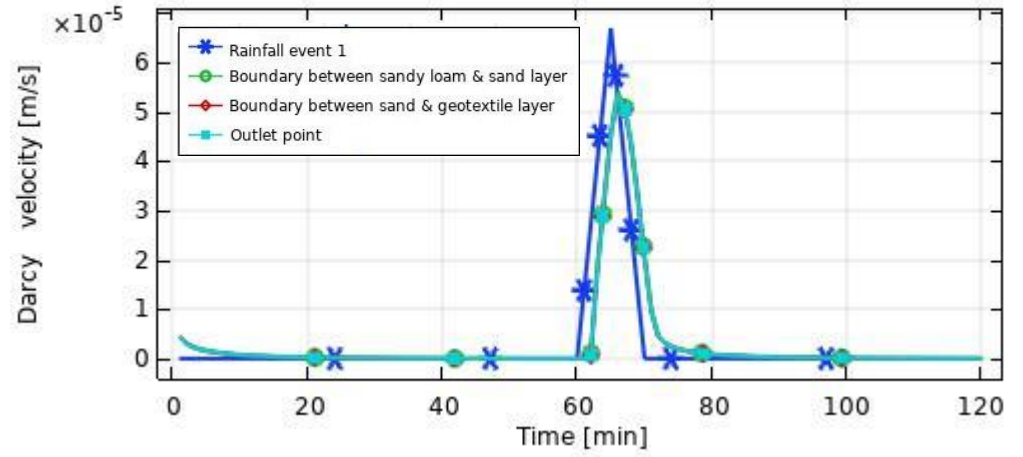


Figure 3-5 Model result of Darcy velocity with inlet rainfall intensity R0 with different initial soil pressure: (a) field capacity, (b) saturated, and (c) and (d) is the permanent wilting point. (a), (b) and (c) plotted Darcy velocity with time, and (d) shows the Darcy velocity changes with depth.

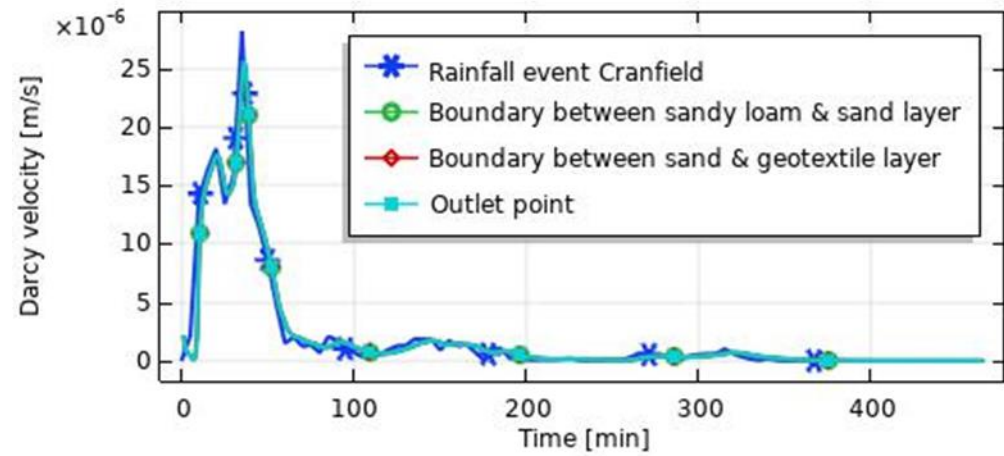
(a) FR1



(b) SR1



(c) FR2



(d) SR2

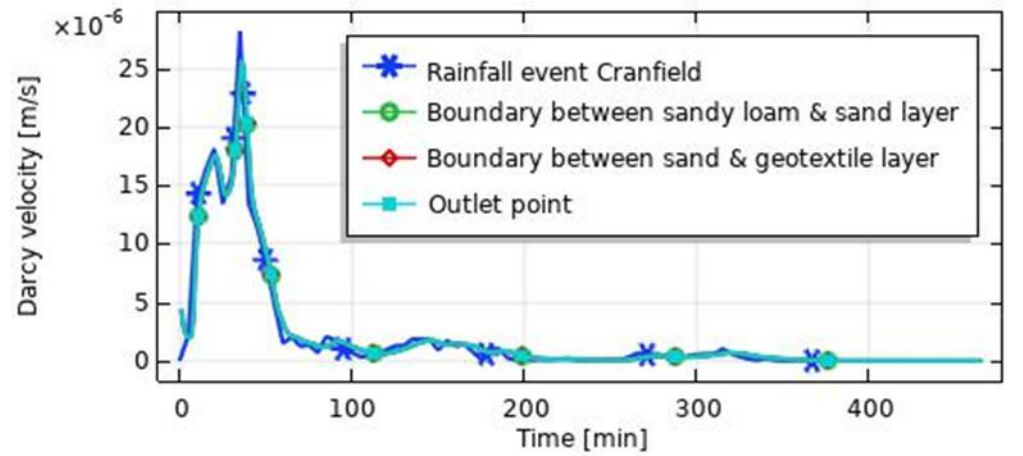


Figure 3-6 Model result of Darcy velocity with inlet rainfall events R1 and R2. The initial soil pressure for (a) and (c) is field capacity, (b) and (d) is saturated. (a) and (b) are results for Rainfall event 1, (c) and (d) are results for Rainfall event Cranfield.

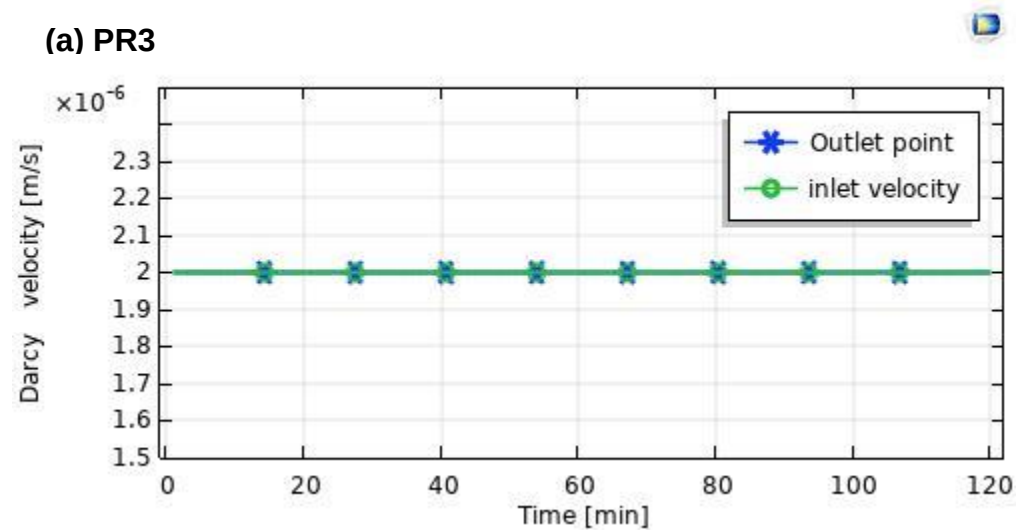
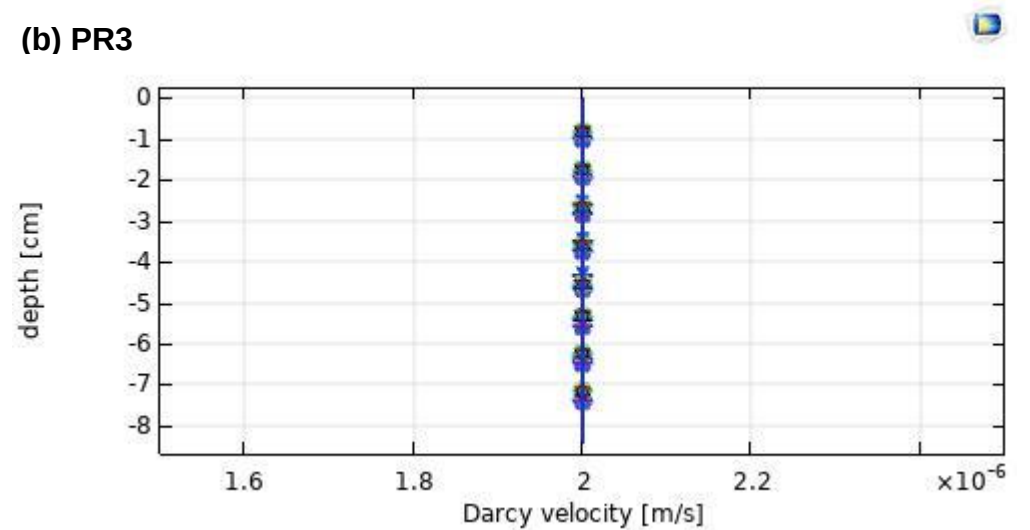


Figure 3-7 Model result of Darcy velocity with inlet rainfall intensity R3. The initial soil pressure for both (a) and (b) is at a permanent wilting point. (a) plotted Darcy velocity with time, and (b) shows the Darcy velocity changes with depth.

3.7 Discussion

3.7.1 Sensitivity analysis

The sensitivity analysis suggested that the Richards equation is sensitive to the changes in saturated hydraulic conductivity (K_s) and porosity (θ_s) (Figure 3-4 b, e). The changes caused by K_s are the most dramatic. These results agreed with previous research that K_s reduces much rapidly than θ_s (Philip, 1969).

On the other hand, the residual liquid volume fraction (θ_r) will not bring a significant effect to Darcy velocity unless the value of θ_r is too close to the porosity value (Figure 3-4 a).

The model is not sensitive to Van-Genuchten parameters α and n when the rainfall intensity stays at a high level (Figure 3-4 c, d). However, when a slight rainfall occurs, α and n will affect Darcy velocity value at the outlet point. It is also vital to notice that the Van-Genuchten parameters reflect the unsaturated condition of the soil; an unreasonable n value will make the calculation to fail.

3.7.2 Infiltration performance of the PGP system

Under an initially saturated condition, there is a low-level water discharge for Test SR0. Water held inside soil flows out from the column because of gravity in the first 20 minutes, and the water content becomes stable. For Test FR0, the water discharge phenomena are similar to Test SR0. However, less water is held inside soil pores, and the different porosity causes recharge between layers.

When the column is initially at field capacity (FR0), Darcy velocity inside soil starts with a slightly higher value and is then reduced to 0 before increasing again. This profile indicates that the soil absorbed part of the water to reach saturation. From 18 minutes to 40 minutes, the drainage rate of FR0 is lower than the infiltration rate, so that water can accumulate in the soil until it is saturated. Notably, the value of constant rainfall R0 is smaller than the saturated hydraulic conductivity of sandy loam, meaning it takes longer for the soil to reach saturation.

The initial water content of FR0 is lower than SR0; thus, FR0 needs more water to reach equilibrium. However, FR0 and SR0 reach equilibrium almost at the same time. It could be suggested that the drainage rate of SR0 is larger than FR0, or the infiltration rate of FR0 is greater than SR0. Although Shaw (2010) reported that the soil tends to have a lower infiltration rate when the soil is wetter, it may not be the case here as the PGP system is a multi-layer structure. The infiltration rate depends on the inlet, outlet and water storage in the whole structure.

During most of the operation time, the Darcy velocity at the outlet point equals the rainfall intensity, indicating an excellent permeability of the PGP system. Moreover, rainfall events R1 and R2 require a shorter time for the drainage Darcy velocity to reach the same level as the rainfall intensity. Rainfall events are not consistent and continuous as R0, and in actual weather conditions, the rainfall intensity also changes with location and seasons. Therefore, the differences caused by water storage in the first several minutes of a rainfall event are neglectable for a ten-year long-term simulation.

Different from expectations, the three curves (outlet point, the bottom of the sand layer, and the geotextile layer) are analogous and smooth. As previously presented in Section 2.2.3, geotextile shows that it can quickly convert between permeable (saturated) and impermeable (unsaturated). The geotextile properties might cause hydraulic performance to be less effective. However, the darcy velocity results indicate that the geotextile will not result in any significant water constraint. As concluded by Ho (2000) and Iryo and Rowe (2003), the high permeability of nonwoven geotextiles may be due to the continuous near-saturation of the surrounding sand layer. Also, without the drying process, the nonwoven geotextile is hardly dehydrated underground.

3.7.3 Hysteresis phenomenon for water infiltration in the PGP system

Depending on the pore size and the water travel path, the time for water to infiltrate through the whole soil column will be much longer than free water flow. The longer water flow path in soils caused a delay of peak velocity compared to inlet velocity and resulted in hysteresis.

According to previous research, filling and emptying the pores resulted in different wetting-drying curves due to the hysteretic effect. Defined by Philip (1969), the wetting curve is “the function appropriate to the analysis of absorption or infiltration into a homogeneous soil of uniform initial moisture content.”, and the drying curve describes “the proper and uniform soil moisture removal process.”. Typically, the wetting and drying process happens simultaneously for a drainage system like the PGP system (Philip, 1969). It is difficult to predict the hysteresis behaviour in a multi-layer structure accurately.

The hysteresis effects are more evident in the geotextile water retention curve. GWCC shows a robust hysteretic tendency, meaning nonwoven geotextile contained less water during the wetting path (compared to the drying path) at the same pressure head (Ho, 2000; Iryo and Rowe, 2003, 2004). These highly hysteretic curves indicated a low hydraulic conductivity when the geotextile is unsaturated (Stormont, Henry and Evans, 1997; Bouazza *et al.*, 2006). However, although theoretically, there is a massive difference between the wetting and drying curve for geotextile, only little hysteresis was observed (Siemens and Bathurst, 2010).

Fortunately, there are only several minutes differences caused by the capillary hysteresis for the PGP structure. For long-term simulation, the hysteresis behaviour can be neglected.

3.7.4 Darcy velocity changes with different initial water content

The initial soil water content is an indicator of the soil moisture situation before the simulation starts. For long-term simulations, the soil water content varies on the environmental conditions and the gaps between rainfall events. The

evapotranspiration effects (caused by solar radiation, evaporation and turfgrass transpiration) will result in a dehydrated soil surface. The soil water permeability during the dry period could result in considerable differences in long-term hydraulic simulations. However, the hydraulic model results have not observed the massive differences caused by the initial soil water content.

With the same rainfall intensity, the differences between initial field capacity (FR1 and FR2) and initially saturated (SR1 and SR2) can hardly be noticed with the same rainfall event. This phenomenon also denoted that the initial soil condition can be neglected during long-term simulation. The behaviour of the PGP system during the simulation was barely affected by the rainfall initial soil condition, and it mainly depended on the media properties and inlet rainfall intensity. The dry initial soil condition (PR0 and PR3) does not bring a significant difference for Darcy velocity, and it will not result in noticeable changes in long-term hydraulic performance.

This chapter tested the hydraulic model of a PGP system and evaluated its performance under different initial water content conditions. In general, the impact of the initial soil pressure head is too tiny to be considered for long-term hydraulic simulation. Water flows through the structure without dramatic restrictions. The PGP system expresses an excellent infiltration ability and significantly reduces urban surface runoff.

Typically, a different equation should be considered when simulation drying processes of porous media (especially geotextile). However, this model simulated a relatively thin porous media (8.43 cm), ignoring the differences between wetting and drying processes will not lead to significant errors. The drying equation should be taken into account when simulating a thick column while using similar methods.

The permanent wilting point means that the soil is under an extreme dry initial condition, and it is difficult for the model to find a consistent initial value. A tiny error will lead to a non-linear solution and cause the simulation to crash. The solution for continuing the simulation was to add a stationary study step in COMSOL. A stationary model step was added before the time-dependent

calculation started. The steady state of the stationary step helps to stabilise the calculated initial condition, minimizes errors, and allows the calculation to continue.

3.8 Conclusion

The hydraulic model has been tested under specific rainfall intensities (constant, invented short-term and actual rainfall events) and initial soil water content.

In general, as Darcy velocity results of the hydraulic model show a good consistency trend with the rainfall intensity, the PGP system can improve the permeability of the urban surface area. Based on the current results, Darcy velocity at the outlet point is slightly lower than the input rainfall intensity at a high value. However, water can still infiltrate and drain out quickly with a slight hysteresis phenomenon.

The sensitivity analysis results indicate that the hydraulic model is most sensitive to saturated hydraulic conductivity. Soil properties (including porosity, residual liquid volume fraction and hydraulic conductivity) are the most critical parameters affecting the Darcy velocity results, as the permeability of any medium is dependent on its properties.

Previous research indicated that geotextile in the PGP system would reduce the infiltration rate, but the model results from this chapter suggest otherwise. The nonwoven geotextile layer also shows an excellent infiltration ability.

Furthermore, the initial soil condition will not significantly change the Darcy velocity, and the soil will be saturated within an hour. Especially for long-term simulation, the changes caused by dry or wet soil conditions can be neglected.

The hydraulic model is the first part of the PGP-HRC model, and it also provides the driving force for water and contaminants. As an essential driving force of the clogging model, factors affecting Darcy velocity must be considered carefully for the PGP-HRC model. Especially those parameters that Darcy velocity is sensitive to, such as porosity and saturated hydraulic conductivity. The impacts of

geotextile water constraint and initial soil water content can be neglected for further long-term PGP-HRC model operation.

4 Chapter Four: Rainfall-infiltration boundary condition

4.1 Introduction

The infiltration rate is an essential indicator of the PGP system's performance. As clogging developed, the infiltration rate of the PGP system could be reduced until the system lost the infiltration ability. In COMSOL hydraulic model (Chapter 2 and Chapter 3), the infiltration rate is controlled by the soil properties, such as porosity and saturated hydraulic conductivity. In general understanding, the maximum infiltration rate is equal to the saturated hydraulic conductivity of filter media. Excess rainwater will become surface runoff, gather in the traditional pipeline system, or directly flow into natural water bodies. The PGP system is not designed for ponding water. Instead of ponding, rainfall exceeding the infiltration capacity of the PGP system will converge into a surface runoff in the urban environment.

Although the COMSOL Earth science module provides promising results and a user-friendly environment, some crucial hydrological boundary conditions have not been implemented (Chui and Freyberg, 2009). These limitations include the rainfall-infiltration-runoff relationship, and the relationship is essential when simulating the PGP system performance. Thus, this chapter will introduce the development of RIB as the second part of the PGP-HRC model.

To test the performance of the hydraulic model with RIB, another infiltration model (Green-Ampt) has been introduced and compared with the RIB results.

4.1.1 Limitation of the COMSOL hydraulic model

Ponding only happens when the rainfall intensity is higher than the infiltration capacity in natural urban surface areas. Moreover, most of the ponded water will become surface runoff.

The result of COMSOL hydraulic (Figure 4-1 a) shows a pressure head on the soil surface under a rainfall event (Figure 4-1 b). The rainfall event includes a 10 minutes rainfall period in which most of the intensity is higher than the soil saturated hydraulic conductivity. The results of the COMSOL hydraulic model

(Figure 4-1 a) show a surface pressure head higher than 0 cm when the rainfall intensity exceeds the saturated hydraulic conductivity. The pressure head results indicate that ponding water may occur when the pressure head higher than 0 cm from 60 [min] to 70 [min].

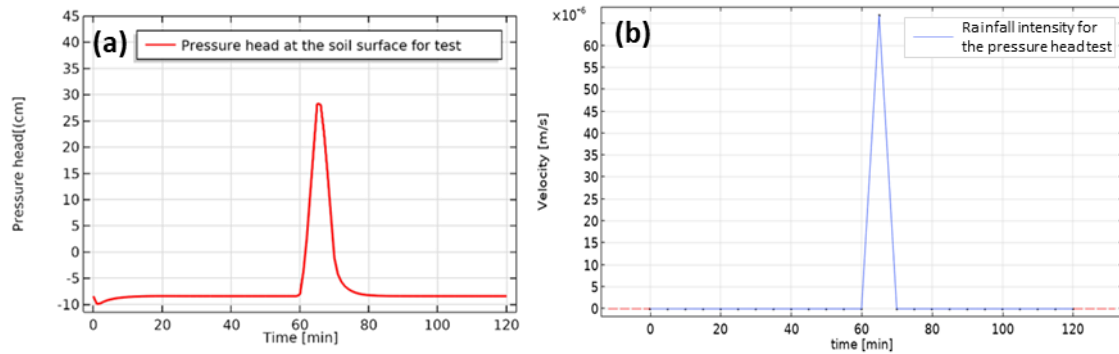


Figure 4-1 Pressure head on the soil surface of the COMSOL hydraulic model (a) and the rainfall event that has been used for the test (b).

Since the water flow is mainly driven by pressure gradient, a higher pressure on the soil surface will enormously impact the water movement inside the PGP column, which increases the Darcy velocity. A RIB condition was designed based on the soil suction potential changing with soil saturation; moreover, the rainfall intensity higher than K_{sat} will not be infiltrated into the soil layer.

4.1.2 Background to infiltration models

Over the past century, a large number of infiltration models have been developed (Becker, 2016). These infiltration models are based on a simplified concept and assumptions made under a specific environment (Becker, 2016). The infiltration models can generally be characterised as theoretical, semi-empirical, and empirical models (Mishra and Singh, 1999; Mishra, Tyagi and Singh, 2003; Becker, 2016). Mishra, Tyagi and Singh (2003) concluded the principles and examples for the three groups:

- *Theoretical models*: Theoretical models rely on the Darcy law and the law of conservation of mass, but the complexity of their derivation is different. The models considered fluid dynamics, dimensions, water retention relationship, and initial and boundary conditions.

Examples: Theoretical models include the Green-Ampt model (Heber Green and Ampt, 1911), Philip model (Philip, 1957, 1969), Mein and Larson model (Mein and Larson, 1971, 1973), Smith model (Smith, 1972), and Smith and Parlange model (Smith and Parlange, 1978).

- *Semi-empirical models:* Compared to the theoretical models, semi-empirical models apply a more straightforward continuity equation. The assumptions are superficial, depending on the infiltration rate-cumulative infiltration relationship or the flux-concentration relations.

Examples: Semi-empirical models include the Horton (1938) model, Holtan (1961) model, Overton (1964) model, Singh and Yu (1990) model and Grigorjev and Iritz (1991) model.

- *Empirical models:* The mathematical relations of the empirical models purely comply with data obtained from laboratory experiments or field measurements.

Examples: Empirical models include the SCS-CN (Mishra and Singh, 1999), Kostiakov model (Kostiakov, 1932), modified Kostiakov model (Smith, 1972), Huggins and Monke model (Huggins and Monke, 1966), and Collis-George model (Collis-George, 1977).

4.1.3 Selection of Green-Ampt model

Three of the most frequently mentioned infiltration models have been compared in Table 4-1: the Green-Ampt model, the Horton model, and the Philips model.

Therefore, some critical requirements have listed below for the selection:

1. Similar to the Richards equation used in the COMSOL hydraulic model, the selected model is better theoretical.
2. The infiltration rate or cumulative infiltration must be included in the model output data.
3. The input coefficients are accessible from the literature, and no experiments or field measurements are needed.
4. The soil characteristic parameter depends on the soil type instead of laboratory measurements.
5. The program or code is readily available.

6. The ponding-infiltration-surface runoff relationship on the surface boundary has been taken into account.

The Horton model is a semi-empirical model which requires several experimental data, and the experimental data are not accessible for this research. Meanwhile, the Philip model cannot provide stable results with different surface conditions (water ponding on the surface or continuous rainfall) (Whisler and Bouwer, 1970). Hence, the Green-Ampt model will be the adequate model that meets the requirements listed above.

Table 4-1 Summary of requirements and model operation conditions for the Green-Ampt, Horton, Philips and PE-GER model.

Model requirements		Green-Ampt	Horton	Philips
<i>Hydraulic process</i>	Infiltration	√	√	√
	Evapotranspiration	×	×	×
	Surface runoff/ Ponding	√	×	√
	water retention	√	×	×
<i>Codes</i>	Method clear for coding	√	√	√
	program available online	√	×	×
	MATLAB code available	√	√	√
<i>Model type</i>		Theoretical	Semi-empirical	Theoretical
<i>Input data</i>	Coefficient	Hydraulic conductivity; average suction at the wetting front; saturated water content; initial water content; soil surface water storage	The decay constant of infiltration rate; initial infiltration rate; final infiltration rate	Sorptivity; saturated hydraulic conductivity
	Precipitation	√	√	√
<i>Output data</i>	Infiltration rate	√	√	√
	Volume of infiltration	√	×	√
	Water soil content	√	×	×
<i>Others</i>	Layer	wetted zone/dry soil	-	Homogeneous soil
<i>Resources</i>		(Heber Green and Ampt, 1911; Mishra, Tyagi and Singh, 2003; Becker, 2016; Daliakopoulos, 2016)	(Philip, 1957, 1969; Mishra, Tyagi and Singh, 2003; Becker, 2016)	(Mishra, Tyagi and Singh, 2003; Becker, 2016)

4.1.4 Principles of the Green-Ampt model

Besides Richards equation (used in Chapter 2), other models simulated infiltration processes derived from different assumptions. As one of the most frequently mentioned, the Green-Ampt model (Heber Green and Ampt, 1911) assumed a boundary moving downward through a vertical soil column and divided the soil column into upper wetted and lower dry layers. Unlike the Richards equation, the Green-Ampt model considered the ponding, infiltration and surface runoff on the soil surface in each time interval.

The Green-Ampt model resembles COMSOL hydraulic model in many ways. They are both theory-based models for a one-dimensional soil column derived from Darcy's law. However, the assumptions for simplifying the processes are different from each other.

The Green-Ampt model was developed by Green and Ampt (1911) with an approximate physical theory and an exact solution (Chow, Maidment and Mays, 1988).

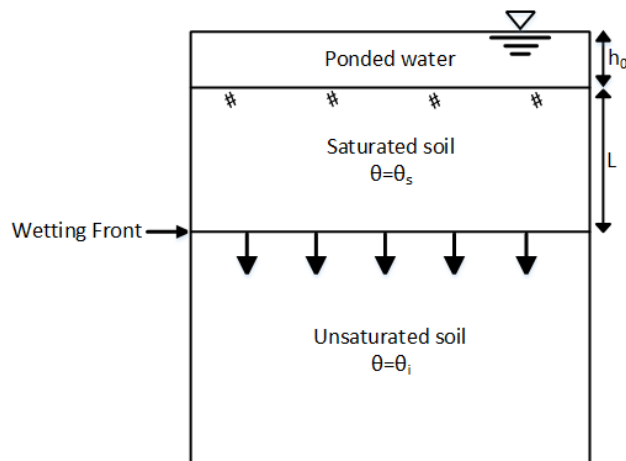


Figure 4-2 Water infiltrates into a unit cross-section of a soil column for Green-Ampt Model. (Source: Chow, Maidment and Mays (1988), Figure 4.3.2)

The simplified picture of the Green-Ampt model has shown in Figure 4-2. The soil column has been divided into two layers by wetting front with a sharp boundary (Chow, Maidment and Mays, 1988). The soil in the upper layer (wetted zone) is saturated with water content equal to total porosity (θ_s), compared to the soil

water content θ_i below the wetting front. Water is ponded on the surface at a small depth of h_0 . Since the infiltration begins, water infiltrates into the soil with a depth of L at time t .

Considering a unit cross-section area of vertical soil column (Figure 4-3), the wetted layer between the soil surface and depth L is defined as saturated soil. The initial soil moisture content throughout the whole soil column was θ_i . The soil moisture content will increase from θ_i to total porosity (θ_s) when the wetting front passed through (Chow, Maidment and Mays, 1988).

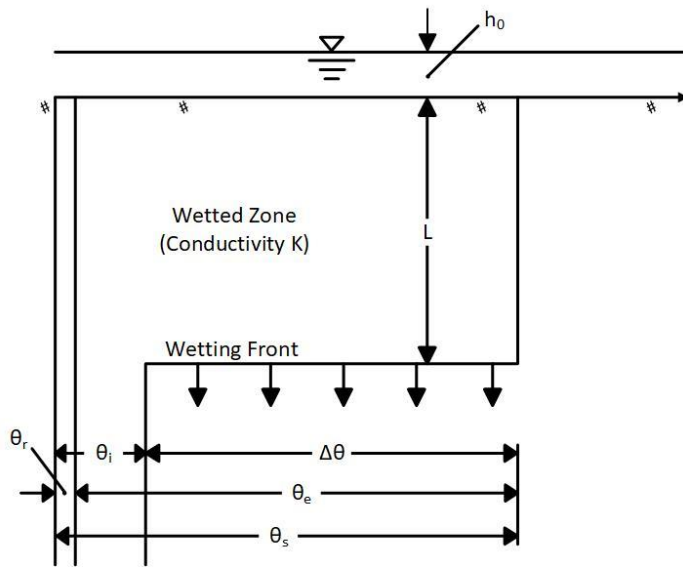


Figure 4-3 Simplified figure of Green-Ampt infiltration Model. The horizontal variable shows the soil moisture content, and the vertical variable indicates the depth of wetting front from the soil surface. (Source: Chow, Maidment and Mays (1988), Figure 4.3.1)

As a result of infiltration, the increase of water stored in the saturated soil (i.e. cumulative infiltration) for a unit cross-section area at a given time, t , can be calculated by equation (4-1).

$$F(t) = L \times (\theta_s - \theta_i) \quad (4-1)$$

Where $\Delta\theta = \theta_s - \theta_i$

$$F(t) = L\Delta\theta \quad (4-2)$$

The Green-Ampt model assumed that water ponded a little initially on the soil surface. However, water will only be ponded on the surface when the rainfall intensity is greater than the soil infiltration capacity.

The value of t_p is defined as ponding time when water starts to pond on the soil surface. The soil moisture content profile of the vertical soil column shows in Figure 4-4.

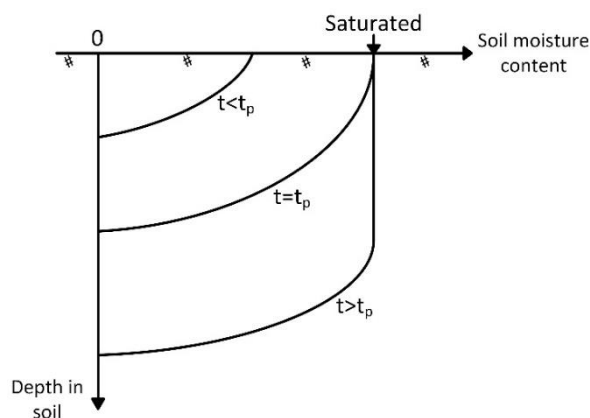


Figure 4-4 Soil moisture content profile of the vertical soil column. Rainfall starts with dry soil, and the figure indicates the changes in soil moisture content before ($t < t_p$), during ($t = t_p$) and after ($t > t_p$) the water ponded on the soil surface. (Source: Chow, Maidment and Mays (1988), Figure 4.4.1)

Assuming that soil is dry when rainfall starts, the soil moisture can be indicated in three different circumstances:

- Before ponding, $t < t_p$, the soil surface is unsaturated, rainfall intensity is smaller than the potential infiltration rates;
- At the time step that ponding happened, $t = t_p$, the soil surface is just saturated;
- If rainfall continues, $t > t_p$, the saturated soil moves downward, and surface runoff occurs.

4.1.5 Aim

This chapter presents a rainfall infiltration boundary condition (RIB) as a upper surface boundary to the COMSOL hydraulic model, to simulate the rainfall-runoff performance of the PGP system. The Green-Ampt model was set up to compare with the RIB model results to test the performance of the RIB.

4.1.6 Overview of approach

The Green-Ampt model has been set up in MATLAB, and the RIB has been applied to the COMSOL hydraulic model. Both models have been tested with different rainfall events and initial water content, and the results will be presented and compared in this chapter. The sensitivity of the Green-Ampt model to saturated hydraulic conductivity is also tested in this chapter.

4.1.7 Rainfall data applied for the Green-Ampt and RIB model tests

The rainfall intensity data have been obtained from the weather station at Cranfield University and the Swindon R25 weather station (Environment Agency). The information of both weather stations is presented in Table 4-1.

Table 4-2 Information about the weather station where rainfall data have been collected

<i>Station name</i>	Swindon R25	Cranfield weather station
<i>Station number</i>	249744TP	N/A
<i>Location</i>	Rodbourn, Swindon	Wolfson Field Laboratory, Cranfield, Bedford, UK
<i>Data format</i>	15 minutes total	5 minutes total
<i>Data start date</i>	12 th August 2014	1 st July 2017
<i>Data end date</i>	30 th November 2015	24 th April 2018
<i>Other</i>	-	Another heavy rainfall event collected on 7 th June 2016

Rainfall data was obtained from 12th August 2014 to 30th November 2015 from the Swindon weather station. The station measured cumulative rainfall every 15 minutes. Cranfield University weather station provided rainfall data from July 1st 2017 to April 24th 2018, and another heavy rainfall event on June 7th 2016

(maximum rainfall intensity in five minutes is 8.46 mm). All data are recorded as the amount of rainfall every five minutes.

More than 30 minutes of continuous rainfall with an interruption of no more than one hour (≤ 60 minutes) will be selected as one rainfall event. Based on the hydraulic model results from section 3.6, infiltration drops to 0 $[m\ s^{-1}]$ with no rainfall for 1 hour, and the soil becomes dry. Cranfield data have been divided into 113 rainfall events, and the rainfall data from the Swindon station can be divided into 170 rainfall events (283 rainfall events in total).

The data of rainfall events from the Cranfield and Swindon weather station are presented in Figure C-1 and Figure C-2 (Appendix C). As shown in Figure 4-5, most of the cumulative rainfall of each event from both stations are less than 10 mm. In general, rainfall events from the Swindon station have a long duration; however, the largest cumulative rainfall events among the collected data came from the Cranfield Weather Station.

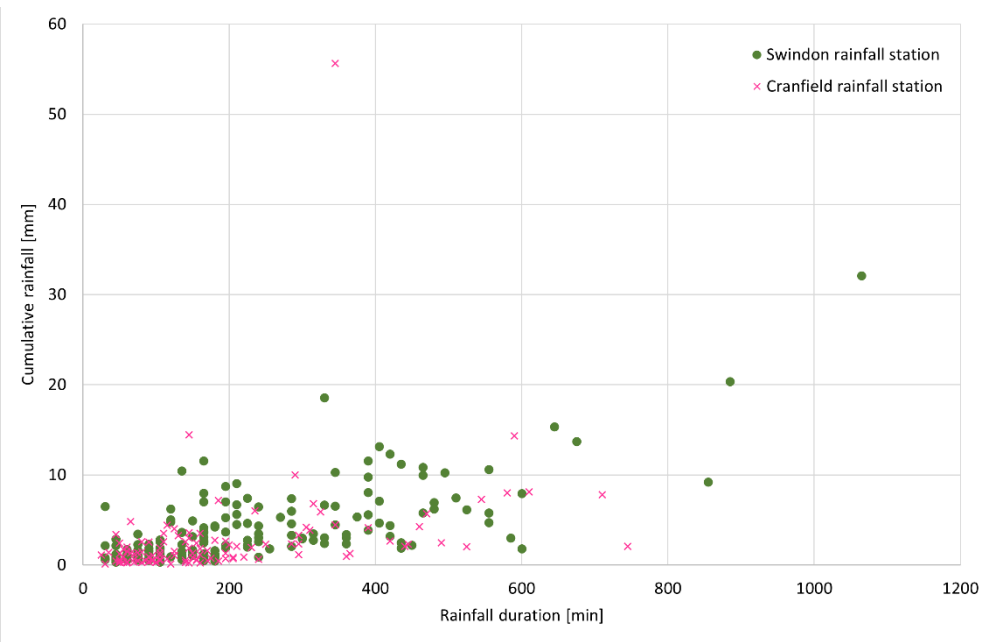


Figure 4-5 Rainfall duration plotted against the cumulative rainfall for each event of rainfall data from Swindon and Cranfield rainfall stations.

4.2 Mathematical implementation of the Green-Ampt model

Chow, Maidment and Mays (1988) thoroughly explained the equation derivation of the Green-Ampt model. Green-Ampt is based on Darcy's Law and the conservation of mass (Dingman, 2008). In saturated zone, Flux q of Darcy's Law can be expressed as

$$q = -K \frac{\partial h}{\partial z} \quad (4-3)$$

Under the circumstances of the soil column described in Section 4.1.4, the Darcy flux q is constant. f is defined as the Green-Ampt infiltration rate and is positive downward, and K is the hydraulic conductivity. Define points 1 and 2, point 1 located on the soil surface, and point 2 in the unsaturated soil right after the wetting front. Equation (4-3) can be approximated as

$$f = K \left(\frac{h_1 - h_2}{z_1 - z_2} \right) \quad (4-4)$$

The head of point 1 is equal to the depth of ponded water h_0 . Meanwhile, the head of point 2 equals $(-\psi_f - L)$ (ψ_f is the capillary pressure of the wetting front, L is the depth of the wetting front from the soil surface). The depth difference between point 1 and point 2 in the z -direction in the Green-Ampt soil column equals L . Equation (4-4) of Darcy's Law can be thus written as

$$f = K \left(\frac{h_0 - (-\psi_f - L)}{L} \right) \approx K \left(\frac{\psi_f + L}{L} \right) \quad (4-5)$$

Compared to $(\psi_f + L)$, the value of h_0 is so small that it can be neglected. It is usually assumed that ponded water becomes surface runoff and will not remain on the soil surface.

Based on Equation (4-2), the depth of the wetting front can be calculated by ($L = F/\Delta\theta$). Equation (4-5) can be substituted and written as

$$f = K \left(\frac{\psi_f \Delta\theta + F}{F} \right) \quad (4-6)$$

The infiltration rate f is equal to (dF/dt) , defined as the change of cumulative infiltration F in each time step. A differential equation can be used to express equation (4-6) with one unknown variable F

$$\frac{dF}{dt} = K \left(\frac{\psi_f \Delta \theta + F}{F} \right) \quad (4-7)$$

The Green-Ampt equation of cumulative infiltration can be obtained by solving equation (4-7), which is

$$F(t) - \psi_f \Delta \theta \ln \left(1 + \frac{F(t)}{\psi_f \Delta \theta} \right) = Kt \quad (4-8)$$

Chow, Maidment and Mays (1988) solved equation (4-8) by using the successive substitution method, it can be rearranged

$$F(t) = Kt + \psi_f \Delta \theta \ln \left(1 + \frac{F(t)}{\psi_f \Delta \theta} \right) \quad (4-9)$$

Successive substitution calculates the value of $F(t)$ at time step t using the value of $F(t - 1)$ which at the last time step $(t - 1)$, until cumulative infiltration F reaches a constant value.

The constant value of cumulative infiltration F can be substituted into equation (4-6) to calculate the corresponding potential infiltration rate f

$$f(t) = K \left(\frac{\psi_f \Delta \theta}{F(t)} + 1 \right) \quad (4-10)$$

When the ponded water cannot generate surface runoff, the depth of ponded water (h_0) is not negligible. In this case, the value of ψ_f should be replaced by $(\psi_f - h_0)$ in equations (4-8), (4-9) and (4-10).

4.2.1 Calculation of the ponding time

Mein and Larson (1973) presented a method appropriate for calculating the ponding time for the Green-Ampt infiltration model. Prior to water ponded on the soil surface, the cumulative infiltration F equals wt , which w is defined as the water input rate at time t . At the time step that ponding starts, the infiltration rate

can be set by $f_p = w$. Cumulate infiltration F_p when water ponded and the ponding time t_p can be obtained by solving equation (4-10) with substituted $F = wt$ and $f_p = w$.

$$F_p = \frac{K\psi_f\Delta\theta}{w - K} \quad (4-11)$$

$$t_p = \frac{F_p}{w} = \frac{K\psi_f\Delta\theta}{w - (w - K)} \quad (4-12)$$

Infiltration rate changes infiltration at a specific time step and is limited to the soil infiltration capacity. The infiltration can be calculated by solving equations (4-6) and (4-7):

$$f(t) = f_c(t) = \frac{dF}{dt} = K \left(1 + \frac{\psi_f\Delta\theta}{F} \right) \quad (4-13)$$

Under the ponded condition, variables have to be separated and integrated into equation (4-13). The cumulative infiltration F_p at the ponding starting time t_p to any cumulative infiltration (F) after ponded at time t .

$$t - t_p = \frac{F - F_p}{K} + \frac{\psi_f\Delta\theta}{K} \ln \left(\frac{F_p + \psi_f\Delta\theta}{F + \psi_f\Delta\theta} \right) \quad (4-14)$$

The depth of cumulative infiltration after ponded can be calculated by equation (4-14), infiltration rate after ponded can be calculated by equation (4-13).

4.2.2 Coupling relationship among parameters

Input parameters required for the Green-Ampt model including saturated hydraulic conductivity K , porosity θ_s , and the wetting front suction head ψ_f . It has been concluded by Brooks and Corey (1964) that the wetting front suction head ψ_f have a logarithmic relationship with the effective saturation S_e (Brooks-Corey equation).

$$S_e = \left[\frac{\psi_b}{\psi_f} \right]^r \quad (4-15)$$

Where ψ_b and r are constants that can be obtained by the fitting curve of S_e and ψ_f from experimental data.

Meanwhile, the effective saturation S_e can be expressed by an equation of total porosity θ_s , effective porosity θ_e , soil moisture θ at time step t , and residual moisture of soil after fully drained θ_r .

$$S_e = \frac{\theta - \theta_r}{\theta_s - \theta_r} \quad (0 \leq S_e \leq 1, \theta_r \leq \theta \leq \theta_s) \quad (4-16)$$

Where $(\theta - \theta_r)$ is the available soil moisture content. $(\theta_s - \theta_r)$ equals to effective porosity θ_e , also known as maximum available soil moisture content.

For the initial condition in Green-Ampt Model, $\theta = \theta_i$, and substituting θ_i into equation (4-16).

$$\theta_i = S_e \theta_e + \theta_r \quad (4-17)$$

The change in soil moisture content can be rewritten as

$$\Delta\theta = \theta_s - \theta_i = (1 - S_e)\theta_e \quad (4-18)$$

Average values of Green-Ampt Model parameters $(\theta_s, \theta_e, \psi_f, K)$ for different soil classes have been calculated by analysing 5000 soil samples across the United States using the Brooks-Corey equation. (Rawls, Brakensiek and Miller, 1983) The average value has shown in Table 4-3.

Table 4-3 Green-Ampt parameters for different soil texture classes (Source: Rawls, Brakensiek and Miller (1983), Table 2)

<i>Soil texture class</i>	Total porosity θ_s [cm³ cm⁻³]	Effective porosity θ_e [cm³ cm⁻³]	Wetting front suction head (capillary pressure) ψ_f [cm]	Hydraulic conductivity K [cm h⁻¹]
<i>Sand</i>	0.437	0.417	4.95	11.78
<i>Loamy sand</i>	0.437	0.401	6.13	2.99
<i>Sandy Loam</i>	0.453	0.412	11.01	1.09
<i>Loam</i>	0.463	0.434	8.89	0.34
<i>Silt loam</i>	0.501	0.486	16.68	0.65
<i>Sandy clay loam</i>	0.398	0.330	21.85	0.15
<i>Clay loam</i>	0.464	0.309	20.88	0.10
<i>Silty clay loam</i>	0.471	0.432	27.30	0.10
<i>Sandy clay</i>	0.430	0.321	23.90	0.06
<i>Silty clay</i>	0.479	0.423	29.22	0.05
<i>Clay</i>	0.475	0.385	31.63	0.03

4.2.3 Initial soil water content

Instead of experimental field measurement, infiltration has been estimated using default soil physical parameters. Consider simulating the impact of soil moisture content reduction because of the turf-grass transpiration and soil evaporation on the soil surface, the initial water content can be set in three different conditions:

- θ_{sat} , the soil is saturated.
- θ_{fc} , the moisture content at field capacity.
- θ_{pwp} , the moisture content at permanent wilting point.

The soil moisture content values for different soil texture classes under three soil moisture conditions (Table 4-4) are obtained from Hess, Harrison and Counsell (2000).

Table 4-4 Default initial soil moisture content for Green-Ampt Model, under three conditions. (Source: Hess, Harrison and Counsell (2000), Table 2)

<i>Soil texture class</i>	θ_{sat}	θ_{fc}	θ_{pwp}
<i>Sand</i>	0.437	0.115	0.033
<i>Loamy sand</i>	0.437	0.168	0.055
<i>Sandy loam</i>	0.453	0.245	0.095
<i>Loam</i>	0.463	0.279	0.117
<i>Silt loam</i>	0.501	0.324	0.133
<i>Sandy clay loam</i>	0.398	0.241	0.148
<i>Clay loam</i>	0.464	0.321	0.197
<i>Silty clay loam</i>	0.471	0.350	0.208
<i>Sandy clay</i>	0.430	0.311	0.239
<i>Silty clay</i>	0.479	0.371	0.250
<i>Clay</i>	0.475	0.368	0.272

4.2.4 Model summary

The model flow chart has been summarised in Figure 4-6. The flow chart has explained the equation relationship and the feedback loop of the Green-Ampt model. The model has been set up using MATLAB, and in each time step, the cumulative infiltration and rainfall.

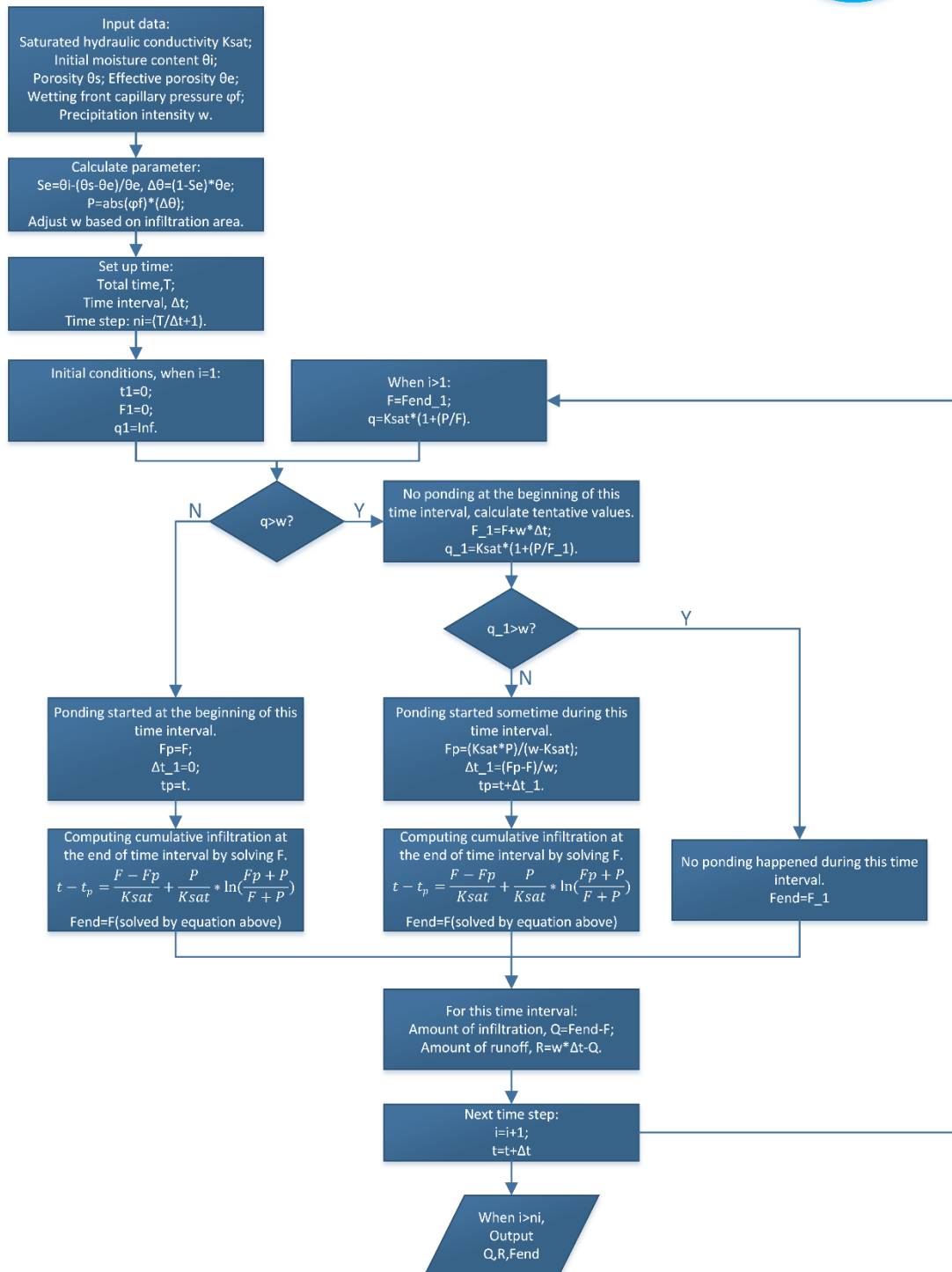


Figure 4-6 Model flow chart of the Green-Ampt model

4.2.5 Green-Ampt model parameterisation and test design for comparison

It is crucial to note that the Green-Ampt results are only presented here to test the RIB's performance. Unlike the PGP-HRC model, the surface material used for this chapter is loamy sand. Therefore, to avoid confusion with the PGP-HRC model, this chapter has used a different soil type (loamy sand) for Green-Ampt and RIB models. Consider the turf-grass layer, soil type of loam, sandy loam, and loamy sand will be the best choice for a long-term-use lawn with low maintenance (*Soil Preparation for a Beautiful Lawn*, 2018). The input parameters, rainfall and initial soil moisture (water) content used for the Green-Ampt model are shown in Table 4-5 and Table 4-6.

Table 4-5 Input parameters of loamy sand used in the Green-Ampt model

Parameter	Symbol	Value	Unit	Source
<i>Saturated hydraulic conductivity</i>	K_s	2.99	[cm h ⁻¹]	(Rawls, Brakensiek and Miller, 1983)
<i>Wetting front capillary pressure</i>	ψ_f	6.13	[cm]	
<i>Porosity</i>	θ_s	0.437	[-]	
<i>Effective liquid volume content</i>	θ_e	0.401	[-]	

Table 4-6 Green-Ampt model tests design and their input parameters (Source: Hess, Harrison and Counsell, 2000)

Rainfall	Initial moisture content [-]	Timestep [min]
113 rainfall events from the Cranfield weather station (Figure C-1) Unit: [mm] per 5 minutes	0.437 Saturated (High)	5
	0.168 Field capacity (Medium)	
	0.055 Permanent wilting point (Low)	
170 rainfall events from the Swindon weather station (Figure C-2) Unit: [mm] per 15 minutes	0.437 Saturated (High)	15
	0.168 Field capacity (Medium)	
	0.055 Permanent wilting point (Low)	

4.3 Sensitivity analysis of Green-Ampt model

4.3.1 Hydraulic conductivity (K_s) sensitivity analysis

Assuming that all other parameters stay constant, based on the basic Green-Ampt equation (Equation (4-3)), the main reason for the decrease of infiltration rate is hydraulic conductivity.. The decrease in infiltration rate can be estimated to be equal to the reduction rate of hydraulic conductivity. Some conditions, such as soil compaction, may cause an extreme reduction of hydraulic conductivity.

4.3.2 Rainfall characteristic for K_s sensitivity analysis

The Green-Ampt model calculation is a time dependent simulation. On the soil surface, the ponding possibility is affected by the rainfall intensity in each time step. Hence, besides the cumulative infiltration, rainfall intensity characteristic can also be an indicator for testing the sensitivity for K_s .

4.3.3 Sensitivity analysis parameterisation

Based on the previous research above, the hydraulic conductivity for the sensitivity analysis has been set into five different stages (Table 4-7). The initial soil water content at field capacity has been chosen for this analysis. 283 rainfall events from both the Cranfield and Swindon weather station have been applied to the Green-Ampt model sensitivity analysis. The reduced K_s values are shown in Table 47, and all 283 rainfall events were calculated with the K_s listed.

Table 4-7 Hydraulic conductivity sensitivity analysis. The hydraulic conductivity number (K_s) after reduced have been shown in this table.

Tests	Percentage of hydraulic conductivity compared to original	K_s [mm h ⁻¹]
1	100% hydraulic conductivity	29.900
2	92% hydraulic conductivity	27.500
3	75% hydraulic conductivity	22.425
4	50% hydraulic conductivity	14.950
5	30% hydraulic conductivity	8.970

4.3.4 Green-Ampt model sensitivity analysis results and discussion

Figure 4-7 (a) and (b) describes the results of the saturated hydraulic conductivity sensitivity analysis for the Green-Ampt model. The figure shows the cumulative infiltration [mm] against the total rainfall [mm]. Figure 4-7 (c) shows the cumulative infiltration results with different K_s changing with time using a single rainfall event (Cranfield 113).

The intensity in each time step of rainfall events from the Cranfield and Swindon weather station have been analysed. The rainfall characteristic analysis results have illustrated in Table 4-8. Notably, the rainfall intensity units of the Swindon station (mm/ 15 minutes) are different from the ones of the Cranfield weather station (mm/ 5 minutes). The differences are caused by the measuring resolution of the weather station.

For Cranfield station rainfall data (Figure 4-7(a)), the cumulative infiltration starts to decrease when the total rainfall is higher than 14 mm. The higher the total rainfall, the lower the hydraulic conductivity, which will result in less cumulative infiltration. The results indicate that the Green-Ampt model is sensitive to saturated hydraulic conductivity, a lower K_s will reduce the infiltration capacity of the soil.

However, although the total rainfall of several Swindon station rainfall events is higher than 14 mm, hydraulic conductivity reduction does not significantly affect the cumulative infiltration for Swindon station rainfall data (Figure 4-7(b)).

To understand the differences in the sensitivity analysis results between Cranfield and Swindon stations, a rainfall characteristic has been analysed and shown in Table 4-8. For Swindon station rainfall data, only 0.57% of the rainfall intensities are higher than the $30\%K_s$, and 0% higher than the $50\%K_s$. In contrast, 0.25% of the rainfall intensities are larger than the original K_s value. With the same total rainfall, the Swindon station rainfall events tend to last a long time with small intensity, and Cranfield station rainfall events are short-term with large intensity.

Also, a much larger maximum rainfall intensity value can be noticed for the Cranfield station rainfall data. The maximum intensity value of Swindon station

rainfall data is 3.42 [mm/ 15 min], which is only 1.177 [mm/ 15 min] bigger than the $30\%K_s$. The changes caused by the exceeded rainfall intensity (reduction in cumulative infiltration) would be difficult to notice in Figure 4-7(b).

It can be noticed in Figure 4-7 (c), in each time step, the cumulative infiltration was calculated. At the same time, a smaller hydraulic conductivity resulted in a lower cumulative infiltration. The Green-Ampt model calculates the ponding in each time step based on the rainfall intensity and saturated hydraulic conductivity. Therefore, the Green-Ampt model is sensitive to the relations between K_s and rainfall intensity. This result coincides with the concepts of the RIB.

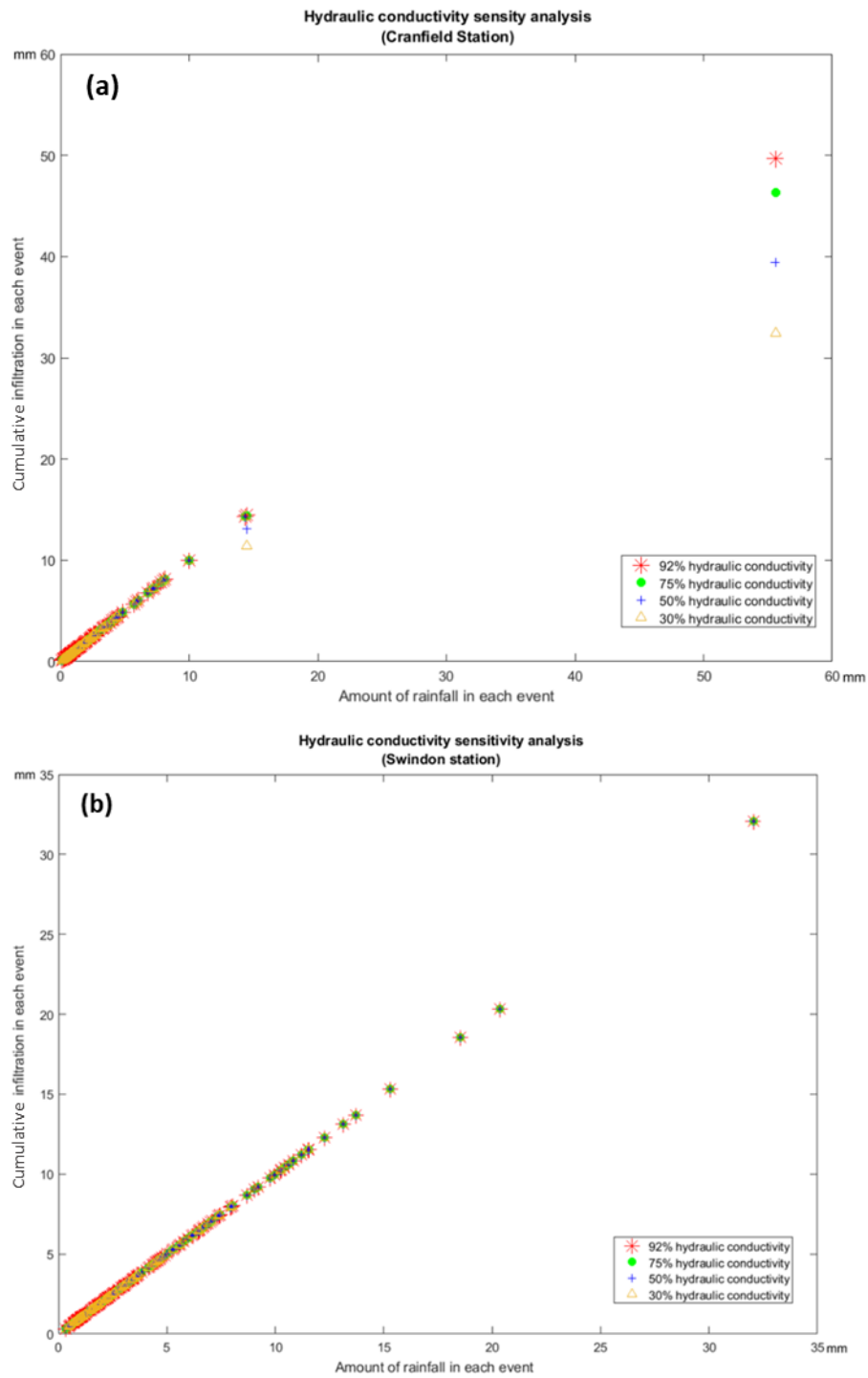


Figure 4-7 The Green-Ampt model sensitivity analysis results with different hydraulic conductivity. 113 rainfall event data from Cranfield weather station (a) and 170 rainfall event data from Swindon weather station (b). The relationship was plotted between the amount of rainfall and cumulative infiltration for each rainfall event.

(continued)

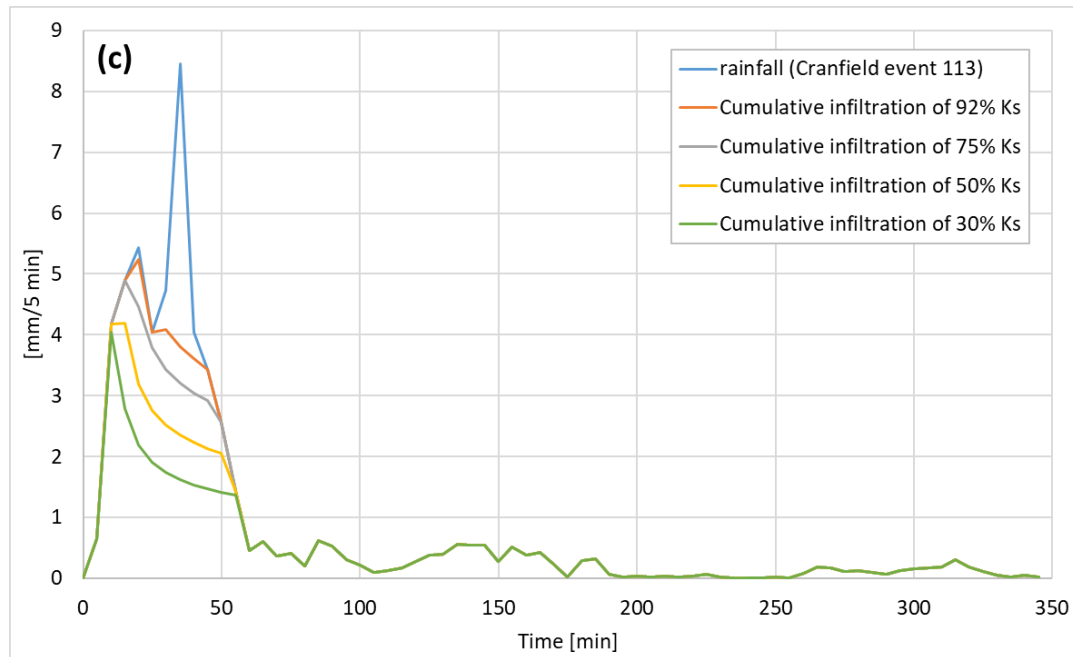


Figure 4-7 The Green-Ampt model sensitivity analysis results with different hydraulic conductivity. 113 rainfall event data from Cranfield weather station (a) and 170 rainfall event data from Swindon weather station (b). The relationship was plotted between the amount of rainfall and cumulative infiltration for each rainfall event. (c) shows the single rainfall event Cranfield 113 and the cumulative infiltration with different Ks (using single rainfall event Cranfield 113) changing with time series.

Table 4-8 Rainfall characteristic analysis of the rainfall event data from the Cranfield and Swindon weather station

Rainfall event from the Swindon weather station					
<i>Total number of 15 minutes rainfall data</i>	<i>Max value of 15 minutes rainfall data</i>	<i>% Ks</i>	<i>Ks [mm 15min⁻¹]</i>	<i>Number of 15 minutes rainfall data higher than Ks</i>	<i>Percentage of 15 minutes rainfall data in each time step higher than Ks</i>
2813	3.42 [mm 15min ⁻¹]	30%	2.243	16	0.57%
		50%	3.738	0	0.00%
		75%	5.606	0	0.00%
		92%	6.877	0	0.00%
		100%	7.475	0	0.00%
Rainfall event from the Cranfield weather station					
<i>Total number of 5 minutes rainfall data</i>	<i>Max value of 5 minutes rainfall data</i>	<i>% Ks</i>	<i>Ks [mm 5min⁻¹]</i>	<i>Number of 5 minutes rainfall data higher than Ks</i>	<i>Percentage of 5 minutes rainfall data in each time step higher than Ks</i>
4480	8.46 [mm 5min ⁻¹]	30%	0.748	40	0.89%
		50%	1.246	22	0.49%
		75%	1.869	13	0.29%
		92%	2.292	11	0.25%
		100%	2.492	11	0.25%

4.4 RIB model

4.4.1 Mathematical implementation of the RIB boundary

RIB is an extra rainfall-infiltration boundary condition that could be set up on the soil surface to simulate the surface runoff generation process. On the soil surface, the real-time inlet velocity (u_{in}) has been decided based on the relationship between saturated hydraulic conductivity (K_{sat}), rainfall intensity (v_{rain}), real-time liquid volume fraction (θ) and porosity (θ_s) as below.

$$\text{When } \theta \geq \theta_s, u_{in} = 0 \quad (4-19)$$

$$\text{When } \theta < \theta_s, u_{in} = \begin{cases} \text{if } v_{rain} < K_{sat}, v_{rain} \\ \text{if } v_{rain} \geq K_{sat}, K_{sat} \end{cases} \quad (4-20)$$

4.4.2 Tests design

The initial water content and model test design of the hydraulic model with RIB tests design are shown in Table 4-9. Nine tests have been designed with different initial pressure head and inlet rainfall events to test the performance of the RIB model. The results were recorded every 1 minute.

The rainfall intensity has been set for five different patterns:

R1: Rainfall event 1. A rainfall event lasts for 10 minutes, starting at 60 minutes, and reaches the largest value at 240.48 [mm h⁻¹], and finish at 70 minutes. Rainfall event 1 is a created extreme event.

R4: Constant rainfall intensity at $\frac{1}{2} K_{s(loamy\ sand)}$, 14.958 [mm h⁻¹].

R5: Constant rainfall intensity at $2 K_{s(loamy\ sand)}$, 59.832 [mm h⁻¹].

R10E: Ten of the selected rainfall events from the Cranfield weather station.

R113: Rainfall event Cranfield 113 (Figure A-2, Appendix A). An actual rainfall event was measured from Cranfield weather station on June 7th 2016, which was the heaviest rainfall event in the collected rainfall data.

The rainfall intensity R1 is an invented short-term rainfall event, and the data has shown in Figure A-1 (Appendix A). R4 and R5 are designed to test the RIB, and they are both continuous rainfall events that last for 30 minutes.

Ten rainfall events (R10E) from the Cranfield weather station have been selected to represent the rainfall characteristic and compare with the Green-Ampt model results. The selected rainfall events include Cranfield 8, 25, 45, 50, 53, 57, 90, 97, 110 and 113. These rainfall events consist of the six heaviest rainfall events, and four events covered the range of rainfall amount 0.5 mm to 7.15 mm with average interval. The data of events have indicated in Figure C-3 (Appendix C).

The letter R and N of the test name in front of the rainfall number represents the upper boundary condition with RIB (R) or normal inlet velocity condition (N). The extra letter S of the test name of SRR113 indicates the saturated initial soil water content that is different from other tests.

The rainfall duration for RR1 and NR1 is 120 minutes, and RR4, NR4, RR5 and NR5 last for 30 minutes. SRR113 is a 345 minutes rainfall event. The model operation time for tests RR10E and NR10E is the same as the rainfall event duration.

Previous COMSOL hydraulic model results demonstrated that the water flow inside the porous media would not be significantly affected by initial soil water content. Thus, the initial soil pressure head has been fixed field capacity (-8.43 cm), except for test SRR113. With initially saturated soil water content, the SRR113 test is an additional design to compare with the results of the Green-Ampt model.

Table 4-9 RIB tests design and their input parameters

Rainfall	No.	RIB	Initial soil pressure head [cm]	Time [min]
Rainfall event 1	RR1	√	-8.43 (field capacity)	120
	NR1			
$\frac{1}{2}K_{s(loamy\ sand)}$	RR4	√		30
	NR4			
$2K_{s(loamy\ sand)}$	RR5	√		
	NR5			
10 selected rainfall events (R10E)	RR10E	√		Same as the rainfall event
	NR10E			
Cranfield 113	SRR113	√	-0.01 (saturated)	345

4.4.3 Model parameterisation

The RIB was applied to the three layers COMSOL hydraulic model presented in Chapter 2 and Chapter 3. The RIB model results are only presented here to test its performance; thus, with the purpose of control variables for comparison, the top layer of the COMSOL hydraulic model has been replaced by loamy sand parameters of the Green-Ampt model (Section 4.2.5).

Since the RIB is a surface boundary condition, the parameters used for RIB calculation were the same as the loamy sand layer. The real-time liquid volume fraction is a calculated variable that changes as the simulation continues. The RIB will be re-calculated in each time step.

Table 4-10 Input parameters for loamy sand, sand and clean non-woven geotextile for RIB tests

Parameter	Symbol	Loamy sand and RIB calculation	Sand	Geotextile	Unit
The saturated liquid volume fraction (porosity)	θ_s	0.437	0.375	0.86	-
The residual liquid volume fraction	θ_r	0.036 ($\theta_r = \theta_s - \theta_e$)	0.053	-	-
Saturated hydraulic conductivity	K_s	8.31×10^{-6}	7.47×10^{-5}	1.64×10^{-3}	m s ⁻¹
Van-Genuchten parameter	α	3.74	3.55	-	m ⁻¹
Van-Genuchten parameter	n	1.7466	3.16	-	-
Mass per unit area	M.A.	-	-	468	g m ⁻²
Apparent opening size	AOS	-	-	0.15	mm
Thickness	a	50	30	4.30	mm
Permittivity	κ	-	-	0.382	s ⁻¹
Source	-	(Rawls, Brakensiek and Miller, 1983) Van-Genuchten parameter (Schaap, Leij and van Genuchten, 2001)	(Schaap and Leij, 1998)	(Ho, 2000)	-

4.4.4 Upper and lower boundary condition

The lower boundary is a free flow boundary condition, as described in Chapter 5. The RIB has been applied to the upper boundary on the soil surface.

4.4.5 RIB model results and discussion

The COMSOL hydraulic model results with and without RIB have been compared in Figure 4-8, Figure 4-9 and Figure 4-10 (a). Figure 4-8 shows the pressure head results on the soil surface; Figure 4-9 demonstrates the differences in the Darcy

velocity caused by RIB. Figure 4-10 concludes the cumulative infiltration results of ten selected rainfall events (RR10E).

4.4.5.1 Pressure head

As indicated in Section 4.1.2, a positive pressure head on the soil surface can be observed for tests NR1 and NR5 (Figure 4-8 (a) and (c)).

After RIB is applied for RR1 and NR1, the pressure head maintains at -1 cm or less. The differences are more noticeable when the rainfall intensity is higher than K_s . Since the rainfall intensity stays at a low level (smaller than K_s), the pressure head results for NR4 and RR4 are indistinguishable.

RIB application prevents ponding on the soil surface and keeps the pressure head constant at a lower value. It is roughly consistent with the reality of surface runoff converging after heavy precipitation in urban areas.

4.4.5.2 Darcy velocity

The Darcy velocity results of the COMSOL hydraulic model with and without RIB are shown in Figure 4-9. Cranfield 113 and Cranfield 50 are actual rainfall events recorded by the Cranfield weather station. These rainfall have been chosen to represent the rainfall event with intensity higher (Cranfield 113) and lower (Cranfield 50) than K_s .

For Cranfield 50, the Darcy velocity results with (RR10E) or without (NR10E) RIB are unidentical. The Darcy velocity at the outlet point follows the rainfall intensity trend, and mostly all the rainwater can be infiltrated and drained from the structure.

For R1 and Cranfield 113, when rainfall intensity is higher than K_s , rainwater cannot infiltrate into the structure, and the Darcy velocity at the outlet point is much smaller than rainfall intensity. The total amount of rainfall intensity exceeds K_s can be considered as the volume of surface runoff.

Saturated hydraulic conductivity (K_s) denoted a quantitative measurement of the capability of saturated soil to transport water. It can be predicted as how much water is allowed to flow through pores of saturated soil. Under a high rainfall

intensity, the surface soil is nearly or fully saturated. Therefore, only the intensities equal to or lower than K_s are allowed to transmit through the soil.

In general, the application of RIB can indeed allow the COMSOL hydraulic model results closer to the actual rainfall-ponding-infiltration situation of the urban areas.

4.5 Results of comparison of RIB and Green-Ampt model

Figure 4-10 compared the cumulative infiltration results of the COMSOL hydraulic model, RIB and the Green-Ampt model. The NR10E and RR10E results are shown in Figure 4-10 (a), and the results of the Green-Ampt model with Cranfield station rainfall events are shown in Figure 4-10 (b). Figure 4-10 (b) also contains the results of test SRR113 to compare the changes caused by the initial soil water content.

Figure 4-10 (a) plots the cumulative infiltration results of the COMSOL hydraulic model in the same graph as the Green-Ampt model. Interestingly, COMSOL model results for both NR10E and RR10E are not strictly landed on the 1:1 line as it is of the Green-Ampt model. The cumulative infiltration results of the COMSOL model with initial field capacity soil moisture content is slightly higher than the total rainfall. The discrepancy between these two models could be caused by the calculation method and geometry.

The hydraulic model results with RIB at saturated and field capacity initial soil water content are compared with the Green-Ampt results of Cranfield rainfall events (Figure 4-10 b). Notably, the initial soil water content does not dramatically change the cumulative infiltration results of the hydraulic model, and the cumulative infiltration value is similar to the Green-Ampt result with saturated initial soil water content.

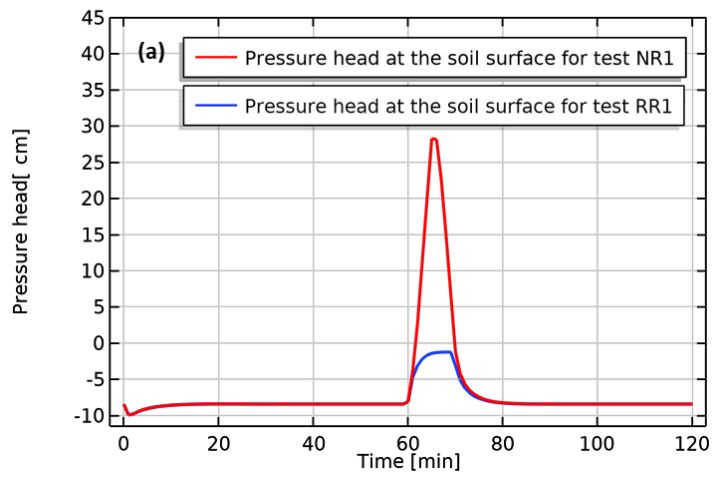
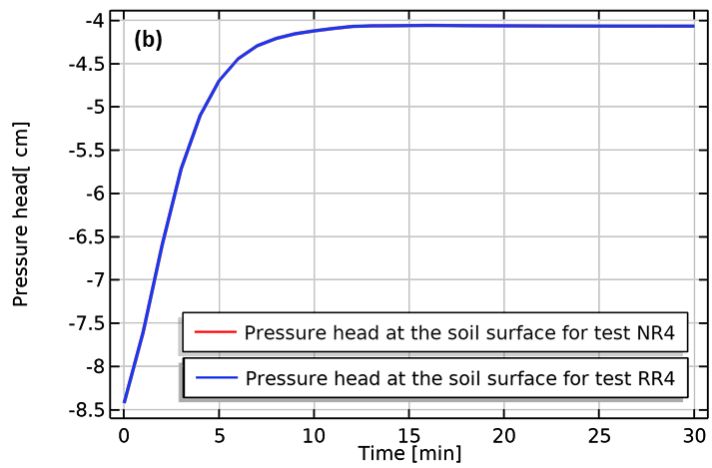
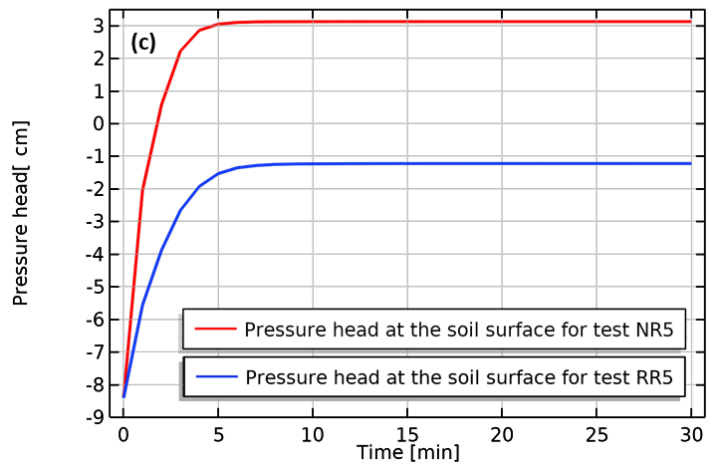


Figure 4-8 Pressure head results of COMSOL hydraulic model of NR1, RR1, NR4, RR4, NR5 and RR5. The figure plotted the surface pressure head with time.

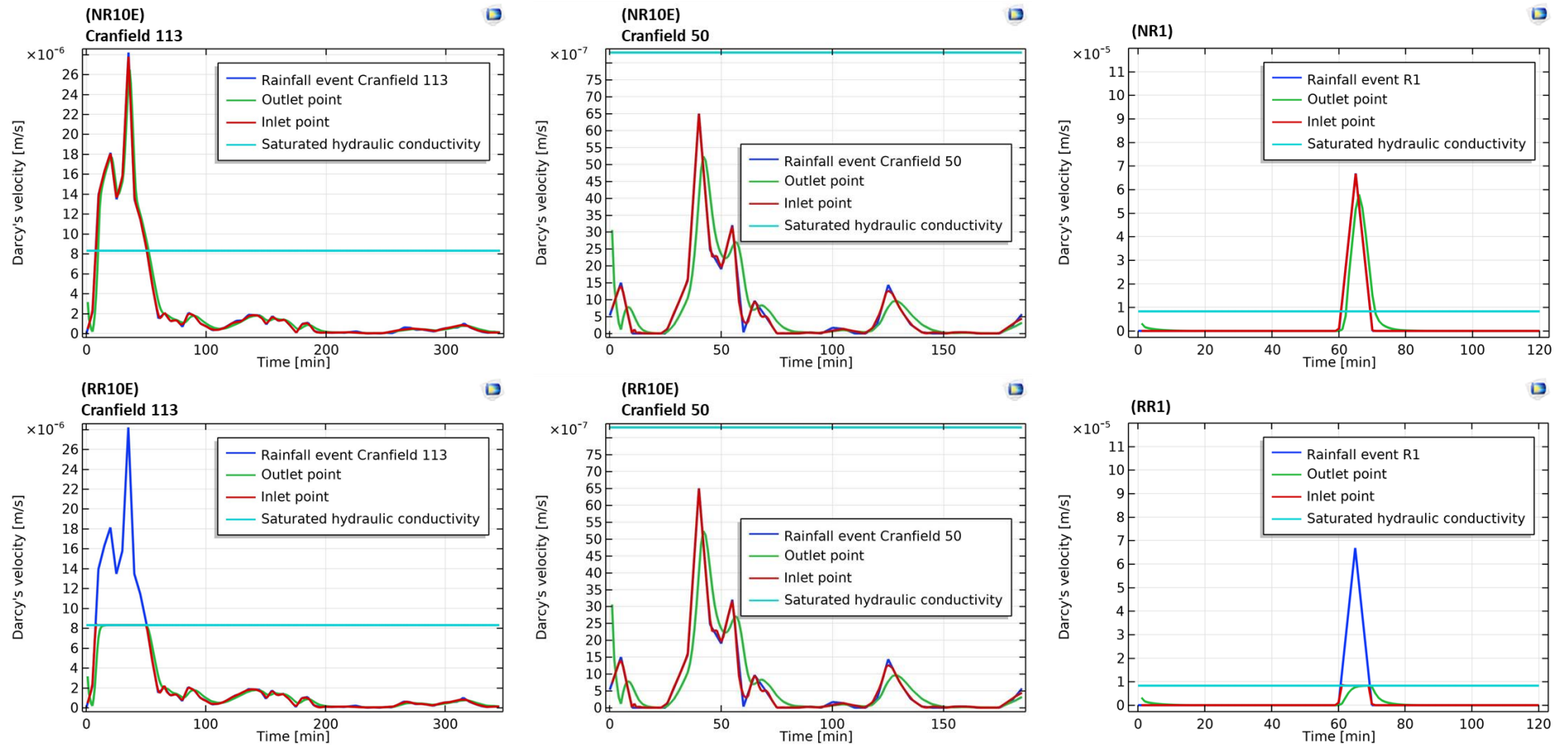


Figure 4-9 Darcy velocity results from COMSOL hydraulic model were plotted with time. Rainfall events of Cranfield 113 and Cranfield 50 have been shown here as the representative of ten selected rainfall events tests (R10E). The Darcy velocity result of R1 is also shown in the figure. Each figure plotted the Darcy velocity of rainfall, inlet, outlet, and saturated hydraulic conductivity against time.

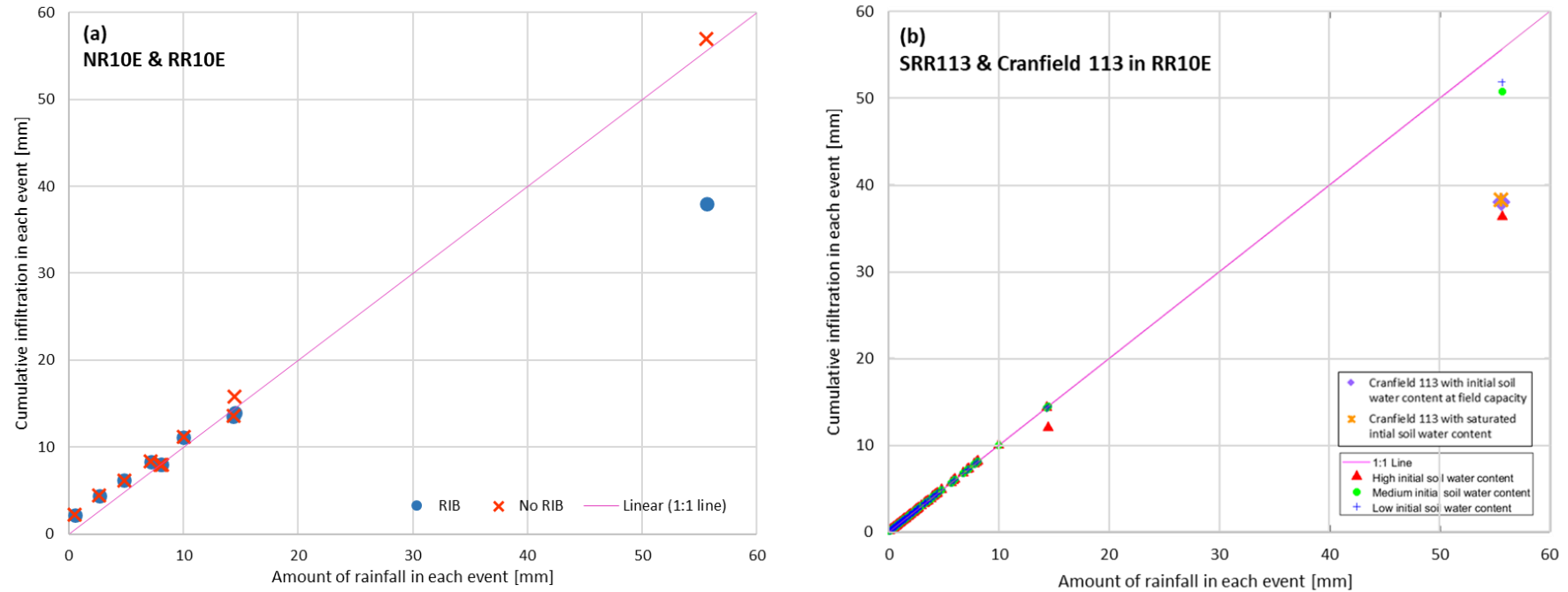


Figure 4-10 Cumulative infiltration in each event plotted against the amount of rainfall from the COMSOL hydraulic and Green-Ampt models. (a) indicates the results of the COMSOL hydraulic model with and without RIB, the inlet rainfall in the ten selected rainfall events (R10E). (b) shows the results of the Green-Ampt model (283 rainfall events), and the results of the COMSOL hydraulic model with RIB, including SRR113 and the results of Cranfield 113 from tests RR10E.

4.6 Discussion of comparison of RIB and Green-Ampt model

The cumulative infiltration of the RIB model was calculated from the Darcy velocity at the outlet point, and the depth of the structure is only 8.43 cm. However, the Green-Ampt only considered the amount of rainwater and infiltration depth without an outlet boundary. The hydraulic performance could be different with a thin soil layer, and the water withheld by soil can drain out with pressure gradient and cause slightly higher drainage compared to inlet rainfall.

For rainfall events whose intensity is not larger than K_s (Figure C-3), the cumulative filtration results of the hydraulic model with or without RIB are basically consistent. For Cranfield 25 and Cranfield 113 (Figure C-3), the cumulative filtration results of the hydraulic model with RIB indicate a much lower value than the hydraulic model without RIB. However, the value is closer to the Green-Ampt model results with saturated initial soil water content. The hydraulic model results with RIB can be supportive of each other, and more accurate results can be obtained with the RIB application.

4.6.1 Initial soil water content

According to Chapter 3, the initial soil water content will not significantly change the COMSOL hydraulic model results. However, initial soil water content did make a difference for the Green-Ampt model. As mentioned earlier, the thin layer geometry of the hydraulic model could compromise some of the soil hydraulic properties. The thin soil layer could absorb rainwater and become saturated rapidly. Therefore, the initial soil water content has less impact with a thin soil layer.

4.7 Design storm test (DST)

In order to determine the impact of clogging on runoff generation, the PGP system has been tested with a 'design storm'. A short, high-intensity design storm is a standard method to test the SUDS performance against flooding when the rainfall event is likely to exceed drainage system capacity (Woods-Ballard *et al.*, 2007). Commonly, the design storm used to test a SUDS technique considers the

regional landscape, overland flow requirement and SUDS design life (Woods-Ballard *et al.*, 2007; Ballard *et al.*, 2015). With local rainfall and geography information of a developed area or catchment, an appropriate design storm (design rainfall) can also be used to estimate the surface runoff rate and volume (Ballard *et al.*, 2015).

Although the concept of a 'design storm' has been widely used for hydraulic infrastructure design and testing worldwide, the methods for estimating the design storm are very different. The design storm estimation storm also differs in different countries. In the USA, these methods include the Chicago Design Storm (Keifer and Chu, 1957), Alternating Block Method (Chow, Maidment and Mays, 1988), Triangular Method (Yen and Chow, 1980), the Illinois State Water Survey design storm (Terstriep and Stall, 1974), and the National Resources Conservation Services design storm (Cronshey, 1986). In Canada, the Linear/exponential design storm method (Watt *et al.*, 1986) has been exploited. The Average Variability Method (Pilgrim and Cordery, 1975) was developed for design storm estimation in Australia. In Europe, the Sifalda method (Sifalda, 1973), the Double Triangle storm (Desbordes, 1978), and the G2P design storm (García-Bartual and Andrés-Doménech, 2017) were used in Czechoslovakia, France, and Spain. However, in the UK, the standard method for design storm estimation suggested by the 2015 CIRIA SUDS manual (Ballard *et al.*, 2015) is the Flood Estimation Handbook (FEH) method (Faulkner, 1999).

Depending on the design requirement, the CIRIA approach uses the return period, critical duration and local rainfall profile to estimate a design storm (Ballard *et al.*, 2015). The critical duration is "the length of a rainfall event that results in the greatest flow rate, flood volume or flood level at a particular location." (Ballard *et al.*, 2015).

A different design storm is expected to be used for different SUDS techniques. In England, the design return period varies from 1:1 year to 1:100 year, and the duration varies from 15 minutes to 48 hours (Ballard *et al.*, 2015). Woods-Ballard *et al.* (2007) specified that a 1:100 year design storm with less than 1-hour duration should be used for a drainage system. The design storm with a 1:30 year

return period should be considered for a conveyance system (Ballard *et al.*, 2015).

4.7.1 Rainfall intensity estimation for a design storm

The ReFH2 model (Wallingford Hydrosolutions, 2019) is the current FEH rainfall-runoff model, which the FEH design storm estimation method has also been implanted into the model.

Faulkner (1999) produced the FEH method for design storm estimation depending on the distance from the sea, the proximity of mountains, continentality and elevation. Six parameters are required, including the maximum rainfall (R_{FEH}), duration (D_{FEH}) and return period (T_{FEH}). The depth-duration-frequency relationship estimation equations of rainfall at a specific hour of D_{FEH} has presented by Faulkner (1999) as follows

$$D_{FEH} \leq 12 \text{ hours}, \ln R_{FEH} = (cy_{FEH} + d_1) \ln D_{FEH} + ey_{FEH} + f_{FEH}$$

$$12 < D_{FEH} \leq 48 \text{ hours}, \ln R_{FEH} = \ln R_{12} + (cy_{FEH} + d_2) \times (\ln D_{FEH} - \ln 12) \quad (4-21)$$

$$D_{FEH} > 48 \text{ hours}, \ln R_{FEH} = \ln R_{48} + (cy_{FEH} + d_3) \times (\ln D_{FEH} - \ln 48)$$

Where the Gumbel reduced variate $y_{FEH} = -\ln \left[-\ln \left(1 - \frac{1}{T_{FEH}} \right) \right]$, D_{FEH} is the rainfall duration [h], R_{FEH} is the maximum D_{FEH} -hour rainfall depth [mm], T_{FEH} is the return period of the event [year], c , e , f_{FEH} , d_1 , d_2 , d_3 are the parameters that have been mapped for the UK [-], and the local data can be downloaded from the FEH web service.

4.7.2 Design storm tests design

The rainfall duration should be less than 1 hour to maximize the rainfall intensity to challenge the hydraulic performance of the PGP system (Woods-Ballard *et al.*, 2007). Therefore, a 1:30 year 31 minutes storm was designed using the ReFH2 model (for the ReFH method, the duration is required to be an odd number). A rainfall event with a 1:30 year return period has a 57% probability of occurrence during the design life of 25 years, 82% for 50 years, and 97% for 100 years. The summer rainfall distribution profile was assumed for the design storm, as

recommended for an urban catchment. Using the depth-duration-frequency relationship (Cranfield, UK parameters are used for equation (4-21)) and the UK rainfall profile, the rainfall intensity for the design storm can be estimated and is shown in Figure 4-11. The every 1-minute rainfall intensity data has demonstrated in Appendix B Table B-1.

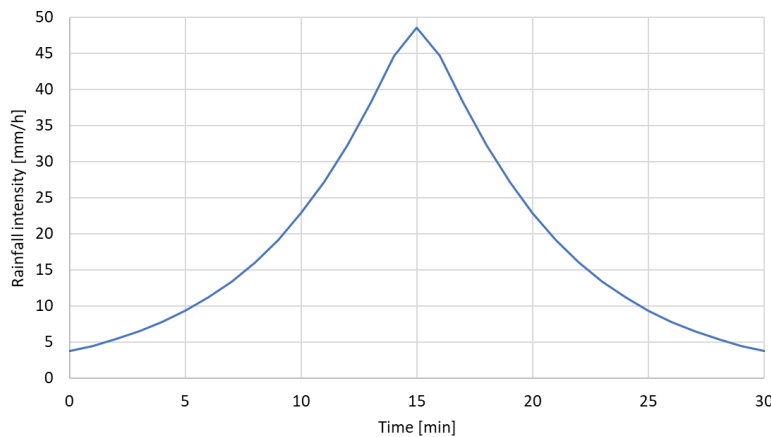


Figure 4-11 Rainfall intensity for a 1:30 year return period 31 minutes design storm at Cranfield, UK

According to the conclusion of Section 6.6.2, the clogged particles tend to concentrate on the top 2 cm of the surface soil. Different hydraulic parameters have been applied for the top 2 cm of sandy loam to determine the PGP system hydraulic performance by clogging (Figure 4-12). The parameters for sandy loam (from 2 cm to deeper), sand and geotextile layer stay the same as Table 4-10 in Section 4.4.3.

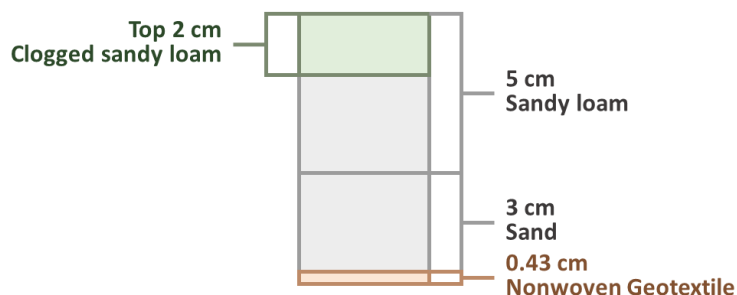


Figure 4-12 Cross-section of three layers structure for simulating the hydraulic performance of sandy loam, sand and geotextile. The top 2 cm sandy loam parameters have been modified for clogging.

The sandy loam porosity (θ_s) decreases as clogging develops, causing an increase in residual liquid volume fraction (θ_r), reduction in the saturated hydraulic conductivity (K_s), and changes of Van-Genuchten parameters (α and n). Therefore, the porosity reduction can represent the clogging level. Meanwhile, with a changing θ_s , the values of θ_r , K_s , α and n can be estimated using the ROSETTA soil characteristic parameters estimation model (Schaap, Leij and van Genuchten, 2001) and the curve fitting method (the curve fitting method will be introduced in Section 5.3.3.3). The ROSETTA model estimation data for the curve fitting method is shown in Appendix E, Table E-2. The characteristic parameters of the clogging top 2 cm sandy loam were estimated depending on porosity reduction using the equations summarized in Table 5-4 (Section 5.3.3.3).

To simulate the surface runoff in the urban area, the excess rainfall in each time step has been considered to become runoff immediately. Therefore, the RIB model (Section 4.4) has been applied to the design storm test. The saturated hydraulic conductivity is an important parameter determining whether the rainwater can infiltrate the soil surface (Section 4.4.5.2). The maximum inflow velocity on the soil surface would never be larger than the K_s value (Section 4.4.1). Under this premise, the characteristic parameters of the clogged upper 2 cm sandy loam were calculated (with equations in Table 5-4). When K_s is less than the minimum design storm rainfall intensity, it can be considered a failure of the PGP system (all rainwater becomes runoff). Starting at 0% of clogging (0% porosity reduction), the parameter calculation takes 5% increments until 15% when the K_s (at 0.0556 [mm min⁻¹]) less than the minimum rainfall intensity (at 0.0623 [mm min⁻¹]). The comparison of design storm rainfall intensity and K_s values at 0%, 5%, 10% and 15% clogging level has demonstrated in Figure 4-13.

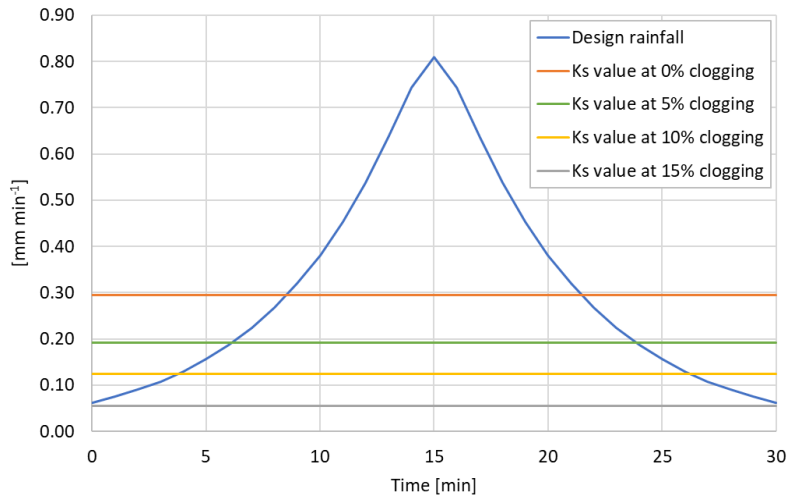


Figure 4-13 Comparison of rainfall intensity in design storm and Saturated hydraulic conductivity at 0%, 5%, 10% and 15% clogging levels.

The clogged top 2 cm sandy loam characteristic parameters is presented in Table 4-11. The values of θ_s , θ_r , K_s and n were calculated using the ROSETTA model curve fitting equations (Table 5-4, Section 5.3.3.3). However, values of van-Genuchten parameter α were not calculated by the curve fitting equations (Table 5-4) but were obtained directly from the ROSETTA model predicted value (according to Appendix D.2.3, when θ_s is greater than 0.25, the fitted curve of α is not accurate enough).

Table 4-11 Sandy loam characteristic parameters for the clogged top 2 cm soil (calculation based on equations in Table 5-4).

Porosity reduction		0%	5%	10%	15%
Porosity	θ_s [-]	0.3900	0.3705	0.3510	0.3149
Residual liquid volume fraction	θ_r [-]	0.0299	0.0296	0.0302	0.0321
Saturated hydraulic conductivity	K_s [cm d ⁻¹]	42.4610	27.5252	17.8431	8.0018
van-Genuchten parameter	α [cm ⁻¹]	0.0469	0.0444	0.0429	0.0423
	n [-]	1.3862	1.3435	1.3048	1.2436

The design storm and clogged sandy loam characteristic parameters have been applied to the 3-layer RIB model (Section 4.4). The results were recorded every 1 minute.

4.7.3 Design storm test results

The total amount of rainfall of the design storm is 9.5493 [mm]. The runoff depth can be calculated by the difference between total rainfall and infiltrated water at 2 cm depth (bottom of the clogged sandy loam layer). The results have been summarized in Table 4-12 blue rows. As the clogging level increases, the cumulative infiltration dramatically decreases, and the runoff depth rises significantly. According to the model results, nearly half of the rainfall becomes runoff at 5% clogging and more than 60% at 10% clogging. When there is no clogging, runoff is still generated since the design storm has exceeded the infiltration capacity of the designed PGP system.

The infiltration at 2 cm depth of the bottom of the clogged sandy loam layer has been plotted in Figure 4-14. As described above in Section 4.7.2, the maximum infiltration reduces as the clogging level increases and K_s decreases. The infiltration rises as the rainfall starts to the maximum K_s , then declines with the rainfall intensity with a slight hysteresis phenomenon (the hysteresis phenomenon has been discussed in Section 3.7.3). When the rainfall finishes at 30 minutes, the infiltration also reduces to 0 [mm min⁻¹].

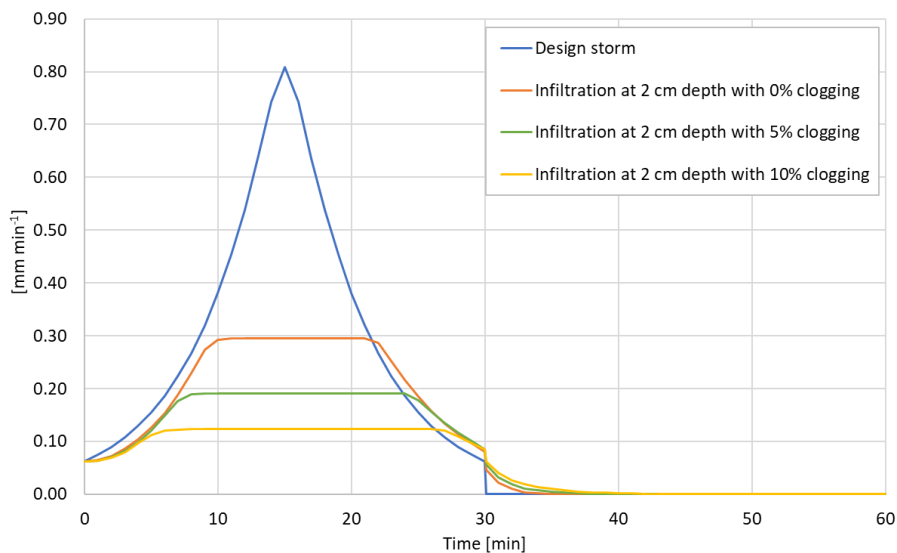


Figure 4-14 Modelled infiltration results at 2 cm depth (bottom of clogged sandy loam layer) and rainfall intensity of the design storm.

The simulated infiltration curve is basically consistent with rainfall, and the maximum value does not exceed the K_s value. Therefore, it is possible to predict runoff using only K_s . In each time step, use rainfall intensity minus K_s to estimate runoff. The results of this calculation are shown in Table 4-12 (green rows).

Comparing the modelled runoff and K_s calculated runoff, at the same clogging level, the runoff reduction differences are less than 2%. Therefore, the runoff reduction of 15% clogging has been predicted using K_s calculation method, more than 82% of rainfall has become runoff.

Table 4-12 Runoff calculation for different clogging levels using the RIB model (blue rows) and K_s value (green rows).

<i>Clogging level</i>	<i>0%</i>	<i>5%</i>	<i>10%</i>	<i>15%</i>
<i>Modelled cumulative infiltration at 2 cm depth [mm]</i>	6.5069	4.9808	3.6941	-
<i>Modelled runoff [mm]</i>	3.0423	4.5685	5.8551	-
<i>Modelled runoff reduction compared to rainfall</i>	68.1%	52.2%	38.7%	-
<i>Calculated runoff using K_s value [mm]</i>	3.1200	4.6850	6.0285	7.8266
<i>Calculated runoff reduction using K_s value compared to rainfall</i>	67.3%	50.9%	36.9%	18.0%

4.7.4 Design storm test discussion

As the clogging level increases, the infiltration capacity of the PGP structure reduces dramatically, especially when the clogging level is higher than 10%. This is not surprising; when the porosity is reduced to half or less than its original value, the properties of the soil are changed, and the saturated hydraulic conductivity is significantly reduced. The infiltration capacity will be strongly compromised by clogging and cause a system failure for the design storm test. When the top 2 cm of soil is more than 15% clogged, the structure can still maintain a small amount of infiltration at K_s value. It is unlikely to completely seal the surface by clogging for a permeable pavement system (Ballard *et al.*, 2015). Although the PGP system can no longer maintain a high permeability at this time, under light rain events, PGP can increase the permeable surface area of the city

to a certain extent. The SUDS Manual (Ballard *et al.*, 2015) recommends applying a safety factor of 10 times higher than adequate design to all permeable pavement designs to tolerate reduced drainage capacity due to blockages during the design life.

Compared to an absolute impermeable surface (all rainfall becomes runoff), the no clogged PGP systems exhibited high surface runoff removal rates (68%) for an extreme design storm event. The runoff reduction rate is roughly consistent with a previous study that the runoff reduction for the PGP system is 52-62% (Jayasuriya and Kadurupokune, 2010). As expected, the runoff removal rates decrease dramatically as clogging develops. The runoff removal rate for 10% and 15% of clogged PGP systems is 38.7% and 18%, respectively.

It is vital to notice that the runoff estimation between RIB modelled results and K_s estimation results do not show an enormous difference. The runoff can be roughly estimated using only the estimated K_s value under different reduced porosity values. Therefore, the hydraulic performance of the PGP system can be approximated by K_s at different clogging levels. This simple method is helpful for PGP system design at the early point.

4.8 Conclusion

In summary, the application of RIB fulfilled the requirement of simulating the surface runoff in the COMSOL hydraulic model. The concept of RIB corresponded to the Green-Ampt model principles, and the hydraulic model results with RIB also agreed with the Green-Ampt results.

Although the initial soil water content makes a dramatic difference for the Green-Ampt model, the thin layer geometry of the RIB model makes it negligible.

The results of the Green-Ampt model and the RIB model have been mutually verified with 283 rainfall events. The results of both models roughly agreed with each other, which indicates a trustworthy accuracy for the RIB to be applied to the PGP-HRC model.

The design storm test results suggested that the infiltration capacity of the PGP system is strongly affected by clogging. With a 1:30 year 31 minutes design storm for Cranfield, UK, the infiltration capacity of the PGP system reduces significantly with a larger clogging level. Under the design storm, the surface runoff will be generated with the initial condition of the PGP system. However, a large amount of runoff can be generated with clogged permeable pavement. The saturated hydraulic conductivity is a vital parameter for the runoff generation rate of a PGP system. A predicted K_s value at different clogging levels can be used to approximate the hydraulic performance of the PGP system.

5 Chapter Five: Method of the clogging model

5.1 Introduction

This chapter purposed to introduce the method of the third part (clogging model) of the PGP-HRC model. Because of the small pore spaces, the PGP system can be clogged with suspended particles in infiltrated water, organic matters and fines ground by heavy load or traffic (Siriwardene, Deletic and Fletcher, 2007a). Eventually, it led to a significant decrease of the infiltration rate to an unacceptable level and caused system failure (Gerrits and James, 2002; Borgwardt, 2006; Siriwardene, Deletic and Fletcher, 2007a; Yong *et al.*, 2010; Mullaney and Lucke, 2014).

5.1.1 Definition of suspended particles

Suspended particles are particles that can be found in the water column. The particle size varies from 0.01 μm to 10 μm (Tufenkji and Elimelech, 2004). Most suspended particles are composed of inorganic materials (Langland and Cronin, 2003). However, bacteria, plankton, algae, organic matter, plants and animals decay, and chemical precipitates can also be considered as suspended solids (Wetzel, 2001). During a rainfall event, the particles are washed from the surface, carried by the surface runoff and moved to the permeable pavements (Razzaghmanesh and Borst, 2018). Due to the infiltration process, the suspended particles accumulate in the openings of permeable pavement and cause clogging (Razzaghmanesh and Borst, 2018). Suspended particles that cause clogging come from various biological, chemical or physical sources (Yong, McCarthy and Deletic, 2013).

5.1.2 Aim

The clogging process of the PGP system can be treated as the suspended particle transport and accumulating processes in porous media. The clogging model aims to provide the suspended particle transport profile for the PGP system long-term simulation clogging model, and it is also the third part of the PGP-HRC model.

5.1.3 Overview of approach

A theory-based mathematical model has been presented in this chapter to simulate the clogging processes in the PGP system. Notably, the clogging model presented in this chapter is part of the PGP-HRC model (Section 1.12).

The three parts of the PGP-HRC model are connected and influenced by each other. The relationship between the hydraulic, RIB and clogging part of the PGP-HRC model indicated in Figure 5-1. An integral approach is linking the clogging model to the hydrological model to evaluate the feedback.

The hydraulic model provides the water flow as a driven force and carries the suspended particles across the matrix. The clogging model simulated the process of suspended particles retained in the matrix and changing the matrix properties (porosity, hydraulic conductivity, media grain size and retention parameters). The new matrix property parameters will then be input into the hydraulic and RIB model, influencing the infiltration performance. Among the loops and feedback, estimating soil properties and van-Genuchten parameters during the clogging process is critical in connecting the clogging model with the RIB and hydraulic model.

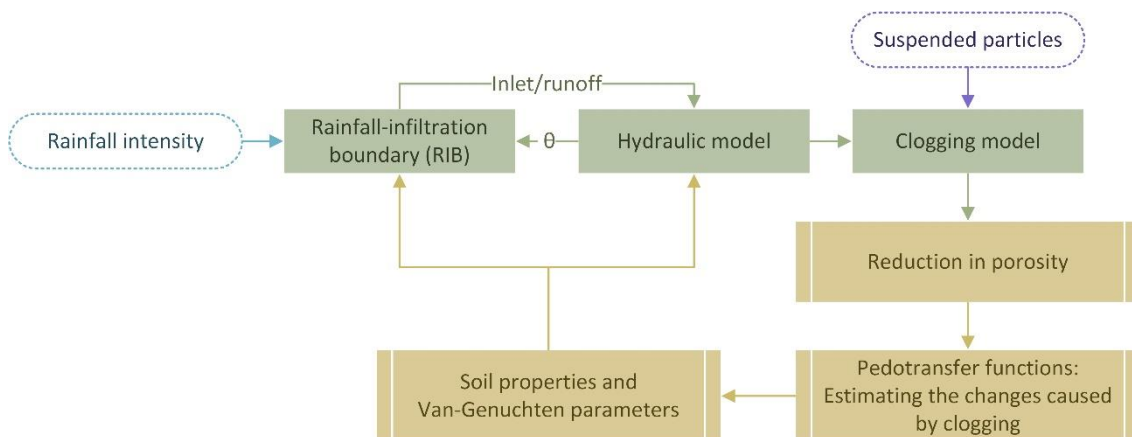


Figure 5-1 Three parts (hydraulic, RIB and clogging) of the PGP-HRC model and the connection process between them.

The model contains three layers: a soil layer, a sand layer, and a geotextile layer (as presented in Section 1.12.1). The model has been built up using COMSOL Multiphysics.

5.2 Model method

5.2.1 Model principles for suspended particle clogging

Suspended particle transport in porous media is affected by different forces. Apart from advection and dispersion behaviours affecting the concentration of suspended particles (SS) and PGP bed depth and changes with time, total suspended particles (TSS) are also transported due to interception, sedimentation, and diffusion processes.

There are two distinct and separate steps involved in the suspended particle removal process within a filter media:

1. The suspended particle travels along the velocity direction until it reaches the vicinity interface between solid and liquid (the interface can be made by the media grain or previously remaining particles);
2. Suspended particles attach to the surface of the media grain or previously remaining particles.

In each time interval, the amount of adsorbed particles on the solid is proportional to the concentration of suspended particles and the velocity of the fluid. Driven by the water flow velocity, the sedimentation and deposition of contaminant particles happen in the upper layer, causing a decrease in particle concentration in water. The percentage of clay increases and fills up the pores inside the soil structure; the porosity decreases and causes clogging.

5.2.1.1 Yao Model

To develop a conceptual model for particle destabilisation and transport in water and wastewater, Yao, Habibian and O'Melia (1971) introduced a particle trajectory calculation concept to investigate the physicochemical particle flow through pack-bed filtration. As shown in Figure 5-2, the concept considered a single spherical particle of the filter media as a collector. A suspended particle flowing along the streamline in a concentric spherical space surrounding the single collector may contact the collector affected by three transport mechanisms (interception, sedimentation and diffusion). Typically, the main flow direction depends on gravity (Yao, Habibian and O'Melia, 1971).

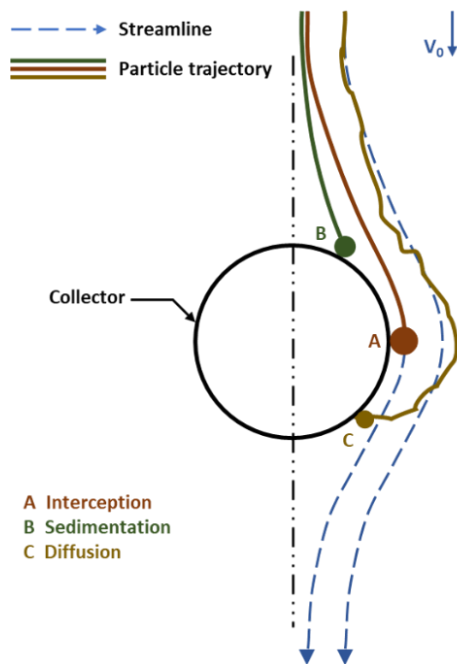


Figure 5-2 Basic transport mechanisms in water filtration processes (Source: Yao, Habibian and O'Melia, 1971)

These processes of particle transport are affected by different forces (Yao, Habibian and O'Melia, 1971):

- *Interception:* The suspended particle follow the streamline and might contact the collector by its size.
- *Sedimentation:* The density of the suspended particle is more extensive than water. The particle affected the combined force of buoyancy and gravitational force, causing it to deviate from the streamline.
- *Diffusion:* The particle is involved in random molecular bombardment with the suspended medium, resulting in Brownian diffusion.

5.2.2 Modelling for suspended particle clogging

Several models have been developed for the transport and deposition of suspended particles in porous media. Yao, Habibian and O'Melia (1971) started with a general equation derived from a widely used equation by engineers for describing the pollutants' behaviours in the atmosphere and estuaries streams. This is followed by a theory-based single collector efficiency model for presenting the contact efficiency between the suspended particles and the collector (media

grain). The Yao model was later improved by the R-T model (Rajagopalan and Tien, 1976) for a deep packed-bed filtration model. Most recently, the R-T model has been corrected by Tufenkji and Elimelech (2004).

5.2.2.1 Traditional R-T model

Rajagopalan and Tien (1976) developed a straightforward semi-empirical equation based on the trajectory method from Yao, Habibian and O'Melia (1971), which is widely known as the R-T model. R-T model also uses the Happel (1958) initiated method to better address the flow restriction caused by adjacent collectors in the packed bed filter.

For the R-T model, all terms of particle removal mechanisms considered the restricted flow within the concentric spherical space surrounding the collector. Their calculations include reducing collisions due to the resistance of the incompressible fluid being pushed out between two colliding particles, called the lubrication effect, and the influence of the London van der Waals gravitational force (Logan *et al.*, 1997). Compared to the Yao model, the R-T model developed two equations that numerically represent the suspended particle removal by interception and gravitational sedimentation (Logan *et al.*, 1997).

Rajagopalan and Tien (1976) proposed a function between the initial collection efficiency and four parameters (ratio of suspended particle size to the media grain size, the extent of the surface attraction force, and the magnitude of the gravitational collection Brownian diffusion).

Logan *et al.* (1997) have clarified the original R-T equation by Rajagopalan and Tien (1976) with an accurate version and was used by Martin *et al.* (1996) and Yu and Rowe (2012). The R-T model makes it possible to use the trajectory solution directly without extensive numerical calculations (Rajagopalan and Tien, 1976).

5.2.2.2 Corrected R-T model

Tufenkji and Elimelech (2004) presented several limitations of the R-T model that caused inaccurate practical relevance for predicting the single collector efficiency. The major disadvantage of the R-T model is that it ignores the effects

of the interaction between hydrodynamics and Van der Waals forces on particle deposition that is dominated by Brownian diffusion (Tufenkji and Elimelech, 2004).

Tufenkji and Elimelech (2004) re-evaluated the R-T equation based on the strict numerical solution of the convection-diffusion equation, and all particle removal mechanisms and interaction forces have been taken into account.

Although the corrected R-T model is a relatively thorough filtration model, the model can be applied to a range of controlled environments, especially for suspended particle size and media grain size restriction.

Table 5-1 Summary of parameter values used in corrected R-T model numerical calculations (Source: Tufenkji and Elimelech, 2004)

Parameters	Range
Suspended particle diameter	0.01 to 10 [μm]
Collector (Media grain) diameter	0.05 to 0.50 [mm]
Fluid approach velocity	7×10^{-6} to 2×10^{-3} [$m s^{-1}$]
Hamaker constant	3×10^{-21} to 4×10^{-20} [J]
Suspended particle density	1.0 to 1.8 [$g cm^{-3}$]
Fluid absolute temperature	298 [K]
Media porosity	0.36

Table 5-1 summarises the range of parameters that have been used for the calculation of Tufenkji and Elimelech (2004). For the suspended particle diameter range from around 0.1 to 4 μm , the corrected R-T model is more accurate in predicting single collector efficiency. (Tufenkji and Elimelech, 2004).

5.2.3 Modelling the effect of clogging on soil hydrology

Porous media hydraulic characteristics (Including water retention and hydraulic conductivity) are essential parameters for hydraulic and clogging modelling. As described above in Section 5.1.3, soil properties and water retention change as clogging develop and alter the PGP-HRC model's hydraulic profile. The conceptual model in Section 1.7 indicates that the hydraulic profile drives the contaminant transport in the PGP system, which makes it crucial for the PGP-

HRC model. The interaction creates a loop between hydraulic performance and clogging progress.

Previous research uses different semi-empirical or empirical methods to predict each soil property value separately, ignoring the van-Genuchten parameters' changes. However, Pedotransfer functions (PTFs) allow simultaneous prediction of each soil property (saturated hydraulic conductivity, residual liquid volume fraction) and van-Genuchten parameters changes with porosity.

Most of the previous soil hydraulic parameters prediction models can be classified as PTFs. PTFs are calibrated by measuring the soil hydraulic database, and then estimating other soil hydraulic data by the surrogate parameters (such as bulk density, particle size distribution and matter content). However, the best results of PTFs highly depend on the existing database that has been used and requires some detailed or specific variables (Schaap, Leij and van Genuchten, 2001).

5.2.3.1 ROSETTA Model

ROSETTA is a computer program developed by Schaap, Leij and van Genuchten (2001) that implements many PTFs for estimating water retention parameters, saturated and unsaturated hydraulic conductivity, and related uncertainties. ROSETTA also implemented the author's previously proposed model (Schaap and Leij, 1998, 2000; Schaap, Leij and van Genuchten, 1998).

To accurately predict the water retention parameters, saturated and unsaturated hydraulic conductivity, ROSETTA implements five hierarchical PTFs models. The hierarchical structure in PTFs allows the use of limited (texture only) to more extended (texture, bulk density, and one or two water retention points) input data to predict characteristic water parameters (Schaap, Leij and van Genuchten, 2001). ROSETTA depends on the combination of neural network analysis and the bootstrap method, making it possible to estimate the uncertainty during the prediction of hydraulic parameters (Schaap, Leij and van Genuchten, 2001). The coefficients of determination and root mean square errors (RMSEs) were applied to characterise the general performance of ROSETTA.

A large number of hydraulic soil data from three databases has been used for ROSETTA calibration. The database contains 2,134 soil samples, most of them derived from North America and Europe (Schaap and Leij, 1998).

5.3 Mathematical approach

TSS removal efficiency model combined three models 1) the traditional mass balance equation, 2) the Filtration coefficient model (Yao, Habibian and O'Melia, 1971; Martin *et al.*, 1996), 3) the Single collector contact efficiency model (Rajagopalan and Tien, 1976; Tufenkji and Elimelech, 2004) and 4) ROSETTA model (Schaap, Leij and van Genuchten, 2001).

The Governing equation for the clogging model is the mass balance equation (COMSOL, 2018)

$$\theta \frac{\partial C}{\partial t} = \underbrace{\theta \cdot D \frac{\partial^2 C}{\partial x^2}}_{\text{Dispersion}} - \underbrace{\theta \cdot v \frac{\partial C}{\partial x}}_{\text{advection}} \pm \underbrace{R}_{\text{Reaction}} \quad (5-1)$$

Here, θ is the liquid volume fraction, C is the particulate matter concentration in the liquid phase [kg m^{-3}], D corresponds to the coordinate components of the hydrodynamic dispersion coefficient (explained in Section 5.3.4), and R is the reaction field, increase or decrease of particles/concentration because of a chemical reaction or clogging.

In the model presented in this thesis, the reaction field is mainly caused by decreasing particles from the liquid phase. The particles that leave the liquid phase will stick to the solid phase, resulting in clogging.

5.3.1 Filtration coefficient

The reduction rate of particles from the liquid phase can be described as

$$R = \rho_p(1 - \theta) \frac{\partial C_s}{\partial t} = \theta v \lambda C \quad (5-2)$$

ρ_p is the density of the suspended particle [kg m^{-3}], C_s is the accumulative particle concentration in the solid phase [kg m^{-3}], v represents the Darcy velocity [m s^{-1}], and λ is the filtration coefficient [m^{-1}]. The driving force in this equation is velocity. The equation can be obtained from the chain rule as (Martin *et al.*, 1996)

$$\frac{\partial C}{\partial t} = \frac{\partial x}{\partial t} \frac{\partial C}{\partial x} \quad (5-3)$$

$$\frac{\partial x}{\partial t} = \theta v \text{ (Darcy velocity)} \quad (5-4)$$

5.3.1.1 Filtration coefficient for soil and sand (Yao model)

For soil/gravel, the filtration coefficient can be described by λ_s (Yao, Habibian and O'Melia, 1971)

$$\lambda_s = -\frac{3(1 - \theta)}{2d} \alpha_{collision} \cdot \eta_s \quad (5-5)$$

$\alpha_{collision}$ is the collision efficiency factors [-], η_s is the single spherical collector efficiency for soil and sand [-] (Section 5.3.2.1), and d is the real-time porous media grain diameter [m].

The one-dimensional filtration equation developed by Yao, Habibian and O'Melia (1971) can be used to calculate the fraction of particles remaining in the liquid phase after transmission through a column (Point A to B) of porous media with a length L

$$\ln \frac{C}{C_0} = -\frac{3(1 - \theta)}{2d} \alpha_{collision} \eta_s L \quad (5-6)$$

The suspended particle concentration at point A is C_0 , and C for point B. Along the x axis

$$\frac{\partial C}{\partial x} = -\frac{3(1-\theta)}{2} \frac{1}{d} \alpha_{collision} \eta_s C \quad (5-7)$$

The collision rate can be calculated as (Martin *et al.*, 1996)

$$\frac{\partial C}{\partial t} = \frac{\partial x}{\partial t} \frac{\partial C}{\partial x} = \theta v \cdot (-\lambda C) \quad (5-8)$$

Therefore, equation (5-5) can be obtained by combining equations (5-7) and (5-8).

Collision efficiency factors

In most water-based environments, because of the repulsive colloidal interaction between suspended particles and media grains, the actual collector removal efficiency would be lower than the single collector contact efficiency (Tufenkji and Elimelech, 2004). When the suspended particles collide with the media grains, the probability of the particles sticking to the grains needs to be quantified by the collision efficiency factor $\alpha_{collision}$. Therefore, α should consider 1) all the chemical reactions between particles and media, and 2) the inaccuracy during calculating single collector contact efficiency (Martin *et al.*, 1996).

Collision efficiency factor $\alpha_{collision}$, which is reflected the chemistry of the system and is defined as the ratio of the rate of suspended particles sticking to the collector to the collisions that occur between the suspended particles and the collector (Yao, Habibian and O'Melia, 1971).

Under the assumption of a completely destabilised environment (the particles are entirely randomly distributed), the removal of suspended particles is ideally random, $\alpha_{collision} \approx 1$ (Yao, Habibian and O'Melia, 1971; Tufenkji and Elimelech, 2004).

The value of the collision efficiency factor $\alpha_{collision}$ for clean filters has been successfully calibrated by Siriwardene, Deletic and Fletcher (2007b) and summarised in Table 5-2.

Table 5-2 Collision efficiency factors value (Source: Siriwardene, Deletic and Fletcher, 2007b)

Medial grain classification	Particle size fraction	Collision efficiency factor $\alpha_{collision}$ [-]
Clay	< 2 [μm]	2.208
Fine silt	2 to 6 [μm]	0.931
Medium silt	6 to 20 [μm]	0.142
Coarse silt	20 to 63 [μm]	0.047
Fine/medium sand	60 to 300 [μm]	0.013

5.3.1.2 Filtration coefficient for geotextile

Iwasaki (1937) concluded that the filtration coefficient for geotextile could be described as equation (5-9), and the same method was also used by Liu *et al.*, (2018).

$$\lambda_g = \frac{-\ln(1 - \eta_g)}{a} \quad (5-9)$$

a is the thickness of the geotextile [m], η_g is the retention ratio for clean geotextile ($0 < \eta_g < 1$) [-].

5.3.2 Single collector contact efficiency

5.3.2.1 Single collector contact efficiency for soil and sand (Corrected R-T model)

When all mechanisms are considered simultaneously, the particle deposition rate calculated by adding up the individual contributions of each transport mechanism is consistent with the deposition rate obtained from the rigorous solution of the convection-diffusion equation (Yao, Habibian and O'Melia, 1971; Prieve and Ruckenstein, 1974; Tufenkji and Elimelech, 2004).

For single collector efficiency, we assumed (Yao, Habibian and O'Melia, 1971; Tufenkji and Elimelech, 2004)

$$\eta_s = \eta_D + \eta_I + \eta_G \quad (5-10)$$

Here η_D presents the mechanisms of transport by diffusion, η_I indicates the mechanisms of transport by interception, and η_G denotes the gravitational transport mechanisms.

Therefore, the single collector efficiency for soil η_s can be expressed as (Tufenkji and Elimelech, 2004):

$$\begin{aligned} \eta_D &= 2.4A_s^{\frac{1}{3}}N_R^{-0.081}N_{Pe}^{-0.715}N_{vdW}^{0.052} \\ \eta_I &= 0.55A_sN_R^{1.675}N_A^{0.125} \\ \eta_G &= 0.22N_R^{-0.24}N_G^{1.11}N_{vdW}^{0.053} \end{aligned} \quad (5-11)$$

A_s , N_R , N_{Pe} , N_{vdW} , N_A and N_G are dimensionless numbers.

A_s is porosity-depended parameter.

N_R is aspect ratio of media grain size and suspended particle size.

N_{Pe} is Pelect number (ratio of convective transport to diffuse transport).

N_{vdW} is London-van der Waals number (ratio of van der Waals' interaction energy to particle thermal energy).

N_A is attraction number (characterising the combined effect of van der Waals force and fluid velocity on the particle deposition rate caused by interception).

N_G is gravity number (ratio of Stoke's particle sedimentation velocity to fluid approach velocity).

$$A_s = \frac{2(1 - y^5)}{2 - 3y + 3y^5 - 2y^6} \quad (5-12)$$

Where $y = (1 - \theta)^{1/3}$

$$N_{Pe} = \frac{vd}{D_{\infty}} \quad (5-13)$$

$$D_{\infty} = \frac{BT}{6\pi\mu r_p} \quad (5-14)$$

$$N_A = \frac{Ham}{12\pi\mu r_p^2 v} \quad (5-15)$$

$$N_G = \frac{2 r_p^2 (\rho_p - \rho_f) g}{9 \mu v} \quad (5-16)$$

$$N_R = \frac{d_p}{d} \quad (5-17)$$

$$N_{vdW} = \frac{Ham}{BT} \quad (5-18)$$

θ is Liquid volume fraction [-], D_{∞} is the Diffusion coefficient in an infinite medium [$\text{m}^2 \text{s}^{-1}$], B is Boltzmann's constant [J K^{-1}], T is the absolute temperature [K], μ is the water viscosity [$\text{kg m}\cdot\text{s}^{-1}$], r_p is the radius of the suspended particle [m], Ham is the Hamaker constant [J], ρ_f is the density of the fluid (water) [kg m^{-3}], and d_p is the suspended particle size [m].

To compute the single collector contact efficiency for soil and sand (η_s), constants and variables A_s , N_R , N_{Pe} , N_{vdW} , N_A and N_G should be brought into equations (5-10) and (5-11).

5.3.2.2 Retention ratio for clean geotextile

Currently, there is no adequate model to predict the retention ratio η_g for clean geotextile. However, Liu *et al.* (2018) determined the values of η_g for different suspended sizes under different porosities in the filtration tests, the values have presented in Table 5-3.

Table 5-3 Retention ratio of geotextile for suspended particles of difference sized under different porosities (Source: Liu *et al.*, 2018)

<i>Particle diameter range [μm]</i>	<i>Representative particle diameter [μm]</i>	<i>Retention ratio η_g (here, x represents the tested porosity)</i>	<i>Additional information</i>
65 to 150	107.5	0.999	The geotextile would retain all the particles larger than 65 μm at the beginning of the test.
30 to 60	47.5	$\eta_g = -0.189 \ln(x) + 0.76$	-
15 to 30	22.5	$\eta_g = -0.475 \ln(x) + 0.26$	-
7.0 to 15	11	$\eta_g = -0.478 \ln(x) + 0.10$	-
1.6 to 7.0	4.3	$\eta_g = -0.148 \ln(x) + 0.10$	-
0.4 to 1.6	1.0	0.15	η_g is relatively low and abnormal.

5.3.3 Coupling relationship among parameters

Based on the mass balance theory, the total mass loss in the liquid phase equals the increased sedimentary particle concentration in the solid phase.

$$\text{(Solid phase)} \quad \rho_p \frac{\partial C_s}{\partial t} = -\theta \frac{\partial C}{\partial t} \quad \text{(Liquid phase)} \quad (5-19)$$

Here, ρ_p is the density of suspended particles in the solid phase [kg kg^{-1}], C_s is particle concentration in solid phase [kg m^{-3}], and C is particle concentration in liquid phase [kg m^{-3}].

5.3.3.1 Porosity changes with time

The suspended particle deposition in the porous media leads to a reduction of porosity, and the relationship can be presented as (Liu *et al.*, 2018):

$$\theta = \theta_0 - \frac{C_s}{\rho_p(1 - \varepsilon_p)} \quad (5-20)$$

θ_0 is the initial porosity [-]. ε_p is the inside porosity of packed suspended particles [-], by dividing $(1 - \varepsilon_p)$, the actual porosity of porous media blocked by suspended particles can be estimated (Vigneswaran and Chang, 1989; Liu *et al.*, 2018).

5.3.3.2 Porous media grain diameter changes with porosity

We assume that when sedimentation happening, the suspended particles uniformly cover the media grains. The equivalent diameter of media grain increases with suspended particles attached to the surface. The equivalent diameter of media grain can be expressed as the following equation:

$$d = d_0 \times \sqrt{1 + \frac{\theta_0 - \theta}{\pi}} \quad (5-21)$$

Here d_0 is the initial porous media grain diameter [m].

5.3.3.3 Soil and sand characteristics parameters estimation (ROSETTA model)

As the smallest particle size considered in the ROSETTA model, clay has been used to simulate the suspended particles. Suspended particle accumulation in porous media can be assumed as increasing clay percentage in soil texture distribution.

In ROSETTA, the value of clay percentage can be set manually. However, the bulk density value is also needed as an input parameter to obtain more accurate results.

The relationship between porosity and initial bulk density of soil can be calculated as follows:

$$\theta_s = 1 - \left(\frac{\rho_b}{\rho_p} \right) \quad (5-22)$$

θ_s is porosity [-], ρ_b is bulk density [g cm⁻³], and ρ_p is soil/sand particle density (2.650 [g cm⁻³]).

For simulating the clogging processes, initially assumed that the porous media column's total volume equals 1 [cm³], and clay mass equals 0 [g]. The total mass per volume can be assumed to be equal to bulk density; the mass of silt and sand can be calculated using their percentage. In each step, the clay mass has been increased by 0.05 [g], and the bulk density, porosity, and percentage of silk, clay and sand have been re-calculated. It is essential to note that the volume taken by clay should not be more than the initial porosity. The calculated values for sandy loam and sand can be found in Appendix E.

Saturated hydraulic conductivity, residual liquid volume fraction (θ_r) and the van-Genuchten parameters α and n were plotted against porosity, and best-fit curves were determined using the MATLAB curve fitting tool. The fitting curve (Appendix D) shows a high correlation between porosity and soil hydraulic characteristics. The correlation equation can be used as an input function for the clogging model to estimate the soil hydraulic characteristics value changes alone with clogging development.

It is vital to notice that the input soil texture distribution and bulk density significantly impact ROSETTA model prediction. The correlation equation needs to be re-calculated when changing the soil texture. Table 5-4 summarises the soil hydraulic parameters for sandy loam and sand changes with porosity.

Table 5-4 Table of soil hydraulic parameters for sandy loam and sand changes with porosity (Appendix D)

Parameters	Fitted curve	R ²
Saturated hydraulic conductivity	$K_{s(sand)} = 0.0003548e^{37.91\theta}$	0.9999
	$K_{s(sandy\ loam)} = 0.007291e^{22.23\theta}$	0.9995
The residual liquid volume fraction	$\theta_{r(sand)} = 0.09389e^{-3.168\theta} + 0.003167e^{4.697\theta}$	1.0000
	$\theta_{r(sandy\ loam)} = -2.173e^{-10}e^{41.82\theta} + 0.06255e^{-2.125\theta}$	0.9979
van-Genuchten parameter α	$\alpha_{(sand)} = 0.08718\theta^{1.41} + 0.01656$	0.9972
	$\alpha_{(sandy\ loam)} = 0.08813e^{-1.153\theta} + -0.06794e^{-4.524\theta}$	0.9746
van-Genuchten parameter n	$n_{(sand)} = 1.872e^{-1.871\theta} + 0.002068e^{18.81\theta}$	0.9996
	$n_{(sandy\ loam)} = 0.8571e^{-3.145\theta} + 0.393e^{2.719\theta}$	0.9979

5.3.3.4 Non-woven geotextile hydraulic conductivity estimation

Kozeny-Carman equation represented the relationship between porosity and saturated hydraulic conductivity (Mitchell and Soga, 2005; Liu *et al.*, 2018):

$$K_{sat,t} = K_{sat,0} \times \frac{\theta^3}{(1-\theta)^2} \times \frac{(1-\theta_0)^2}{\theta_0^3} \quad (5-23)$$

$K_{sat,t}$ is the real-time saturated hydraulic conductivity [$m\ s^{-1}$], and $K_{sat,0}$ is the initial saturated hydraulic conductivity [$m\ s^{-1}$].

5.3.4 The longitudinal hydrodynamic dispersion coefficient D

One of the widely used models for describing solute transport in porous media is the advection-dispersion equation (Equation (5-1)). The method of estimating the dispersion coefficient in Equation (5-1) is described in this section.

For one dimensional porous media, the longitudinal hydrodynamic dispersion coefficient D [$\text{m}^2 \text{s}^{-1}$] can be expressed as the equation below (Chrysikopoulos and Katzourakis, 2015):

$$D = D_m + \mathcal{D}_{bm} \quad (5-24)$$

Where D_m is the mechanical dispersion [$\text{m}^2 \text{s}^{-1}$] (calculation method is described in section 0), $\mathcal{D}_{bm}$ is the effective molecular diffusion [$\text{m}^2 \text{s}^{-1}$] (calculation method is presented in section 5.3.4.1).

5.3.4.1 Effective molecular diffusion $\mathcal{D}_{bm}$

The effective molecular diffusion can be estimated by (Chrysikopoulos and Katzourakis, 2015):

$$\mathcal{D}_{bm} = \frac{\mathcal{D}_{AB}}{\tau} \quad (\tau \geq 1) \quad (5-25)$$

Here τ is tortuosity [-]. For spherical particles, $\mathcal{D}_{AB}$ [$\text{m}^2 \text{s}^{-1}$] can be calculated by the stokes-Einstein diffusion equation:

$$\mathcal{D}_{AB} = \frac{BT}{3\pi\mu d_p} = D_\infty \quad (5-26)$$

Tortuosity (τ) is a fundamental property of porous media, which can describe the interconnectedness of pore spaces. The estimation equation of tortuosity has introduced by Bruggeman (1935):

$$\tau = \theta^{1-m} = \frac{1}{\sqrt{\theta}} \quad (m \approx 1.5) \quad (5-27)$$

5.3.4.2 Mechanical dispersion and dispersivity D_m

The mechanical dispersion can be calculated by (Chrysikopoulos and Katzourakis, 2015):

$$D_m = \alpha_L v^i \quad (5-28)$$

Note that D_m often appears to be almost linearly related to Darcy velocity (i.e. $i \approx 1$).

α_L [cm] is the dispersivity, which is a value usually measured by experiments. The value of α_L affected by the particle travel distance, suspended particle size, velocity and other changing factors. The interaction between different factors makes it very difficult to estimate. However, (Chrysikopoulos and Katzourakis, 2015) summarised an estimation equation based on suspended particle size.

$$\alpha_L \text{ [cm]} = 0.29 + 5.34 \times 10^{-5} d_p \text{ [nm]} \quad (5-29)$$

5.3.5 Model summary

The model flow chart is presented in Figure 5-3. The figure concluded the parameters, equation flow and feedback for the clogging model. The Richards equation velocity field has been altered by the clogging model, further affecting the RIB and hydraulic model.

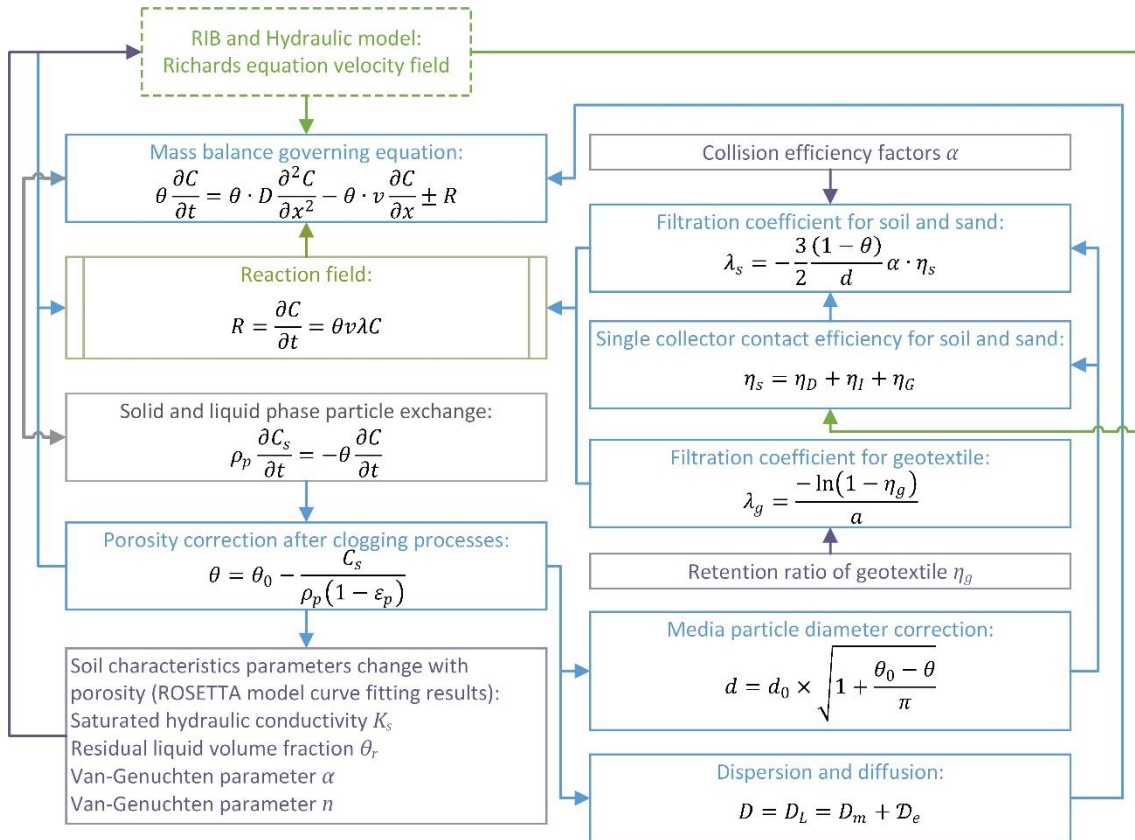


Figure 5-3 Model flow chart for clogging model

6 Chapter Six: Application of the clogging model

6.1 Introduction

This chapter purposed to apply the PGP-HRC model with different input parameters, such as rainfall intensity, suspended particle concentration, suspended particle size and simulation duration for the PGP system. This chapter indicates and discusses the model test results.

The model has been applied to a three-layer geometry with sandy loam, sand and nonwoven geotextile, as described in Section 1.12.1. The cross-section structure of the simulated PGP system is shown in Figure 6-1. Suspended particles carried by urban surface runoff or rainfall fall on the surface of the PGP system (Section 5.1), and the particles infiltrate into the layers following the water flow direction. Particles could accumulate and clog in each layer, and the particles left in water will drain out of the three-layer system.

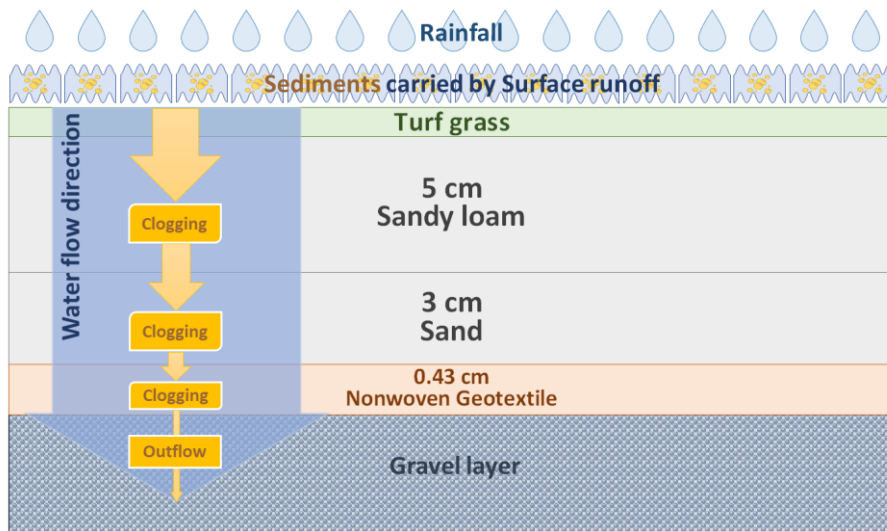


Figure 6-1 Cross-section of three layers structure for simulation of the clogging performance of soil, sand and geotextile. The water flow direction and sediment transport processes are shown in the figure.

Different input parameters have been applied to short-term and long-term simulations. The short-term simulation provides a general understanding of the clogging processes. The long-term simulation further tested more variables (including extreme conditions) and explored the mechanisms of the clogging

processes. The results of liquid volume fraction (LVF), Darcy velocity, and relative suspended particle concentration (RPC) were used to assess the degree of clogging.

COMSOL considered the particle individually as molecular. Therefore, the RPC [1] value was calculated by the equation

$$RPC = \frac{C}{C_{in}/molmass} \quad (6-1)$$

Here C is the suspended particle concentration in the liquid phase [mol/m³] (explained in Section 5.3.3), C_{in} is the inflow suspended particle concentration [kg/m³] (value listed in Table 6-5), and $molmass$ is the suspended particle molar mass [g/mol] (value listed in Table 6-3).

As introduced in Section 5.3.3, liquid-phase (water) particle concentration reduction will enter and remain in the solid-phase (soil, sand and geotextile). Instead of the specific concentration value, the RPC represents the ratio of particle concentration remaining in water to C_{in} . The highest value of RPC is 1, indicating that all the inflow particles remain in the water and clogging has not occurred. When RPC equals 0, no particles remain in the water, all the inflow particles have been intercepted by the media, and clogging is under development.

6.2 Model parameterisation

As mentioned in Section 1.12 and Section 5.1.3, the PGP-HRC model consists of three parts. Three model parts affect each other and create a feedback loop as the clogging evolves. The parameters for the PGP-HRC model are introduced in this section.

6.2.1 Matrix properties for sandy loam, sand and nonwoven geotextile layers

The parameters for sandy loam, sand and nonwoven geotextile are summarised in Table 6-1 and Table 6-2. Notably, the matrix properties show the initial value here. As presented in Section 5.3.3, The values of porosity, media grain size, and

saturated hydraulic conductivity varied as clogging progressed. The mathematical relationship among these variables was presented in section 5.3.3.

The equations for the variables of sandy loam and sand characteristic parameters changes with porosity were described in Table 5-4 (Section 5.3.3.3).

Table 6-1 Input parameters for sandy loam and sand

<i>Parameter</i>	<i>Symbol</i>	<i>Sandy loam</i>	<i>Sand</i>	<i>Unit</i>	<i>Source</i>
The initial saturated liquid volume fraction (porosity)	θ_0	0.39	0.38	-	ROSETTA model (Schaap, Leij and van Genuchten, 2001)
The initial media grain size	d_0	0.05	0.6	mm	-
Thickness	a	50	30	mm	-
Collision efficiency factor	$\alpha_{collision}$	0.047	0.013	-	(Siriwardene, Deletic and Fletcher, 2007b)
Molar mass	molmass	0.1	60.08	g mol ⁻¹	-

Table 6-2 Input parameters for geotextile

<i>Parameter</i>	<i>Symbol</i>	<i>Geotextile</i>	<i>Unit</i>	<i>Source</i>
Thickness	a	4.30	mm	(Ho, 2000)
The initial saturated liquid volume fraction (porosity)	θ_s	0.86	-	
The initial saturated hydraulic conductivity	K_s	1.64×10 ⁻³	m s ⁻¹	
Retention ratio	η_g	0.1	-	(Liu <i>et al.</i> , 2018)

The geotextile is a very thin layer at the bottom of the PGP system, the changes of η_g with porosity will not make a significant difference for a long-term clogging simulation. The testing particle size is 0.01 [μm] (section 6.4.2), much smaller than the tested particle size for the geotextile retention ratio in Liu *et al.* (2018). As presented in Table 5-3, the η_g values reduce as the suspended particle

decreases. Therefore, for simplification and stabilised the calculation, the geotextile retention ratio η_g value was estimated to be 0.1 [-].

6.2.2 Other parameters for the clogging model

Besides the matrix properties, some parameters are required for the clogging model. Table 6-3 concluded the parameters used for the clogging model tests.

Table 6-3 Input parameters of suspended particles and others

<i>Parameter</i>	<i>Symbol</i>	<i>value</i>	<i>Unit</i>	<i>Source</i>
Temperature	T	288	K	-
Hamaker number	Ham	10^{-20}	m s^{-1}	(Tufenkji and Elimelech, 2004)
Boltzmann's constant	B	1.3805×10^{-23}	J K^{-1}	-
The specific weight of water	γ_w	9.8	kN m^{-3}	-
Suspended particle molar mass (clay as representative)	molmass	1.0085	g mol^{-1}	-
Actual porosity blocked by the suspended particles in the porous materials	ε_p	0.85	g m^{-2}	(Liu <i>et al.</i> , 2018)
Suspended particle density	ρ_p	1.431	g cm^{-3}	

6.3 Initial, extreme and boundary conditions

6.3.1 Upper boundary (0 cm geometry)

For the hydraulic section, the rainfall-infiltration boundary condition (described in Chapter 4) has been applied to the upper boundary.

For the clogging model part, the inflow concentration of suspended particles should be specified for the upper boundary. The inflow suspended particle concentration is a constant value $[\text{kg m}^{-3}]$ that does not change with inlet velocity.

Thence, the total input mass of the suspended particles will increase with a larger inlet velocity.

6.3.2 Lower boundary (-8.43 cm geometry)

For the hydraulic section, the lower boundary is a free-drainage condition as described in Section 3.2.2.

For the clogging model part, the lower boundary is a free outflow boundary for suspended particles. The suspended particles should be transported across the boundary by fluid motion, assuming that the diffusion can be ignored (COMSOL, 2018). In this model, the fluid motion is provided by the Darcy velocity of the hydraulic section.

6.3.3 Initial condition

The conclusion of Chapter 3 and Chapter 4 revealed that the initial water content does not significantly affect the hydraulic behaviours of the three-layer PGP system. Therefore, the initial water content for the whole structure has been set to field capacity, with the initial pressure head at -8.43 cm.

6.3.4 Extreme condition

Under extreme conditions, the media is fully clogged, not allowing any water or particles to flow through. Liu *et al.* (2018) suggest that the saturated hydraulic conductivity can be reduced to 10^{-10} [m s⁻¹] for a severe clogged nonwoven geotextile. Affected by extreme clogging, the porosity of a porous medium could decrease to 0.01-0.05 (VanGulck and Rowe, 2004; McIsaac and Rowe, 2005; Liu *et al.*, 2018). To avoid non-convergence during the calculation, the minimum hydraulic conductivity was set to 10^{-10} [m s⁻¹] and the minimum porosity to 0.01.

6.4 Tests design

Short-term and long-term simulations have been designed to test the clogging process of the PGP system. The rainfall data and suspended particle data used for the simulations are representative of UK local data. It is crucial to realise that the time presented in this chapter is model duration instead of the natural

calendar year. Hydraulic model results (Chapter 3) concluded that the dry period could be neglected since it has limited impact. Based on this assumption, the rainfall used in this chapter is constant continuous intensity, representing several years of the actual rainfall in the UK.

6.4.1 Rainfall characteristic of the UK

The rainfall characteristic has been studied by McPherson (2019) and summarised in Figure 6-2. McPherson (2019) collected rainfall intensity data from eight locations across the UK, which is Birmingham, Manchester, Carlisle, Exeter, London, Newcastle, Norwich and York. Limited to the resolution of the rainfall gauge, the smallest rainfall intensity value is 0.8 [mm h⁻¹] (0.2 [mm] in 15 [min]) (McPherson, 2019). The average values among the eight locations have been plotted (Figure 6-2), and curve-fitting has been applied.

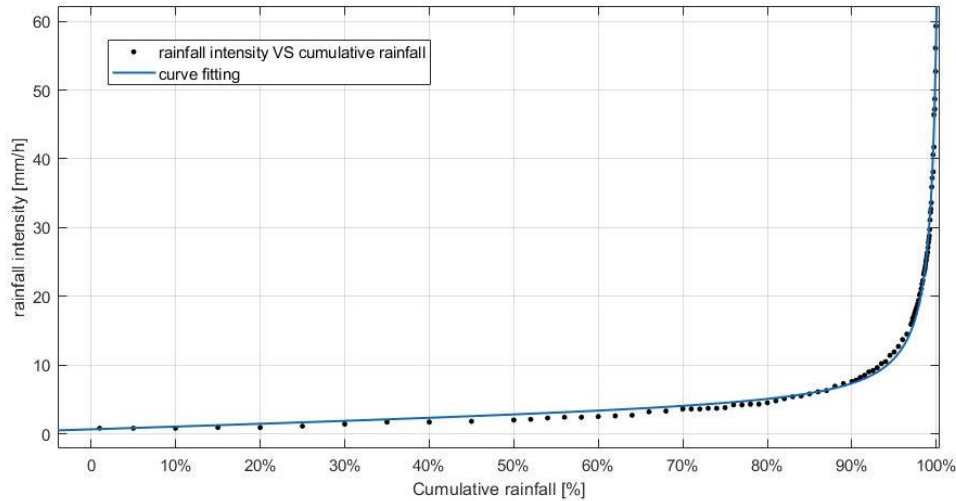


Figure 6-2 Percentage of cumulative rainfall plotted against the rainfall intensity in the UK. (Data source: McPherson, 2019)

$$y = \frac{(-5.491 \times 10^5)x^2 + (5.184 \times 10^5)x + (1.019 \times 10^5)}{x^2 + (-1.568 \times 10^5)x + (1.579 \times 10^5)} \quad (6-2)$$

$$(0 < x < 1, R^2 = 0.996)$$

Here x represents the cumulative rainfall in percentage, and y is the rainfall intensity. The equation (6-2) can be used to estimate the percentage of rain that falls at an intensity less than or equal to a specific value.

According to McPherson (2019), in the UK, 90% of rain falls at an intensity less than 7.2 [mm h⁻¹], and 11% of rainfall intensity is less than 1.08 [mm h⁻¹]. As the representative of the typical UK rainfall condition, the constant rainfall intensity of 1.08 [mm h⁻¹] have been selected for test S1, and 7.3 [mm h⁻¹] for test S2, respectively.

6.4.2 Short-term model tests

The designed tests for short-term simulation are shown in Table 6-4. The letter S in each test name represents the simulation duration (Short-term). The input parameters are inflow particle concentration, inlet rainfall intensity and suspended particle size. These parameters highly depend on the weather scenario and surrounding environment instead of the materials and structure of the PGP system.

The short-term model tests (S1 and S2) duration last 420 days (10,080 hours). 420 days are the minimum duration required for test S1 to be fully clogged (Darcy velocity at inlet point equals 0). The suspended particle inflow concentration is 1.0085×10^{-3} [kg m⁻³], which is a testing value to make sure the input concentration equals 1 [mol m⁻³] (inflow concentration divided by molar mass).

The inflow concentration and suspended particle size are fixed values indicated in Table 6-4.

Table 6-4 Input parameters for short-term simulation tests

No.	<i>Inlet rainfall intensity [mm h⁻¹]</i>	<i>Time [h]</i>	<i>Inflow concentration [kg m⁻³]</i>	<i>Suspended particle size [μm]</i>
S1	1.08	10,080	1.0085×10^{-3}	0.01
S2	7.2	10,080		

6.4.3 Long-term model tests

The designed tests for long-term simulation are summarised in Table 6-5. The letter L in each test name represents the simulation duration (Long-term). The long-term model tests (LR, LP and LC) duration last 3,652.4 days (10 years). The letters R, P and C represented the tested inlet rainfall intensity, suspended

particle size, and inflow concentration. The control exists in every test. The black line on each graph represents the control group in the results.

Table 6-5 Input parameters for long-term simulation tests

No.	Inflow concentration C_{in} [kg m^{-3}]	Inlet rainfall intensity [m/s]	Suspended particle size [μm]
LR	1.0085×10^{-3}	2.85×10^{-5}	0.01
		2×10^{-6}	
		3×10^{-7}	
		5×10^{-8}	
LP	1.0085×10^{-3}	2×10^{-6}	0.01
			0.1
			1.0
			3.0
			5.0
LC	6×10^{-4}	2×10^{-6}	0.01
	1.0085×10^{-3}		
	0.026		
	0.15		
	5.70		
Control	1.0085×10^{-3}	2×10^{-6}	0.01

For test LR, two rainfall intensity values have been chosen in addition to the rainfall selected in Section 6.4.2, representing the rare torrential or minimal rainfall.

As described in Section 5.2.2.2, the corrected R-T model is more accurate with the suspended particle size between 0.01 to 4 μm (Tufenkji and Elimelech, 2004). Thence, the tested suspended particle size for test LP gradually increased from 0.01 to 5 μm to cover a more comprehensive particulate range.

The different suspended particle concentration has been applied to test LC. The particulate matter concentration is hard to confirm since it changes with a wide range (Barlow, 2015). However, the minimum and maximum particle concentration can be estimated by the particulate matter mass (Table 6-6). As

described in Section 1.5, the mass of particulate matter accumulated for 5-continuous days vary from 7.3 [g m⁻²] to 740 [g m⁻²]. The suspended particle concentration for test LC gradually increased from 6×10^{-4} [kg m⁻³] to 5.70 [kg m⁻³] (calculated and highlighted in Table 6-6).

Table 6-6 Estimation of the minimum and maximum value of suspended particle concentration

<i>The total amount of rainfall in 120 hours [m]</i>	<i>The total accumulated mass of particulate matter in 120 hours [g m⁻²]</i>	<i>Suspended particle concentration [kg m⁻³]</i>
12.1824	7.3	6×10^{-4} (minimum)
0.1296	740	5.70 (maximum)
The smallest and largest rainfall intensity in test LR has been used for the calculation	-	

Notably, the total amount of rainfall is a continuous constant inflow, and it is not meant to be actual weather. This is based on the assumption that the dry periods (when it is not raining) do not significantly affect clogging (Chapter 3).

6.5 Results

The clogging model application results for short-term and long-term tests are shown in Figure 6-3, Figure 6-4, Figure 6-5, and Figure 6-6.

6.5.1 Short term simulation

The results of Darcy velocity for short-term simulation tests S1 and S2 are shown in Figure 6-3. Besides the constant rainfall intensity for tests S1 and S2, the Darcy velocity at the boundary between layers and outlet point is plotted against time in Figure 6-3.

For tests S1 and S2, the Darcy velocity at the outlet point and the boundary between layers reduce to zero after a 420 days simulation (Figure 6-3). Compared to test S2, the Darcy velocity result of S1 declines relatively slowly. Outlet Darcy velocity of test S2 decreased to zero at 200 days, and test S1 reached zero velocity at 400 days. Also, the outlet Darcy velocity of test S1 stays

in the highest value (rainfall intensity) for a much more extended period than test S2.

For Figure 6-4 and Figure 6-5, each line illustrates the results changes at a certain geometry depth. At 0 is the sandy loam surface, -5 cm is the boundary between sandy loam and sand, -8 cm is the boundary between sand and nonwoven geotextile, and -8.43 cm is at the bottom of the modelled structure. The results show a dramatic change in the top 0.2 cm depth of geometry. Therefore, more data of the top 0.2 cm is plotted in the figure.

The porosity changing with time for tests S1 and S2 are presented in Figure 6-4. The porosity reduction changing with time for tests S1 and S2 in each structure depth is shown in Figure 6-4. For tests S1 and S2, the porosity shows similar decrease trends with time on the top 0.2 cm soil. The porosity remains unchanged at -2.5, -5 and -8 cm depth.

The porosity decreases rapidly from 0.39 to 0.08 and 0.06 for tests S1 and S2 respectively on the structure's top 0.01 cm. Also, for both tests, the porosity reduction rate decreases as the depth increase. When the simulation stopped at 420 days, the porosity at the soil surface showed a minimal value, and it raised dramatically in the deeper structure.

The RPC in water and SS removal rate changes over time for tests S1 and S2 are shown in Figure 6-5. On the soil surface (0 cm), the RPC stays at 1, which indicates the constant inflow concentration. As clogging progresses, the RPC reduces overtime at depth -0.001, -0.005, -0.01, -0.05, -0.1 and -0.2 cm. The slight increase at time 0 [d] may be caused by the particles travelling to this location and accumulated in the water. The RPC of S2 shows a steeper slope, which means a higher reduction rate. RPC stays unchanged (value is 0) at the depth of -2.5, -5, -8 and -8.43 cm (Figure 6-5).

6.5.2 Long-term simulation

The results of long-term simulation tests LR, LP, and LC are indicated in Figure 6-6. Figure 6-6 presents the liquid volume fraction changes with depth after being simulated for 10 years.

The geometry depth (y-axis of Figure 6-6) has been enlarged to 0 to -2 cm to express the dramatic changes on the surface. The zig-zag lines for test LC are caused by the differential equation calculation method. The finite element calculation method can only control the errors in an acceptable range. Excessive inflow concentration applied on a thin soil layer will cause a sharp decrease in porosity, increasing the calculation uncertainty.

Test results of long-term tests LR, LP, and LC are shown in Figure 6-6. The black line can be seen as a control group, and the parameters are the same for the black line in each figure. In comparison, test LC indicates the most significant liquid volume fraction changes, and test LR shows minuscule differences. The long-term clogging model is most sensitive to the changes in inflow suspended particle concentration. As raising rainfall intensity or suspended particle concentration, the clogging depth increases to around 0.8 cm or 2.5 cm.

Liquid volume fraction (LVF) represents the real-time situation where the soil pores are filled. In each figure of Figure 6-6, LVF will decrease as the structure depth increase, and the smallest value can be found on the soil surface (0 cm on the y-axis). These results are consistent with the previous conclusion (Section 6.6.1.2 and 6.6.1.3) that the clogging started on the soil surface and gradually developed into the depths.

For test LP, the largest and smallest particle sizes tend to be clogged on a thinner surface area (Figure 6-6 LP, black and red line). When the suspended particle size is larger than 1 μm , as the particle size increases, clogging occurs in shallower soils.

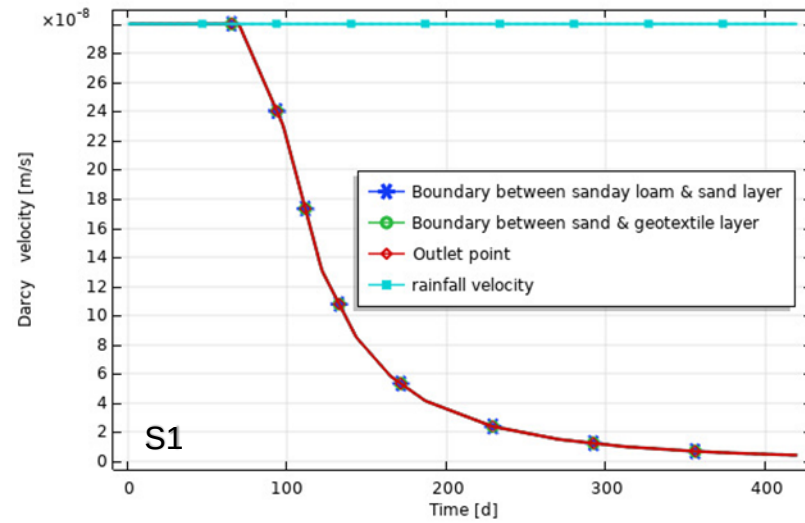
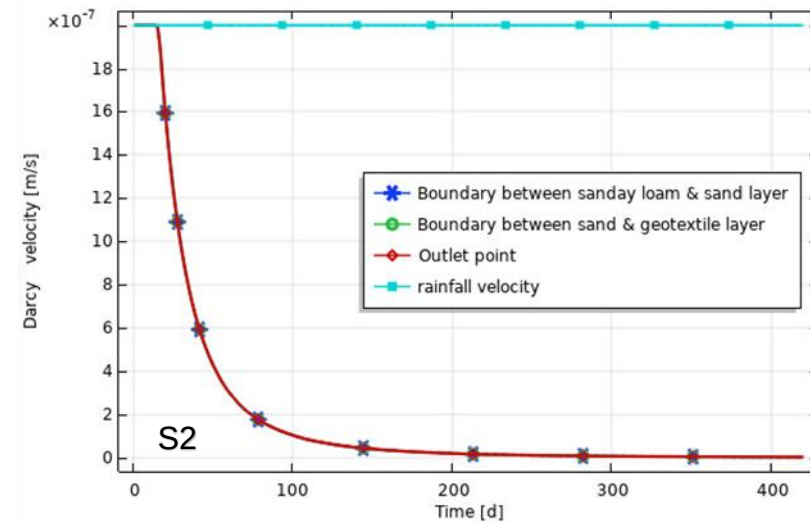


Figure 6-3 Darcy velocity results of short-term simulation tests S1 and S2, the X-axis is time in days. The rainfall intensity for test S1 is 3×10^{-7} [m/s], and test S2 is 2×10^{-6} [m/s].

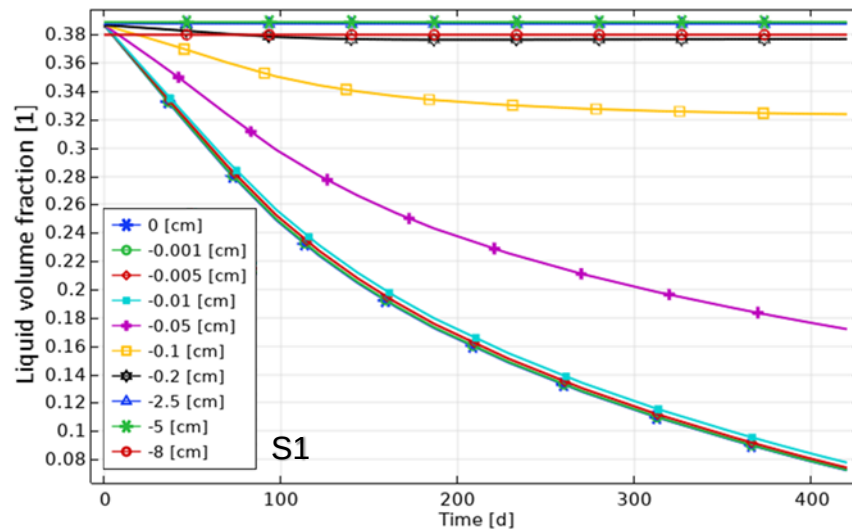
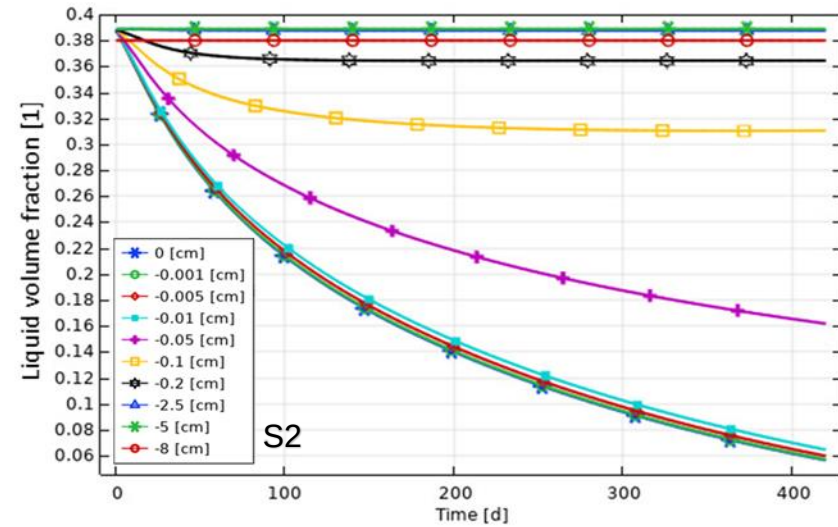


Figure 6-4 Model results of liquid volume fraction (LVF) changes with time for short-term simulation tests S1 and S2.

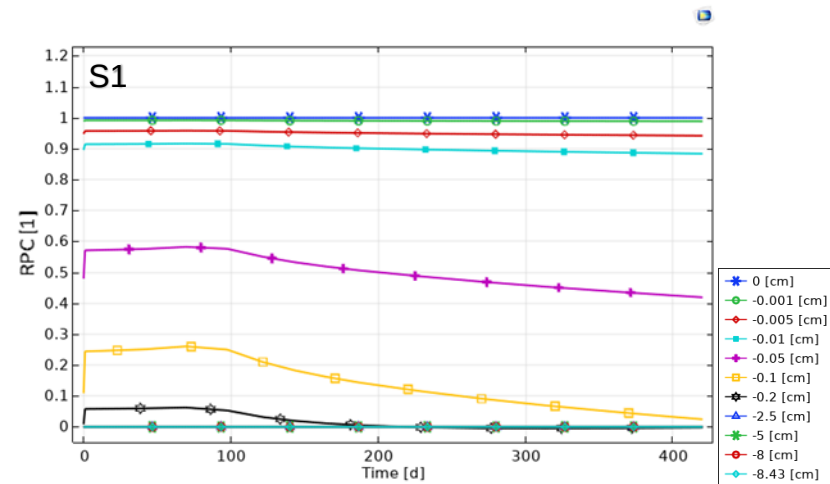
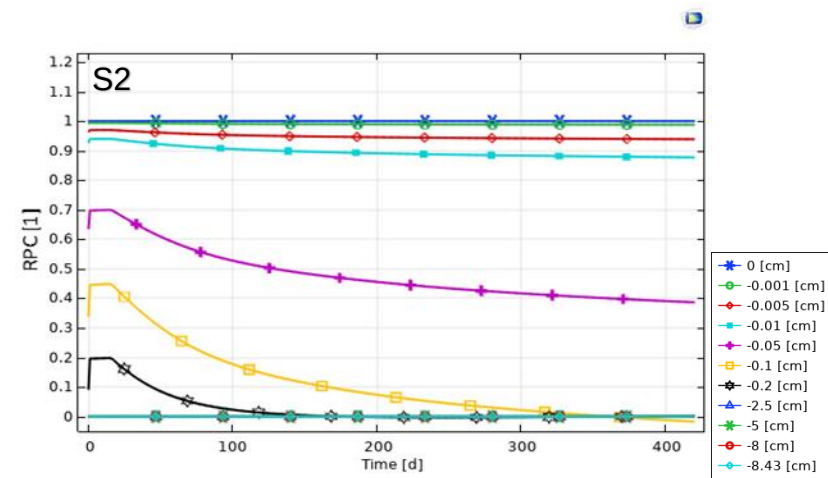


Figure 6-5 Model results of Relative suspended particle concentration (RPC) changes with time for short-term simulation test S1 and S2.

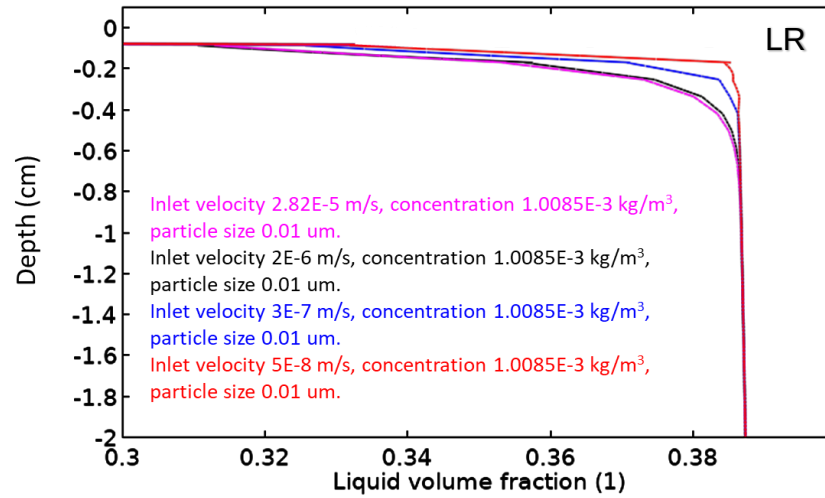
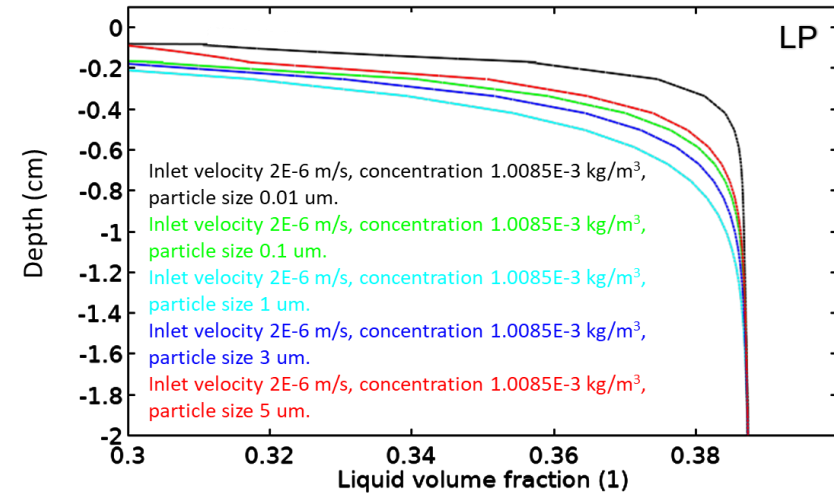
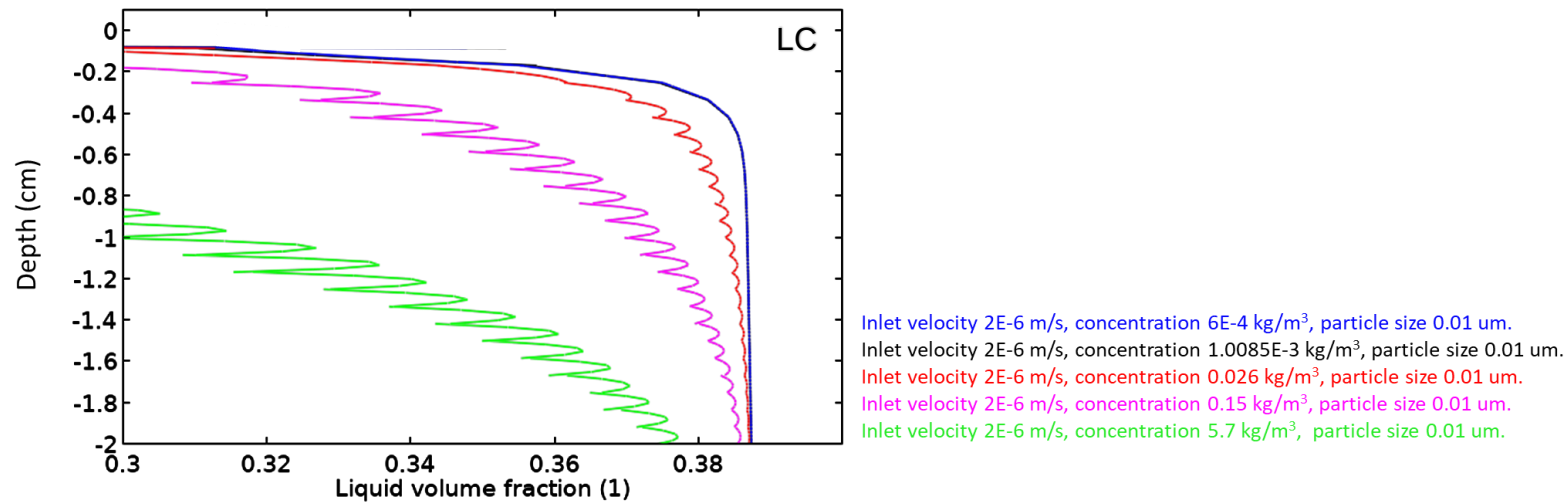


Figure 6-6 Model results of long-term simulation test LR, LP and LC at the 10th year. The y-axis of all the figures denotes depth from the surface (0 cm) to -2 cm, and the x-axis is the liquid volume fraction.

6.6 Discussion

6.6.1 Short-term simulation

6.6.1.1 Darcy velocity changes with clogging development

Darcy velocity value reduces as the clogging develops in the PGP structure. As the clogging develops, porosity will be filled up with suspended particles, and it is more difficult for the water to flow past. Moreover, it appears that the smaller the rainfall intensity, the slower the outlet Darcy velocity decline and the slower the clogging. With the same inflow suspended particle concentration, a larger rainfall intensity carries more particles into the structure and speeds up the clogging processes.

Although the result of Darcy velocity indicates that the PGP structure to lose its infiltration ability within 420 days of model duration, the reality may be different. Tests S1 and S2 use a constant rainfall intensity and inflow particle concentration. After 420 days of stimulation duration, the PGP system is fully clogged. However, natural rainfall intensity and particle concentration change dynamically over time. Also, as presented in Section 6.4.2, most of the rainfall event in the UK is light rain. Therefore, the PGP system will take longer than 420 days to lose infiltration ability in natural environments with the same rainfall intensity and particle concentration level.

Furthermore, the constant inflow of suspended particle concentration used for the simulation also speeds up the clogging process. In the natural urban environment, the particle concentration is not uniformly distributed during a rainfall period. Higher particle concentration levels were measured in the first 5 [mm] rainfall (Ballard *et al.*, 2015; Barlow, 2015).

6.6.1.2 Porosity changes with clogging development

The porosity reduces slower with a smaller rainfall intensity (test S1), and the clogging is also less severe. However, the rainfall intensity is unlikely to affect the clogging depth. For tests S1 and S2, minimal porosity has only been found on the soil surface of 0.01 cm with different rainfall intensities. Deeper than -2.5 cm,

the porosity stays unchanged, indicating that barely any particles travelled to the deeper layer. The suspended particles are hard to reach the sand and geotextile layer.

Surface maintenance can restore the infiltration capacity of the PGP system (Dierkes *et al.*, 2002). Previous research that the clogging concentrated on the upper 50 [mm] pavement structure layer for concrete or aggregates (Baladès, Legret and Madiec, 1995; Haselbach, 2010). At the same time, Hunt (2011) claimed that the clogging depth for a grass reinforcement would be 6-12 mm. The results of S1 and S2 are consistent with the conclusion in the literature that the clogging concentrated pavement surface. However, PGP uses soil as the uppermost layer material, clogging occurs in a thinner layer.

Although the porosity decreases to a very low level (less than 0.1) after 420 days of simulation, this is unlikely in reality in the UK. As presented in Section 6.6.1.1, the rainfall that lasts for 420 days is almost impossible in reality in the UK. With the same inflow suspended particle concentration, the PGP system will remain functional for a longer time.

6.6.1.3 RPC changes with clogging development

When the rainfall intensity is larger, the particles will be intercepted by the media faster, and it takes a shorter time for the structure to be fully clogged.

The RPC always presents the highest value on the soil surface at any simulation time, and the RPC gradually reduces with soil depth. The high concentration of SS will increase the collision probability between SS and media grains, and SS will have more chances to be adsorbed by media grains.

RPC can be used to describe the percentage of suspended particles that remains in the water. Although the suspended particles could result in severe clogging issues to the PGP system, pollutant removal is also one of the main goals for constructing permeable pavement (Ballard *et al.*, 2015). The PGP system removed suspended particles from the water and retained them within the matrix. Section 5.3.3 demonstrated the theory of the transfer of particles between liquid-

phase and solid-phase. The liquid-phase RPC reduction represents the removal, and the solid-phase particle accumulation represents the clogging.

The SS removal rate is generally measured at the outflow point of the structure (-8.43 cm). When RPC equals 1, the SS removal rate is 0%, and the SS removal rate reaches 100% when RPC equals 0. The SS removal rate for S1 and S2 reaches 100% for a 420 days simulation. Notably, after 420 days, the RPC was reduced to 0 at a depth of 0.1 cm for both S1 and S2. The SS has been 100% removed by the PGP system at 0.1 cm. These results are consistent with the conclusion of Section 6.6.1.2 that the particles accumulate within a thin layer on the soil surface. Also, for a short-term operation with the same intensity and particle concentration, the PGP system completely removes the suspended particles.

6.6.2 Ten years of long-term simulation

6.6.2.1 Rainfall intensity and suspended particle concentration affects clogging

The LVF decreases from a deeper layer with higher rainfall intensity. Also, at the same depth, the smaller the rainfall intensity, the higher the LVF. These results indicate that the suspended particles tend to travel to deeper soil with larger rainfall intensity and cause the reduction of LVF.

A similar trend can also be found in test LC (Figure 6-6). With higher inflow concentration, the suspended particles travel deeper into the structure, causing a decline in LVF.

As the trajectory theory explained in Section 5.2.1, the more opportunities for the particles to collide with the media grain, the more likely the suspended particles are to be intercepted by the media grain and transfer from the liquid-phase to the solid-phase. The higher rainfall intensity and inflow particle concentration are two ways to increase the collision probabilities. A high rainfall intensity causes a high Darcy velocity inside the media and encourages a fast movement of the suspended particles. A high inflow concentration can quickly fill up the pores with

particles and dramatically increase the chances of particles colliding with media grains.

Although all the test results indicate that the structure will ultimately lose its infiltration capacity, the literature suggests that the time for a complete clog varies from a few days to several years (Bavor and Schulz, 1993). In reality, the rainfall intensity and inflow particle concentration are event-based instead of a constant value for a long time. The concentration will decrease as the rainfall time lengthens, and the particle concentration is highly related to the surrounding environment and traffic conditions.

A high concentration will result in the particles spreading deep to 2-3 cm, while a low concentration will make the particles stay in the shallower soil. As presented in Section 6.6.1.2, the highest concentration can be found at the start of a rainfall event and gradually decreases as rainfall continues. Combined with the results, the change in concentration in each rainfall event will cause clogging to be distributed in approximately 2 cm of the surface soil. The results agreed with previous research that surface clogging is the main issue that reduces the PGP system infiltration capacity (Welker *et al.*, 2013).

6.6.2.2 Particle size affects clogging

Particle size affects the clogging processes in a way different from flow velocity and particle concentration. The particle size does not seem to directly affect the change of LVF with the structure depth (Figure 6-6). Different particle sizes lead to changes in the inter-force between SS and media grain and the dominant mechanism.

A variety of reasons may cause the clogging depth difference with different particle sizes. When suspended particle size $> 1\mu\text{m}$, the particles tend to accumulate on the shallower surface with a larger particle size. On the contrary, for suspended particle size $< 1\mu\text{m}$, particles accumulate depth decreases with reducing particle size. Previous research suggested that the removal rate increases speedily with particle size for suspended particles larger than $1\mu\text{m}$, yet the removal rate increases with reducing particle size for suspended particles

smaller than $1\mu\text{m}$ (Yao, Habibian and O'Melia, 1971). The clogging depth could be related to the particle removal rate.

$1\mu\text{m}$ is also the demarcation point of the suspended particle transport mechanisms. Different mechanisms can also cause particles to move to different soil depths, affecting the efficiency of collisions and leading to differences in the clogging process. The previous theoretical research on the reasons which cause the differences in suspended particle removal rate changes with particle size have concluded in Table 6-7.

Table 6-7 Summary of previous research on the reason for suspended particle size affecting its transportation trajectory

References	Model prediction theory	
	<i>Particles > $1\mu\text{m}$</i>	<i>Particles < $1\mu\text{m}$</i>
(Smoluchowski, 1917)	Accomplished by velocity gradients or fluid motion	Brownian diffusion ²
(Cookson, 1970; Spielman and Goren, 1970)	Settling and interception	Brownian diffusion ²
(Yao, Habibian and O'Melia, 1971)	Sedimentation and (or) interception	Diffusion
(McDowell-Boyer, Hunt and Sitar, 1986; Newton, 2005)	The diffusion process has a minor effect	Dominated by diffusion
(Galli, 1992; Siriwardene, Deletic and Fletcher, 2007b)	Dominated by sedimentation; Interception has limited significance in coarse media	-

Dispersion and diffusion are only affected by the porosity, tortuosity and suspended particle size. However, sedimentation and interception are dominated by fluid motion. Although the actual phenomenon is still unclear, the clogging situation does not show a continuously regular change with the particle size.

² Brownian diffusion is one type of diffusion that has chaotic and irregular motion of particles immersed in a fluid, caused by their collisions with much smaller surrounding molecules.

It should be noted that most of the suspended particles in water or wastewater are $\leq 1\mu\text{m}$, including virus, bacteria, clay and organic colloid (Yao, Habibian and O'Melia, 1971), although the fine particles may occupy a smaller mass. Therefore, when modelling the suspended particle transport and accumulation in the PGP system, the fine particles should be considered carefully.

6.7 Conclusion

It can be concluded that higher inlet velocity will result in faster clogging. It can also be summarised that clogging reduces porosity and hydraulic conductivity on a thin layer (around 2 cm) of the pavement's surface.

Also, the higher the rainfall intensity and suspended particle concentration, the deeper the clogging develops. Generally, the suspended particle accumulates on the soil surface and move downward. Due to the trajectory method, faster-moving particles can increase the collision opportunities.

Clogging is affected differently by the suspended particle size. For particles smaller than $1\mu\text{m}$, the clogging depth is positively related to the particle size; for particles larger than $1\mu\text{m}$, the clogging depth is negatively related to the particle size. It could be because the mechanism affecting particle transport changes when the particle size equals $1\mu\text{m}$.

7 Chapter Seven: Discussion

The PGP-HRC model has been developed to understand and explore the mechanisms of the clogging process in the PGP system. This chapter purposed to discuss the clogging process in the PGP system in real life and the application of the model results.

7.1 Understanding the influence of individual factors on the behaviour of the PGP system in reality

7.1.1 Rainfall intensity and accumulative rainfall

As presented in Section 1.1.2, the average annual precipitation in the UK for the last 20 years is 1432.3 mm. The years it took to reach the cumulative rainfall in real life can be calculated by dividing the cumulative rainfall in the test by the average annual precipitation. The calculation results are shown in Table 7-1.

Table 7-1 The cumulative rainfall and the equivalent years in reality calculation results for short-term tests S1 and S2 (Section 6.4).

<i>Test</i>	S1	S2
<i>Rainfall intensity [mm h⁻¹]</i>	1.08	7.2
<i>Duration of model [d]</i>	420	420
<i>Cumulative rainfall [m]</i>	10.89	72.58
<i>Equivalent to how many years of rainfall in reality [year]</i>	7.61	50.75

A constant rainfall intensity value has been set for the short-term and long-term tests of the PGH-HRC model. However, rainfall in real life is event-based and constantly changing. The cumulative rainfall for short-term tests S1 and S2 is equivalent to 7.61 and 50.75 years of actual UK cumulative precipitation, respectively.

The constant rainfall intensity was used for variable control and a better understanding of the suspended particle transport in the PGP system. Therefore, the model results can only partly reflect the PGP performance in reality. 420 test days is the time required for test S1 to reach fully clogged (0 [mm h⁻¹] flow rate).

Therefore, with a consistent particle concentration, the actual time required for clogging to be fully developed in the PGP system can be longer than 7 years.

On the other hand, with the same particle concentration, a gradually increased rainfall intensity for the PGP-HRC tests suggested that clogging may happen quicker with higher rainfall intensity. These may be caused by a combination of 1) higher rainfall intensity encouraged particle movement in the water, and 2) more particles have entered the structure. Based on the trajectory method concept, the collision opportunities between the suspended particle and media grain can be increased and accelerate the clogging process.

In test S2, although the test duration of 63 days has reached the cumulative rainfall of test S1, the system is not fully clogged until the 200th test day. Therefore, with rainfall intensity at 7.2 [mm h⁻¹], the PGP system would take 25 years to be fully clogged in real life. This result does not mean that high-intensity rainfall slows down the clogging process. On the contrary, in a high-intensity rainfall event (test S2), the early stage of clogging forms rapidly, the soil porosity and inlet velocity in the soil decline quickly, and the particles that can enter the soil also decreases. Later, when the soil has already clogged, the clogging rate is significantly reduced, and the time for fully clogged is prolonged. When the flow rate drops to 60% of the initial value, test S1 requires 110 test days, while S2 only requires 40 test days. It is crucial to notice that the cumulative rainfall does not equal the inlet water volume since clogging reduces porosity and the inlet velocity, slowing down the clogging process. In reality, rainfall characteristics have a significant impact on the operation of the PGP system. In areas with low and average rainfall intensity, PGP system clogging progresses slowly. However, in areas with high rainfall intensity, PGP system clogging is formed quickly at first, and then the PGP system can be maintained at a low permeability for a relatively long time.

Razzaghmanesh and Beecham (2018) also observed a positive correlation between average annual precipitation and the surface infiltration rate in permeable paving, showing the importance of rainfall characteristics for permeable pavement performance.

7.1.2 Suspended particle size

The PGP-HRC model complied with the trajectory method considering the single suspended particle collision with the media grain. Therefore, compared to the total mass of particulate matter, the fraction and quantity of particle size are more critical. Notably, the collision mechanisms depend on the particle size, causing suspended particles to accumulate in different soil depths (Section 6.6). Extremely fine particles ($< 1 \mu\text{m}$) are dominated by diffusion mechanisms and exhibit different behaviours (Section 6.6.2.2). Although the PGP-HRC model can only accurately simulate suspended particles smaller than $10 \mu\text{m}$, these particles are the most critical pollutants since the fine particles account for a large fraction of urban runoff pollutants. Larger and heavier particles may also affect surface clogging. However, the transportation of large particles highly depends on the water velocity. This study focuses on the suspended particles that travel within the stormwater, most likely to be flushed to the PGP system (footpath or car park pavement).

It is also essential to notice that particle size distribution of the local area has a dramatic impact on the clogging process. Urban particulate pollutants have complex sources and a wide range of particle sizes, which depend on several factors, such as the sediment source, the load of pollutants, and the existence of organic matter (Barlow, 2015). It has been presented by McKenzie *et al.* (2008) that the sedimentation, dissolution, and suspension processes will affect the particulate pollutants' behaviour. It is agreed that finer particles are more critical when dealing with urban pollutants (McKenzie *et al.*, 2008; Thorpe and Harrison, 2008; Mullaney and Lucke, 2014; Barlow, 2015). Fine particles (less than $18 \mu\text{m}$) occupy a large fraction of urban particulate pollution but only contribute a small mass (McKenzie *et al.*, 2008). Westerlund and Viklander (2006) reported that more than 88% of the particles in surface runoff are smaller than $9 \mu\text{m}$ in size. A similar conclusion was also suggested by Li *et al.* (2006) that in a rainfall event, although 90% of the particles sized $< 10 \mu\text{m}$, they accounted for less than 10% of the total mass.

Previous physical clogging model has suggested that the sensitive parameters include (Yong, McCarthy and Deletic, 2013; Radfar and Rockaway, 2016):

- Runoff volume
- Flow rate
- The peak 5 minutes rainfall intensity
- Initial water content
- Cumulative rainfall

In addition to the previous research, the PGP-HRC model results suggest that the physical clogging model is also sensitive to the suspended particle size and fine particle proportion. For the PGP system in real life, the suspended particle size distribution in different areas will lead to differences in soil clogging depth. This means that the PGP system must be customised according to the local environment and runoff pollutants. Because of the complex mechanisms and inter-particle forces, it is difficult to predict the specific depth of the clogging. However, when designing a PGP system, evaluating the clogging process using the PGP-HRC model is still of great value to the construction and maintenance of PGP.

7.1.3 Suspended particle concentration

The suspended particle concentration in real life is not a constant value as in the PGP-HRC model tests. The suspended particle concentration in surface runoff changes depending on the rainfall condition and local particle deposition rate. The suspended particle will only enter the PGP system when rainfall happens. Also, the suspended particle concentration will not stay constant in a single rainfall event because of the First Flush Phenomenon.

The definition of the First Flush Phenomenon is 'The initial period of stormwater runoff during which the concentration of pollutants is substantially higher than during later periods.' (Lee *et al.*, 2002). The First Flush Phenomenon includes two stages, 1) the sediment deposits and accumulates on the surface in dry weather, and 2) the accumulated sediments flushed and carried by a rainfall event (Deletic and Maksimovic, 1998). Furthermore, the First Flush Phenomenon

may vary depending on the location, the length of the previous dry period, and rainfall intensity (Lee *et al.*, 2002; Barlow, 2015). However, some construction standards require that the runoff generated in the first 5 mm of a rainfall event be kept and treated on-site and must not be discharged (Barlow, 2015). Thus, it can be assumed that there are higher levels of pollutants in the runoff of the first 5 mm of rainfall (Ballard *et al.*, 2015).

In summary, due to the uneven distribution of the actual particle concentration in a rainfall event, the number of suspended particles entering the soil is much less than in the PGP-HRC model test. Therefore, it may take longer than 7 years for the PGP system actually to be clogged entirely. Affected by the First Flush Phenomenon and model results (test LC, Section 6.5.2), the large concentration of suspended particles in the first 5 mm rainfall can transport to deeper soil. As the particle concentration changes in a rainfall event, the particles are distributed in soil within the top 20 mm, which could further slow down the occurrence of clogging.

7.1.4 Dry period between rainfall events

The rainfall intensity and inflow suspended particle concentration that has been used for the thesis are constant. A sensitivity analysis has been done with different initial water content (saturated, field capacity and permanent wilting point) to understand the dry period influence on the soil properties. Chapter 3 concluded that the dry initial water content does not significantly affect the PGP system's hydraulic behaviour. Thus the assumption has been made to ignore the dry period for the PGP-HRC model tests.

However, Yong *et al.* (2010) have experimented and suggested that drying affected the longer lifespan of the permeable paving system with asphalt or Formpave surface. The suspended particle may unbind with media grain, and changes the clogging situation. Especially for biology involved processes, the influences of the drying stage would be significant.

The drying period on the PGP pavement is mainly caused by evapotranspiration. Evapotranspiration consists of two main aspects, evaporation (E_0) and

evapotranspiration (E_t) (Shaw, 2010). E_0 represents the direct transfer of water from an open surface to the atmosphere, and it can be assessed with the water capacity of soil and air humidity (Shaw, 2010). E_t includes the wet canopy evaporation and transpiration on the plant surface, and the value can be evaluated by the plant type, available water, and plant transpire ability (Shaw, 2010). E_0 and E_t are crucial processes for the drying stage, removing water from the soil surface and root zone.

Therefore, the involvement of evapotranspiration or evaporation (no plant) can be valuable for simulating the natural matrix drying processes in the future. And also as a supplement of nutrients and metal removal processes.

7.2 Exploring the influence of the combination of multiple factors on the behaviour of the PGP system in reality

7.2.1 The PGP system life span

According to the combined results of Section 7.1, under the conditions of changing rainfall intensity, suspended particle size and concentration, the actual time for the PGP system to be fully clogged could be much longer than 7 years. It is worth noting that, in reality, as clogging occurs, the amount of rainwater and particles that can enter the PGP system decreases, further slowing the clogging process. The initial design of the PGP system could withstand multiple storm attacks. After several years of usage, although clogging develops and the permeability decreases, the PGP is still functional for slight daily rainfall. Therefore, depending on the local surface runoff requirements, a well-designed PGP system can be expected to have a lifetime of 20-25 years, as some manufacturers suggested (Pezzaniti, Beecham and Kandasamy, 2009).

The combination effect of rainfall intensity, particle concentration and particle size makes it difficult to predict the specific value of the PGP lifespan and clogging time. There is no specific number for the permeable pavement lifespan. However, depending on the suspended particle load and the sand grain diameter, clogging problems occur in a wide range between 10 days to 5 years for the sand filter in the wetland system (Bavor and Schulz, 1993).

7.2.2 Clogged soil depth in the PGP system

By the end of the PGP-HRC model long-term tests, the soil surface is seriously clogged. Underneath the surface, the PGP-HRC model results can conclude that the depth of clogging is affected by rainfall intensity, particle concentration and suspended particle size. The impact of these parameters on the clogging development has listed below:

1. Higher rainfall intensity drives the suspended particles to travel into deeper soils, but the suspended particles are still concentrated on the top 8 mm of soil (section 6.6.2.1).
2. Changing the inflow particle concentration has most significantly affected the depth of particle delivery. An extremely large concentration ($5.7 \text{ [kg m}^{-3}\text{]}$) of the suspended particle can distribute in 25 mm deep soil (section 6.6.2.1).
3. Suspended particle size also dramatically influences the particle transport depth in the soil. The clogging depth and particle size positively correlate for particles $< 1 \mu\text{m}$, and negative for particles $> 1 \mu\text{m}$ (section 6.6.2.2). In general, the depth difference can grow to 10 mm.

The PGP-HRC model results also support previous studies that clogging accumulates on the topsoil layer and develop downward. Razzaghmanesh and Borst (2018) confirmed the previous research that suspended particle clogging tends to concentrate on the upper layer. Also, clogging develops from the upper boundary to the lower boundary in a permeable paving system (Razzaghmanesh and Borst, 2018).

The results of Section 6.6.2.1 summarised that the clogging is mainly distributed in the top 20 mm of soil, which confirms the literature's suggestion that removing the upper 20 mm of bedding material could restore the infiltration ability of permeable pavement (Razzaghmanesh and Beecham, 2018). It has been concluded that the clay accumulates on the upper 50 mm layers of permeable concrete during extreme rainfall events (Baladès, Legret and Madiéc, 1995; Haselbach, 2010).

In general, the clogging development of the PGP system accumulated on the pavement's top thin layer and slowly extend down to a particular soil depth (8-25 mm). The clogged soil depth depends on the rainfall intensity, particle concentration and suspended particle size.

The clogging depth is also affecting the geotextile performance in the PGP system. Previous studies suggested that the geotextile used in the permeable paving system could improve the water quality and reduce the infiltration rate (Section 1.3.3). However, the model results of the PGP-HRC model confirmed the previous expectation that the clogging concentrated on the top 60 mm to 200 mm of soil; thus, there is no sign of geotextile clogging. The position of the geotextile inside the structure is crucial. A geotextile at a depth of 8 cm is unlikely to be clogged, and the infiltration ability remains excellent. Overall, the geotextile position in the PGP system structure needs to be considered carefully depending on the infiltration and water quality requirement.

7.2.3 Contribution of PGP system to reducing urban surface runoff

Reducing surface runoff is one of the primary purposes of the PGP system in urban areas. A reduction in infiltration (caused by clogging) means an increase in urban surface runoff while increasing the pressure of urban water treatments. However, the runoff reduction varies with surface type, location and rainfall events. The runoff reduction is generally around 50-100% (Section 1.5.1) for a permeable pavement system. Under a Cranfield 1:30 year 31 minutes design storm test, the runoff reduction is around 70% for a new PGP system (Section 4.7.3).

However, the runoff reduction rate reduces dramatically (Section 4.7.4) as the clogging level increases. The main reason for the rise of runoff is the decrease in porosity and saturated hydraulic conductivity (K_s) caused by the clogging process. For the design of the PGP system, K_s value can help to approximate the hydraulic performance changes with clogging. When the clogging reaches 15%, even the lowest rainfall intensity in the design storm cannot fully infiltrate into the PGP system (Section 4.7.4), which can be regarded as a system failure from a design perspective.

However, the time required for this process is difficult to predict. Suarman, Argue, and Pezzaniti (1999) indicated a 24 years simulation that pavement failure happens when the upper 50 mm surface infiltration rate is reduced to one-third of its initial. Table 7-2 demonstrates the test duration and the actual year (calculated by cumulative rainfall and average annual rainfall at 1.43 m) for the Darcy velocity to reduce to 1/3 of its initial value. With constant rainfall intensity, test S1 could be failed in 2.5 years, and test S2 in 4.6 years.

Table 7-2 The cumulative rainfall and the years it takes in the actual PGP system for the Darcy velocity to reduce to 1/3 of its initial

<i>Test</i>	S1	S2
<i>Rainfall intensity [mm h⁻¹]</i>	1.08	7.2
<i>1/3 of initial Darcy velocity [mm h⁻¹]</i>	0.36	2.4
<i>Duration for the model to reduce to 1/3 of initial velocity [d]</i>	140	38
<i>Cumulative rainfall when the model reduces to 1/3 of initial velocity [m]</i>	3.63	6.57
<i>Equivalent to how many years of rainfall in reality when the model reduces to 1/3 of initial velocity [year]</i>	2.5	4.6

However, it is worth noting that after reducing by one third, the Darcy rate of test S2 is still higher than that of S1, and S2 still has a relatively high permeability. Urban rainfall is not constant, and small rainfall on the test S2 system is unlikely to produce runoff. Therefore, "a reduction of 1/3 infiltration rate means failure of the PGP system" cannot be used as a reference value for all PGP systems.

The acceptable level of surface runoff in the urban areas needed to be determined case by case. The value should be estimated by catchment size, rainfall condition and stormwater return period (Ballard *et al.*, 2015). The estimation surface runoff rate will never be precise since the local runoff is also affected by the larger catchment (Ballard *et al.*, 2015). Although there is a lack of data on the infiltration reduction in the PGP system, a similar result would be expected.

Caused by clogging, the Darcy velocity reduces dramatically on the topsoils, decreasing the infiltration rates and increasing the surface runoff. There is no mandatory regulation of the failure of a permeable paving system. Based on the theory of the rainfall-infiltration boundary condition, the surface runoff could only be generated when the hydraulic conductivity of soil is reduced to a level that smaller than rainfall intensity. The surface runoff situation can be estimated based on the dynamic curve of actual rainfall and hydraulic conductivity reduction, and the PGP-HRC model can help achieve this purpose.

7.2.4 Implication for the maintenance of the PGP system

The pattern of liquid volume fraction changes with rainfall intensity (Section 6.6.2.1) is of great value to the design and maintenance of the PGP system. Although the previous section 7.2.2 indicated that the infiltration ability could be restored by removing topsoil, some maintenance results are temporary and may eventually lead to a complete and irreversible failure of permeable pavement (Razzaghmanesh and Borst, 2018). According to previous studies, a dramatic infiltration reduction can be noticed with voids blockage on the top 50 mm of PICP, even though the underlying layer remains a high hydraulic potential (Berry, 1995). A maintenance method such as high-pressure washing for the PGP system would restore the infiltration capacity temporarily. However, the high inlet intensity of high-pressure water applied on the surface will cause suspended particles to enter deeper soil layers, resulting in clogging. Thus, high-pressure washing may not be a long-term effective method for maintaining the PGP system.

Some turf-grass roots in shallow soil could be an alternative method to increase soil porosity and hydraulic conductivity. It has been suggested by Lu *et al.* (2020) that the plants' roots have significant effects to soil hydraulic properties. Lu *et al.* (2020) have reviewed previous research related to soil hydraulic properties changing in root zones and summarised them as follows:

- *Low-density fine roots (root diameter < 1 mm)*: It is likely to clog soil pores, resulting in a reduction of porosity (θ_s) and saturated hydraulic conductivity (K_s).

- *High-density fine roots (root diameter < 1 mm)*: This type of roots can fill up the macropores in soil and increase pore volume, causing a reduction in K_s and an increase in θ_s .
- *Coarse roots (root diameter > 2 mm)*: Small-scale compaction, particle re-orientation, and macropore development can be caused by coarse root growth. This then leads to a rise of θ_s and K_s .

Root growth, decay and distribution patterns are related to numerous factors, such as plant adaptation to local environments, plant health, solar radiation, water supplies, nutrient availability, diseases and pests (Lu *et al.*, 2020). The complexity of the influence factors makes it difficult to simulate the soil hydraulic properties in a single hydrodynamic model. However, since the PGP-HRC model is a theoretical model with a clear structure, it is feasible to consider the effects of plant roots.

7.3 The implication of the PGP-HRC model for other pollutant removal process

The PGP-HRC model has modelled the suspended particle transport mechanisms in the PGP system. Other pollutants must also be removed from urban surface runoff before discharge to groundwater by the PGP system. The possibility of applying the PGP-HRC model for other pollutant removal processes has been discussed in this section.

7.3.1 Estimate the removal of pollutants in the PGP system affected by similar mechanisms as suspended particle

Any particles smaller than 2 μm will be categorised as dissolved particles. Any dissolved hydrocarbon droplets can be assumed as a particle with tiny particle sizes. Since the PGP-HRC model covers a wide range of particles from 0.01 μm to 10 μm (Table 5-1, Section 5.2.2.2), the hydrocarbon filtration process could be estimated by the PGP-HRC model. However, it is difficult to estimate the accurate removal rate of hydrocarbon droplets due to the lack of data.

It is crucial to notice a decrease in the concentration of heavy metals (such as zinc) with a higher suspended particle removal rate (Section 1.5). Also, the

primary mechanisms for metal removal are sedimentation and filtration (Section 1.5). The mechanisms of sedimentation and filtration have already been included in the PGP-HRC model. Heavy metals tend to attach to suspended particles; as suspended particles cumulate in the solid-phase, the pollutants can also be removed from the water.

It can be justified that by simulating the suspended particle transport and removal mechanisms, the heavy metal filtration rate can also be estimated by the PGP-HRC model. The model has simulated the transport of suspended particles sized 0.01, 0.1, 1, 3, and 5 μm , almost all the particles entering the PGP system can be intercepted, which means around 40-70% of the metals attached to these suspended particles can be removed.

Studies have suggested that metal concentration distribution may be related to particle size (Barlow, 2015). Most heavy metal contaminants tend to attach to sediment with a smaller particle size between 0.1-0.3 μm (Mullaney and Lucke, 2014); others dissolve in inflow water and maybe participate in some chemical reaction. McKenzie *et al.* (2008) suggested that metal is concentrated in the suspended particle with fine size (0.1-1.5 μm). Also, the metal concentration rises as the particle size reduces. However, Herngren, Goonetilleke and Ayoko (2006) indicated slightly different results of the metal concentration relationship with sediment particle size. Herngren, Goonetilleke and Ayoko (2006) reported that metals are enriched in a certain sediment particle size fraction. On all the metals attached to the sediment particles, 45.26% of zinc is distributed in the particles with their size smaller than 0.45 μm (Herngren, Goonetilleke and Ayoko, 2006). Chromium (42.86%) and lead (51.52%) tend to attach to sediment particles sized between 0.45 μm and 75 μm (Herngren, Goonetilleke and Ayoko, 2006). The sediment particle size fraction of 0.45-75 μm indicates the highest average metal loading (Herngren, Goonetilleke and Ayoko, 2006).

The metal concentration in urban areas is highly related to the surrounding areas where the sample was taken (Herngren, Goonetilleke and Ayoko, 2006). Previous research cannot consistently justify the particle size range that metals are more likely to attach. Although these fine particles' relative mass contribution

is small, the potentially toxic metals enrichment and high liquidity make them closely related to water quality (Herngren, Goonetilleke and Ayoko, 2006; McKenzie *et al.*, 2008).

Many pollutants tend to attach to the sediment particles, such as nutrients, metals and metalloids, which increase the potential to estimate their removal rate using the PGP-HRC model. A large amount of fine particles means a larger surface area to absorb pollutants (Barlow, 2015). Previous studies have found that higher pollutant concentrations can be noticed in fine-sized particles (Herngren, Goonetilleke and Ayoko, 2006; McKenzie *et al.*, 2008; Mullaney and Lucke, 2014). However, the particles that nutrients and metalloids preferred to attach have not been determined, which has increased the difficulty of simulating the nutrient and metalloid sedimentation process.

7.3.2 Further development of the PGP-HRC model for other pollutants removal estimation

The current PGP-HRC model will not be sufficient for establishing all the mechanisms mentioned in Table 1-8 (Section 1.5). The chemical reaction mechanisms (precipitation, nitrification, photolysis) and biology involved mechanisms (uptake by plants, biodegradation) can be considered for future research. However, chemical and biological mechanisms can be evaluated by simply modifying the reaction field (Section 5.3) of the PGP-HRC model.

Pollutants removal mechanisms are complex in water, especially when microbial, plant and chemical reactions are involved. The removal mechanisms of pollutants have been studied by Wilson, Bray and Cooper (2004) and summarised by Woods-Ballard *et al.* (2007) (Section 1.5). Other literature also confirmed the removal mechanisms of nutrients (phosphorous and nitrogen), including physical retention, chemical sorption and biological activities (Kuba *et al.*, 1993; Wiesmann, 1994; Li *et al.*, 2017).

On the other hand, reconsidering the filtration coefficient equation will be required for the adsorption mechanism (for hydrocarbon, metals and metalloids). Adsorption is a process that the pollutants directly contact the filter media grain.

It is similar to the trajectory method; however, the ionic state pollutants may bind with media grain by a different force or chemical bond. Nevertheless, the corrected R-T equation used for the PGP-HRC model is sufficient to describe most physical and chemical forces. The PGP-HRC model could estimate the adsorption by changing the inflow concentration unit to $[\text{mol m}^{-3}]$ and re-evaluating the new dominant force and parameters.

The pollutants removal of nutrients, hydrocarbon, metal, and metalloids is likely to result from the collaboration of several mechanisms. The dominant mechanism could differ depending on the matrix materials, fluid properties, and other environmental factors. Instead of considering the mechanisms one by one, a simplified method can be established by creating a relationship function between the pollutant reduction with influence factors (e.g. soil properties, liquid pH value or concentration). The new function can be placed in the reaction field (Equation (5-1)).

Furthermore, some pollutants removal mechanisms require the participation of more than one pollutant. Precipitation required a chemical reaction of cation and anion between pollutants, and biodegradation involved microbial participation.

Several articles found that petroleum hydrocarbon retention performance is significantly affected by bio-degradation processes. The hydrocarbon is mainly biodegraded by bacteria, yeast, and fungi (Das and Chandran, 2011). Das and Chandran (2011) also suggested that the nutrients (nitrogen, phosphorus and iron) will affect the rate of degradation of hydrocarbon. Although the external nutrients will promote the biodegradation rate in the soil, after nine years of degradation, experiments with both conditions (with or without external nutrients) reached the same result (Polyak *et al.*, 2018).

Precipitation and biodegradation can be the dominant removal mechanisms for some pollutants. However, their interaction in the soil has not yet been assessed. As described in Section 1.5, hydrocarbon shows a high removal rate after a few years of biodegradation. For a long-term simulation, the specific instantaneous removal rate can be considered to be neglected.

The pollutant removal process may also accelerate the clogging process, such as bio-clogging. It is also feasible to apply the framework of the PGP-HRC model to other clogging processes. Zhong, Wu and Xu (2013) studied the relationship of hydraulic conductivity, porosity, and hydrodynamic dispersion changes with bio-clogging. As bio-clogging develops, aerobic and anaerobic microorganisms reproduce quickly at different clogging stages and depths (Zhong, Wu and Xu, 2013). Hydraulic conductivity and hydrodynamic dispersion have reduced to 13.7% and 44.25% of their original value, and the porosity also shows a dramatic decline (Zhong, Wu and Xu, 2013). According to the experimental data, Zhong, Wu and Xu (2013) summarised the mathematical relationship for hydraulic conductivity, porosity and hydrodynamic dispersion during bio-clogging.

Although the correlation is still unclear, including the dynamic equilibrium between biodegradation and bio-clogging in the clogging model would be valuable. Increased nutrients and organic matter pollutants can enhance the biodegradation process, but it may also encourage microbial overproduction and results in bio-clogging.

When considering introducing plants' influence into the PGP-HRC model, it is necessary to modify both the hydraulic model and the reaction field of the clogging model. The evapotranspiration process not only dries the soil (Section 7.1.4) but is also a vital driving force for plant water and nutrient uptake. Pollutant uptake by plants is a critical mechanism for removing nutrients (phosphorous and nitrogen) from water (Wilson, Bray and Cooper, 2004; Ballard *et al.*, 2015). Some specific plant types can also accumulate heavy metals from the soil or provide a suitable environment for metal sulphides in the root zone (Wilson, Bray and Cooper, 2004; Ballard *et al.*, 2015). However, this process could only be considered when plants have been involved in the system. In some nutrients affluent areas, plants are necessary to be included in the PGP system.

7.4 Apply the PGP-HRC model as a supplement to the experiment

Compared to laboratory experiments, the model parameters and test time can be easily adjusted for different tests. The theory-based PGP-HRC model can make up for the shortcomings of lab or field experiments as listed:

- The inflow parameters (e.g. rainfall intensity, suspended particle concentration, particle size) cannot be easily changed for lab and field experiments.
- It is difficult to maintain an experiment until the system has been fully clogged (at least 7 years).
- The influences of the surrounding environments (e.g. temperature, experiment equipment, air humidity) are complex to be precluded.

Therefore, the PGP-HRC model can be a valuable tool to complement lab or field experiments to explore the mechanisms behind the hydraulic and clogging behaviours. Before designing complex and time-consuming experiments, the PGP-HRC model can provide a general idea of parameter design, sensitivity analysis, external environmental factors, mechanisms and assumptions.

7.5 Limitation

The PGP-HRC model is a theory-based modelling method which purely depends on the physical mechanisms. Although it is a helpful tool for exploring the mechanisms for long-term evaluation, the model has some limitations. The model results lack data validation, which affects our estimation of accuracy. However, no freely available long-term monitoring exists that could be used to validate the model's long-term hydraulic and clogging performance. In its current form, the PGP-HRC model provides a general estimation of the influence of factors on the clogging process.

8 Chapter Eight: Conclusion

This project investigated the mechanisms of the clogging process in a PGP system through the development of a new theoretical suspended particle transport model (PGP-HRC model). The purpose of the PGP-HRC model was to represent physical correlations that considered feedback between hydraulics, rainfall-infiltration boundary, and clogging in the PGP system. The primary modelling tool is COMSOL Multiphysics.

Although models have been developed and applied to analogous systems (e.g. gravel filters and landfill leachates) (Martin *et al.*, 1996; Siriwardene, Deletic and Fletcher, 2007b; Yu and Rowe, 2012; Kim *et al.*, 2015), this study is novel in that it has: 1) represented the multiple layers of PGP system, 2) incorporated clogging mechanisms, 3) demonstrated clear feedback between hydraulic and clogging profile, and 4) included detailed diffusion and dispersion calculation that changes as clogging evolves in the model. Specifically, the new PGP-HRC model incorporates the following properties, processes and model parts that influence PGP performance:

- Different water flow governing equations were used for soil/sand (Richards equation) and geotextile (Darcy law) to account for their different hydraulic behaviours (Chapter 2).
- The unique geotextile hydraulic behaviours were considered in the hydraulic model, and a different equation was used (Chapter 2). Instead of experimental measurement, the hydraulic parameters for geotextile were calculated by thickness, confined pressure and geotextile permittivity (industrial standard measurement) in the PGP-HRC model.
- The rainfall-runoff relationship was not considered in other models, which is vital because ponding does not apply to PGP systems. The PGP is usually used in low traffic transportation areas, and ponding water is unacceptable. Therefore, a rainfall-infiltration boundary condition has been applied to the PGP-HRC model to simulate urban areas' infiltration and surface runoff relationship (Chapter 4).

- The closed-loop feedback between the hydraulic model, rainfall-infiltration boundary and clogging model for suspended particle transport in porous media is the most important new development of the PGP-HRC model (Chapter 5).
- The PGP-HRC model integrated the ROSETTA model result to estimate the changes in soil properties (saturated hydraulic conductivity, residual liquid volume fraction and van-Genuchten parameters) with decreasing porosity during the clogging process (Chapter 5). This addition is a responsive correlation between clogging and hydraulic properties instead of the semi-empirical methods for estimating the soil properties for a specific media type used in previous models.
- The corrected R-T model has been applied to estimate the PGP-HRC model's single collector contact efficiency (Chapter 5). Compared to the previous version, the corrected R-T model (Tufenkji and Elimelech, 2004) provides a comprehensive solution by considering all the collision mechanisms and interactions.
- It has been indicated in Chapter 6 that diffusion and dispersion are dominant mechanisms for the suspended particles with small size suspended particles (less than 1 μm). Due to the small size of some of the suspended particle fractions, diffusion and dispersion were added to the PGP-HRC model.

The PGP-HRC model was tested with external influence factors (initial soil water content, rainfall intensity, suspended particle concentration, particle size) and duration to understand the clogging mechanisms and parameter sensitivity (Chapter 6). Then the clogging process and the general hydraulic and water quality performance of the PGP system have been discussed. In general, without clogging, the PGP system shows excellent permeability. Clogging can seriously affect the operation of the PGP system. The specific clogging speed, depth, and fully clogged time are influenced by various environmental, rainfall, and contaminant factors.

Geotextile performance in the PGP system has not been clearly described in previous research. The geotextile performance was tested with the PGP-HRC model with different rainfall intensities, initial soil water content and duration (Chapter 3). Without the clogging process, the geotextile shows an excellent drainage ability in the PGP system. The modelling results indicated that clogging of the geotextile is unlikely to be a significant issue because the clogging is concentrated in the top 2 cm of soil, and few particles are transported to the depth of the geotextile (8 cm).

Previous research suggested that rainfall and suspended particle characteristics can significantly affect the clogging process. However, the mechanism and degree of impact have not been clearly stated. The PGP-HRC model was tested with different rainfall intensities and suspended particle concentrations (Chapter 6). Findings from the modelling work indicate that intense rainfall could accelerate the clogging process and encourage the clogging to develop deeper into the soil. Similarly, a greater depth of clogging was also observed when sediment concentrations were higher. These could be caused by the increasing collision opportunities between the suspended particles and the media grain and rising transport efficiency of high rainfall intensity and large particle concentration.

Tests on the particle size distribution of the suspended particles indicate that particle size significantly affects the clogging depth (Chapter 6). The mechanisms that affect the transport of fine particles and coarse particles are different. Fine particles are mainly affected by diffusion and dispersion, while coarse particles are mainly affected by advection. Fine particles ($< 1 \mu\text{m}$) transport deeper with increasing particle size; however, coarse particles ($> 1 \mu\text{m}$) transport deeper with decreasing particle size.

The model could be modified and applied to other porous media systems, such as other types of permeable paving systems, landfill leachate collection, aquifer pollutant transmission, and sand/soil column systems with geotextile. The PGP-HRC model's feedback between hydraulics, particle transport, and clogging allows an advanced understanding of these systems.

The PGP-HRC model can also be used to estimate the pollutants removal rate of the PGP system. Heavy metals attach to the sediments and are retained together with particles. The model could also be modified and applied for pollutants retention estimation (nutrients, hydrocarbons and metalloids). The reaction field needed to be re-evaluated before being used to estimate pollutant retention.

As climate change, urbanisation and permeable pavement ages, the localised design and maintenance requirements become more intense. The theoretical PGP-HRC model is a platform to conduct further research on the PGP pavement to inform the design of systems that increase the longevity and effectiveness of systems in reducing urban surface runoff. The PRP-HRC model helps understand how clogging can lead to decreased performance or failure of PGP systems. The PGP-HRC model would be a time, labour, and cost-saving method to explore the PGP system's changes in different environments.

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APPENDICES

Appendix A Rainfall

A.1 Rainfall intensity values for model tests (every 5 minutes)

Rainfall event 1 (R1)		Rainfall event Cranfield (R2)			
Time (min)	Intensity (m/s)	Time (min)	Intensity (m/s)	Time (min)	Intensity (m/s)
0	0	0	3.33E-08	235	0
5	0	5	2.17E-06	240	3.33E-08
10	0	10	1.39E-05	245	0
15	0	15	1.63E-05	250	6.67E-08
20	0	20	1.81E-05	255	3.33E-08
25	0	25	1.35E-05	260	2.67E-07
30	0	30	1.58E-05	265	6E-07
35	0	35	2.82E-05	270	5.67E-07
40	0	40	1.35E-05	275	3.67E-07
45	0	45	1.14E-05	280	4E-07
50	0	50	8.53E-06	285	3E-07
55	0	55	4.77E-06	290	2E-07
60	0	60	1.5E-06	295	4E-07
65	6.68E-05	65	2.03E-06	300	5.33E-07
70	0	70	1.23E-06	305	5.67E-07
75	0	75	1.37E-06	310	6E-07
80	0	80	6.67E-07	315	0.000001
85	0	85	2.07E-06	320	6E-07
90	0	90	1.77E-06	325	3.67E-07
95	0	95	0.000001	330	1.67E-07
100	0	100	7E-07	335	6.67E-08
105	0	105	3.33E-07	340	1.67E-07
110	0	110	4E-07	345	6.67E-08
115	0	115	5.67E-07	350	0
120	0	120	9.33E-07	355	0
		125	1.27E-06	360	0
		130	1.3E-06	365	0
		135	1.87E-06	370	0
		140	1.83E-06	375	0
		145	1.8E-06	380	0
		150	9E-07	385	0
		155	1.73E-06	390	0
		160	1.27E-06	395	0
		165	1.4E-06	400	0
		170	7.67E-07	405	0
		175	6.67E-08	410	0
		180	9.67E-07	415	0
		185	1.07E-06	420	0
		190	2.33E-07	425	0
		195	6.67E-08	430	0
		200	1E-07	435	0
		205	6.67E-08	440	0
		210	1E-07	445	0
		215	6.67E-08	450	0
		220	1.33E-07	455	0
		225	2E-07	460	0
		230	6.67E-08	465	0

Table A-1 Rainfall intensity for Rainfall event 1 and Rainfall event Cranfield for model testing.

A.2 Rainfall intensity figures as model input

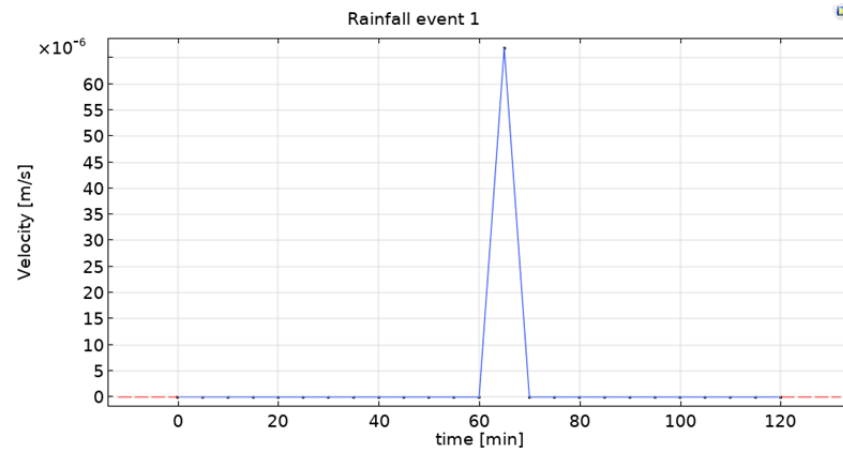


Figure A-1 Rainfall velocity of Rainfall event 1 (R1)

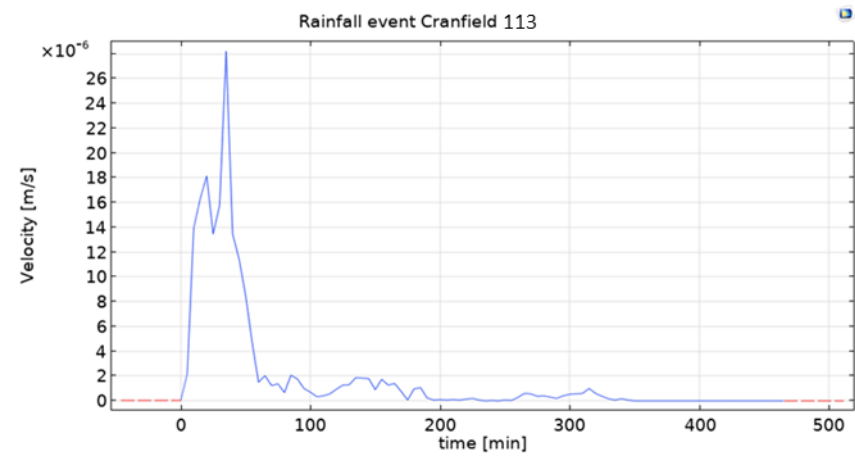


Figure A-2 Rainfall velocity of Rainfall event Cranfield (R2)

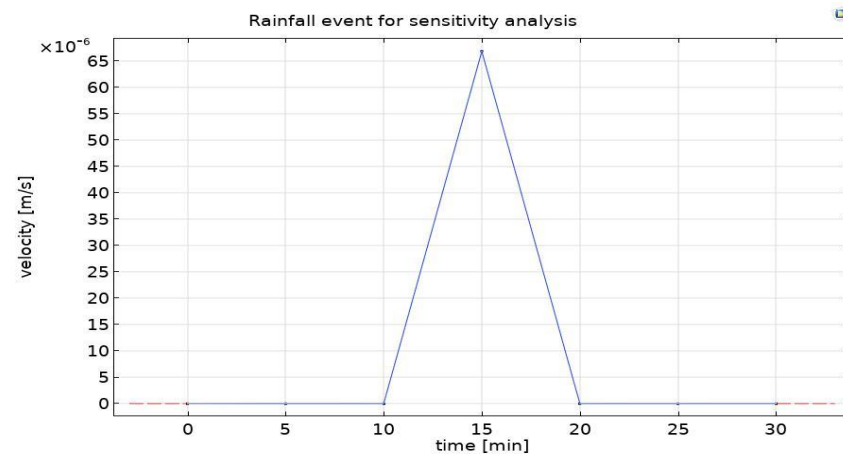


Figure A-3 Rainfall velocity of sensitivity analysis

Appendix B Rainfall and infiltration data for DST

Table B-1 Table of rainfall intensity values for 1:30 year return period 31 minutes design storm (light blue column), and infiltration at the bottom of the 2-cm clogging sandy loam layer (orange column) data.

[min]	[mm min ⁻¹]	[mm min ⁻¹]		
time	Design storm	0% clogging	5% clogging	10% clogging
0	0.0623	0.0623	0.0623	0.0623
1	0.0749	0.0647	0.0638	0.0633
2	0.0900	0.0729	0.0704	0.0689
3	0.1081	0.0860	0.0820	0.0797
4	0.1298	0.1037	0.0986	0.0962
5	0.1556	0.1260	0.1205	0.1128
6	0.1865	0.1541	0.1492	0.1205
7	0.2234	0.1882	0.1769	0.1229
8	0.2673	0.2297	0.1888	0.1238
9	0.3193	0.2732	0.1912	0.1239
10	0.3810	0.2927	0.1913	0.1240
11	0.4536	0.2953	0.1913	0.1240
12	0.5385	0.2951	0.1912	0.1240
13	0.6362	0.2950	0.1912	0.1240
14	0.7435	0.2949	0.1912	0.1240
15	0.8092	0.2949	0.1912	0.1240
16	0.7435	0.2949	0.1912	0.1240
17	0.6362	0.2949	0.1911	0.1240
18	0.5385	0.2949	0.1911	0.1240
19	0.4536	0.2949	0.1911	0.1240
20	0.3810	0.2948	0.1911	0.1240
21	0.3193	0.2946	0.1911	0.1240
22	0.2673	0.2864	0.1911	0.1240
23	0.2234	0.2525	0.1911	0.1240
24	0.1865	0.2168	0.1908	0.1240
25	0.1556	0.1855	0.1782	0.1240
26	0.1298	0.1580	0.1563	0.1240
27	0.1081	0.1340	0.1356	0.1214
28	0.0900	0.1134	0.1167	0.1098
29	0.0749	0.0959	0.1008	0.0971
30	0.0623	0.0807	0.0856	0.0850
31	0.0000	0.0228	0.0323	0.0401
32	0.0000	0.0104	0.0190	0.0269
33	0.0000	0.0041	0.0114	0.0189
34	0.0000	0.0014	0.0071	0.0136
35	0.0000	0.0002	0.0046	0.0101
36	0.0000	0.0002	0.0028	0.0077
37	0.0000	0.0002	0.0017	0.0057
38	0.0000	0.0001	0.0009	0.0041
39	0.0000	0.0001	0.0004	0.0028
40	0.0000	0.0001	0.0000	0.0020
41	0.0000	0.0000	0.0000	0.0014
42	0.0000	0.0000	0.0000	0.0010
43	0.0000	0.0000	0.0000	0.0007
44	0.0000	0.0000	0.0000	0.0005
45	0.0000	0.0000	0.0000	0.0003
46	0.0000	0.0000	0.0000	0.0002
47	0.0000	0.0000	0.0000	0.0001
48	0.0000	0.0000	0.0000	0.0000
49	0.0000	0.0000	0.0000	0.0000
50	0.0000	0.0000	0.0000	0.0000
51	0.0000	0.0000	0.0000	0.0000
52	0.0000	0.0000	0.0000	0.0000
53	0.0000	0.0000	0.0000	0.0000
54	0.0000	0.0000	0.0000	0.0000
55	0.0000	0.0000	0.0000	0.0000
56	0.0000	0.0000	0.0000	0.0000
57	0.0000	0.0000	0.0000	0.0000
58	0.0000	0.0000	0.0000	0.0000
59	0.0000	0.0000	0.0000	0.0000
60	0.0000	0.0000	0.0000	0.0000

Appendix C Rainfall events data from the Cranfield and the Swindon weather station

C.1 Rainfall events data from the Cranfield weather station

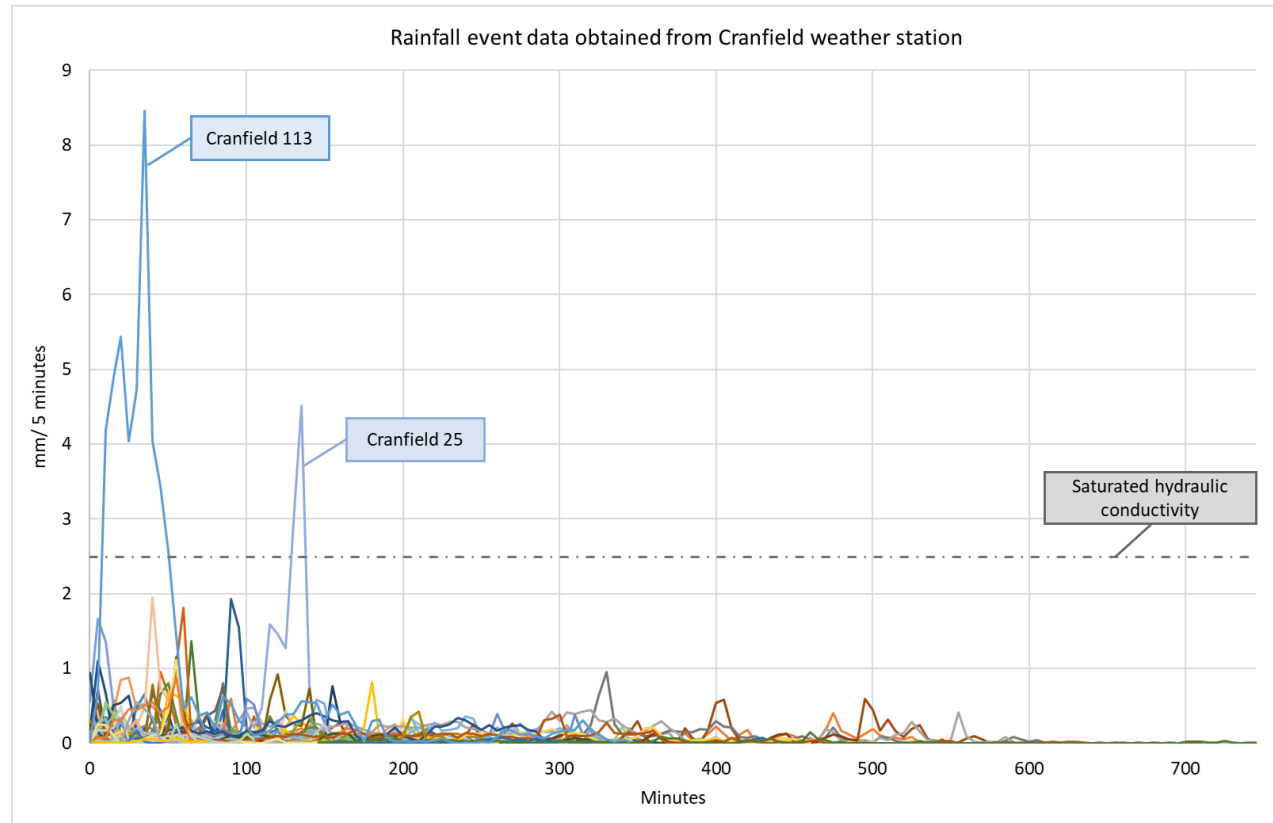


Figure C-1 113 rainfall events data from the Cranfield weather station. Cranfield 113 and Cranfield 25 are the two heaviest rainfall events. The loamy sand saturated hydraulic conductivity is also shown in the figure.

C.2 Rainfall events data from the Swindon weather station

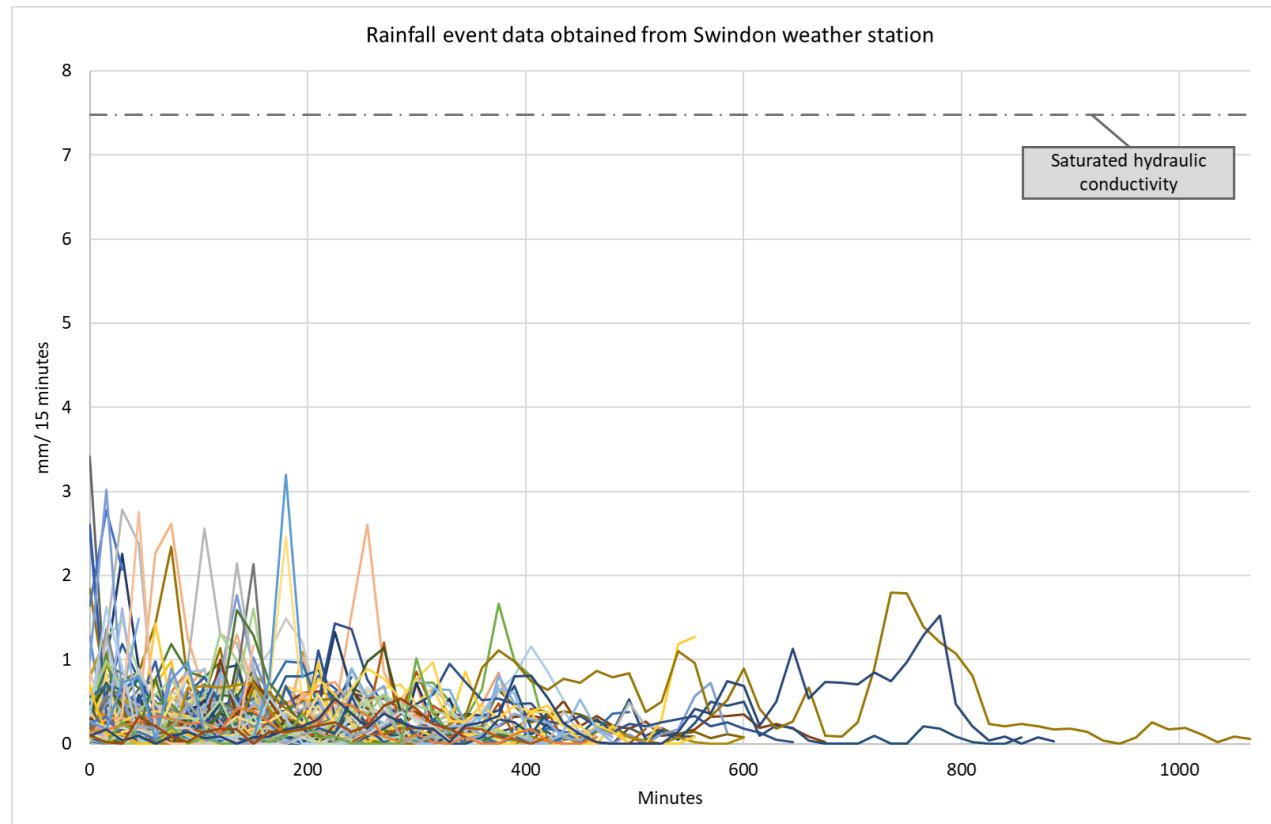


Figure C-2 170 rainfall events data from the Cranfield weather station. The loamy sand saturated hydraulic conductivity is also shown in the figure.

C.3 Ten rainfall events were selected for RIB model tests

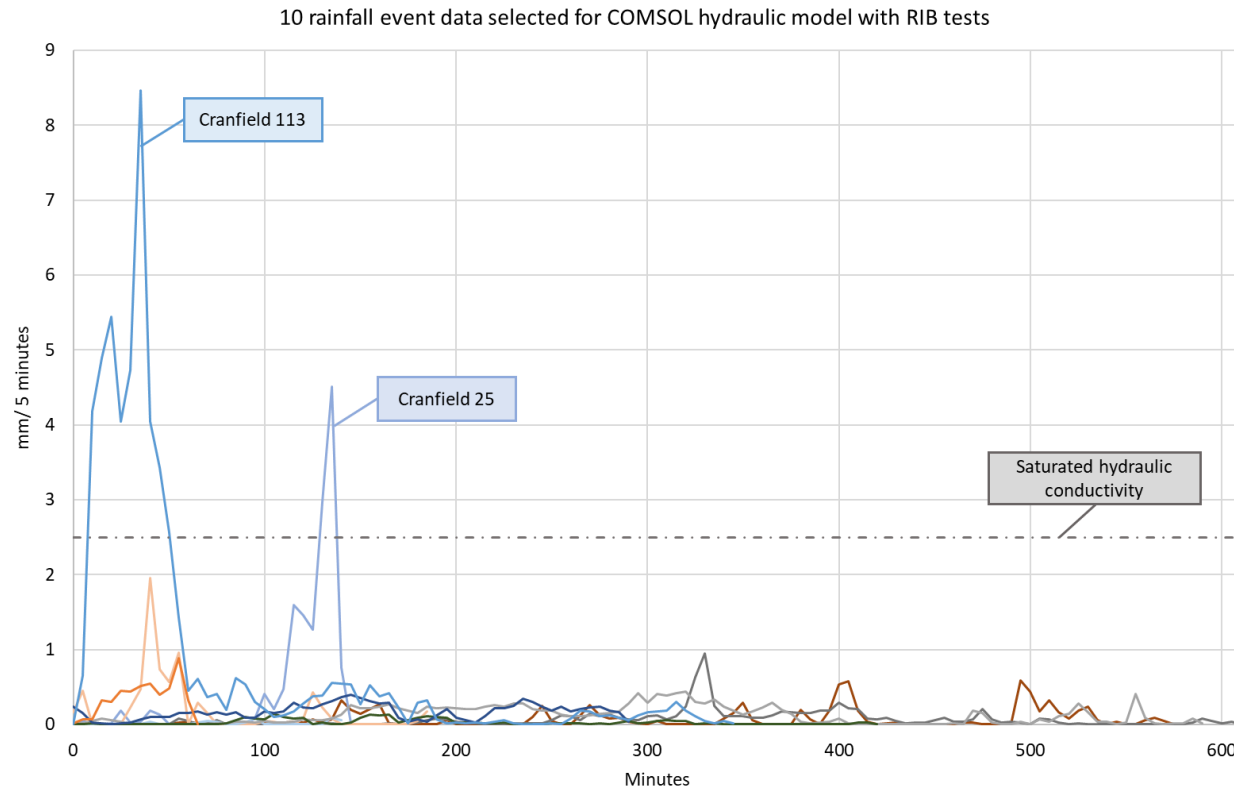


Figure C-3 Ten of the rainfall events from the Cranfield weather station were selected for the RIB tests. Cranfield 113 and Cranfield 25 are the two heaviest rainfall events. The loamy sand saturated hydraulic conductivity is also shown in the figure.

C.4 Rainfall intensity of R1

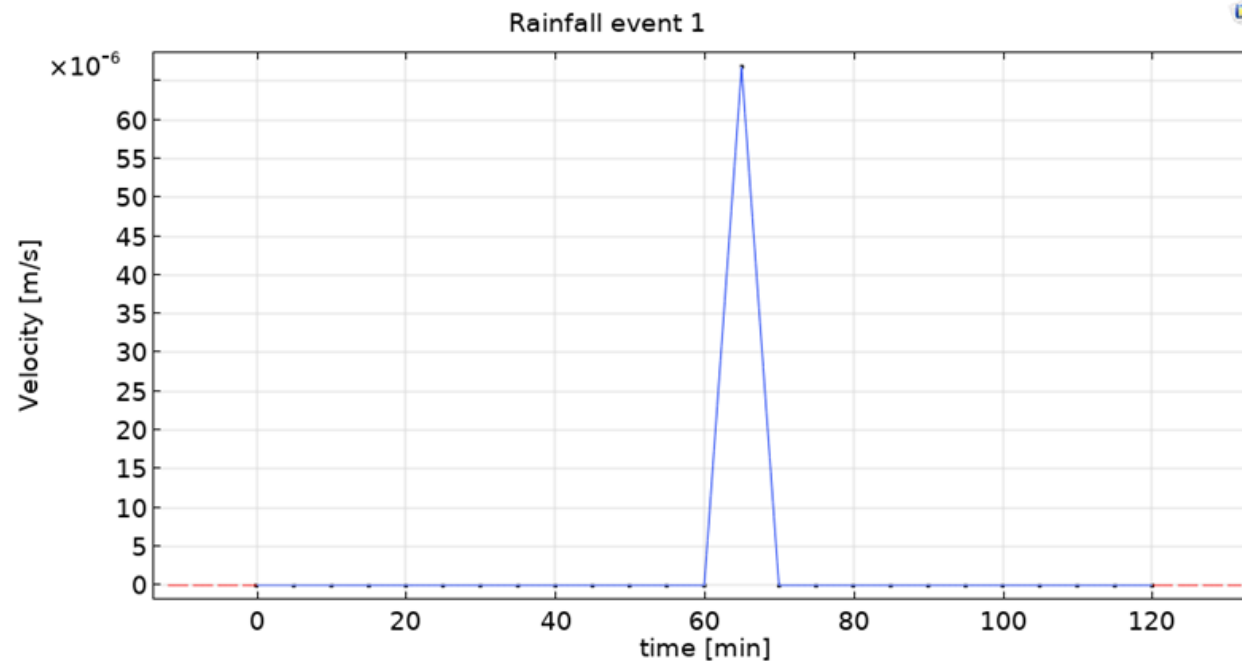


Figure C-4 Rainfall velocity of Rainfall event 1 (R1)

Appendix D ROSETTA curve fitting results

The fitting curve (Figure D-1, Figure D-2, Figure D-3, Figure D-4, Figure D-5, Figure D-6) shows a high correlation between porosity and soil hydraulic characteristics. The correlation equation can be used as an input function for the clogging model to estimate the soil hydraulic characteristics value.

The input soil texture distribution and bulk density have a significant impact on ROSETTA model prediction. The correlation equation needs to be re-calculated when changing the soil texture.

D.1 Sand layer

D.1.1 Saturated hydraulic conductivity

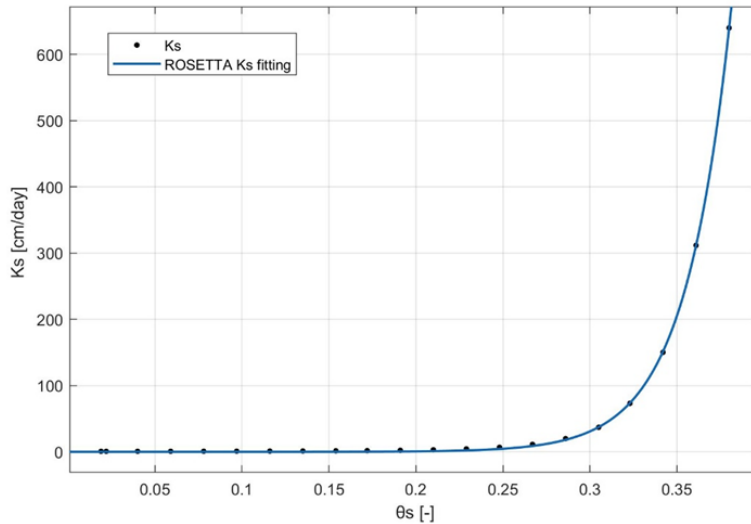


Figure D-1 Saturated hydraulic conductivity (Ks) plotted against porosity and curve fitting

One-term exponential curve fitting equation ($R^2 = 0.9999$):

$$f(x) = ae^{bx}$$

$$a = 0.0003548; b = 37.91$$

D.1.2 The residual liquid volume fraction

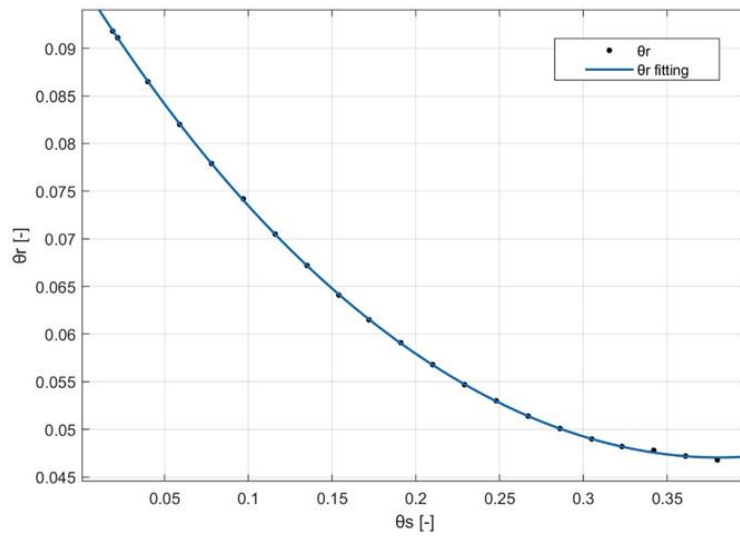


Figure D-2 Residual liquid volume fraction (θ_r) plotted against porosity and curve fitting

Two-term exponential curve fitting equation ($R^2 = 1.0$):

$$f(x) = ae^{bx} + ce^{dx}$$

$$a = 0.09389; b = -3.168; c = 0.003167; d = 4.697$$

D.1.3 van-Genuchten retention model parameters

van-Genuchten parameter α

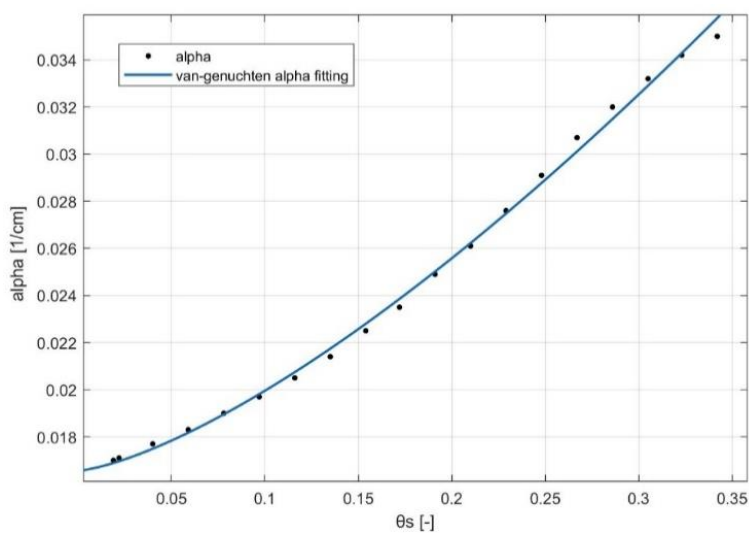


Figure D-3 van-Genuchten α plotted against porosity and curve fitting

The values of curve fitting and calculated α have a significant difference when the porosity is higher than 0.3. The fitting method has been chosen for simplification with fewer coefficients. However, it might cause a larger α value at high porosity. Currently, there are no significant differences found for long-term clogging simulation, but the value might need to be fixed with short-term or event-based simulation.

Two-term power curve fitting equation ($R^2 = 0.9972$):

$$f(x) = ax^b + c$$

$$a = 0.08718; b = 1.41; c = 0.01656$$

van-Genuchten parameter n

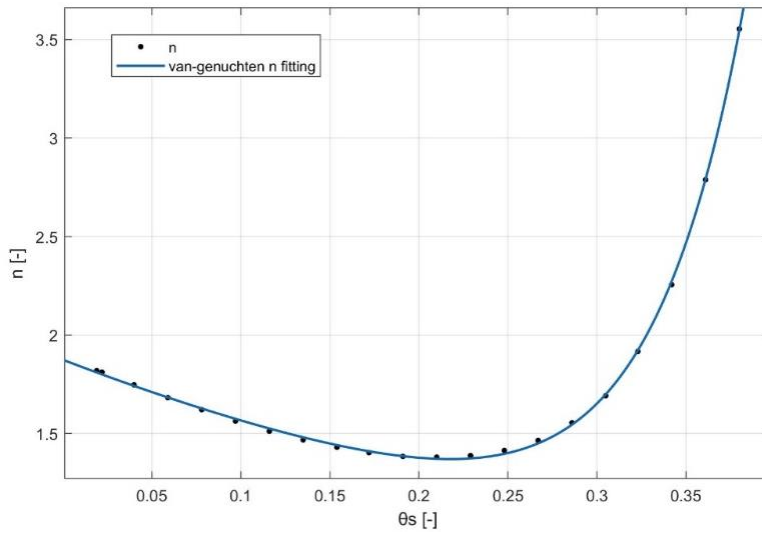


Figure D-4 van-Genuchten n plotted against porosity and curve fitting

Two-term exponential curve fitting ($R^2 = 0.9996$):

$$f(x) = ae^{bx} + ce^{dx}$$

$$a = 1.872; b = -1.871; c = 0.002068; d = 18.81$$

D.2 Sandy loam layer

D.2.1 Saturated hydraulic conductivity

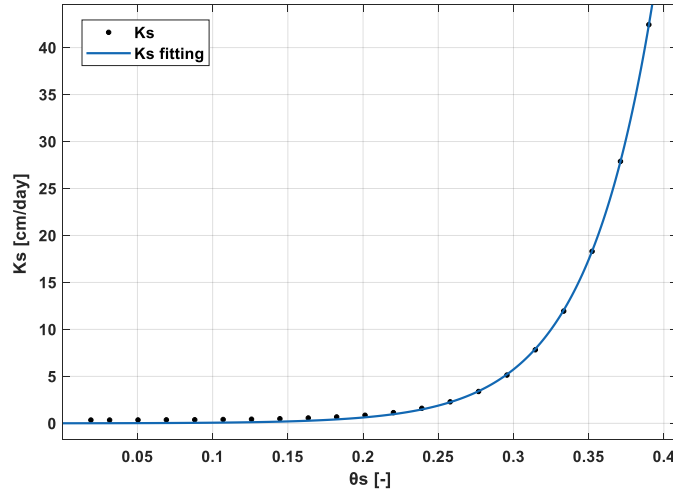


Figure D-5 Saturated hydraulic conductivity (K_s) plotted against porosity and curve fitting for sandy loam

One-term exponential curve fitting equation ($R^2 = 0.9995$):

$$f(x) = ae^{bx}$$

$$a = 0.007291; b = 22.23$$

D.2.2 The residual liquid volume fraction

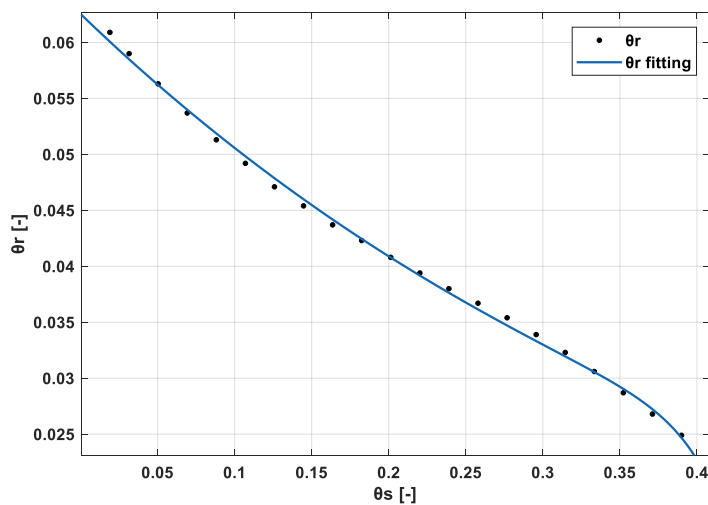


Figure D-6 Residual liquid volume fraction (θ_r) plotted against porosity and curve fitting for sandy loam

Two-term exponential curve fitting equation ($R^2 = 0.9979$):

$$f(x) = ae^{bx} + ce^{dx}$$

$$a = -2.173e^{-10}; b = 41.82; c = 0.06255; d = -2.125$$

D.2.3 van-Genuchten retention model parameters

van-Genuchten parameter α

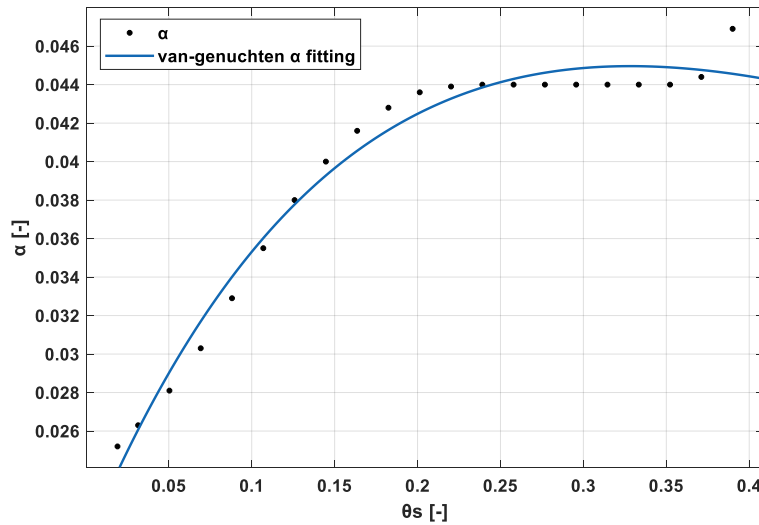


Figure D-7 van-Genuchten α plotted against porosity and curve fitting for sandy loam

The values of curve fitting and calculated α have a significant difference when the porosity is higher than 0.25. The fitting method has been chosen for simplification with fewer coefficients. However, it might cause a lower α value at high porosity. Currently, there are no significant differences found for long-term clogging simulation, but the value might need to be fixed with short-term or event-based simulation.

Two-term exponential curve fitting equation ($R^2 = 0.9746$):

$$f(x) = ae^{bx} + ce^{dx}$$

$$a = 0.08813; b = -1.153; c = -0.06794; d = -4.524$$

van-Genuchten parameter n

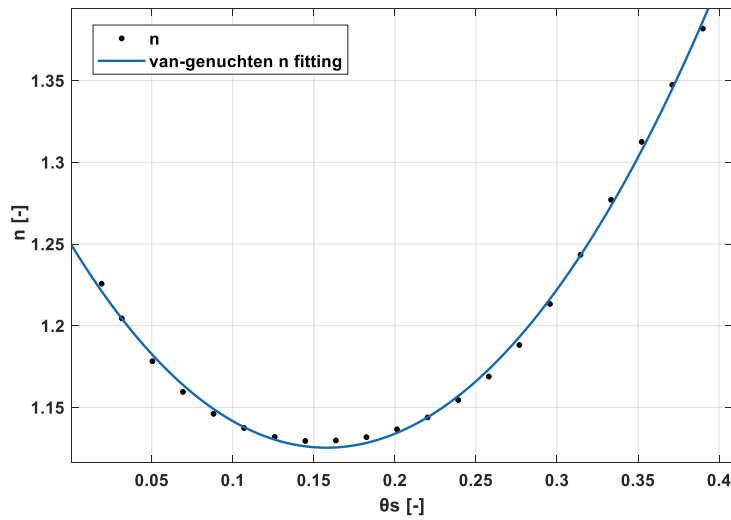


Figure D-8 van-Genuchten n plotted against porosity and curve fitting for sandy loam

Two-term exponential curve fitting ($R^2 = 0.9979$):

$$f(x) = ae^{bx} + ce^{dx}$$

$$a = 0.8571; b = -3.145; c = 0.393; d = 2.719$$

Appendix E Input value and calculated variables for ROSETTA model

Table E-1 ROSETTA model input value and predicted soil hydraulic characteristics value of saturated hydraulic conductivity, residual liquid volume fraction and van-Genuchten parameter α and n (sand layer).

ROSETTA variables for sand												
ROSETTA input value				calculation					Predicted soil hydraulic characteristics value			
bulk density [g/cm ³]	Sand [%]	Silt [%]	Clay [%]	sand mass [g]	silt mass [g]	clay mass [g]	total mass [g]	Porosity [-]	θ_r [-]	α [1 cm ⁻¹]	n [-]	Saturated hydraulic conductivity [cm/day]
1.643	95.0	5.0	0.0	1.5609	0.0822	0.0000	1.643	0.380	0.0468	0.0347	3.5543	640.08
1.693	92.2	4.9	3.0	1.5609	0.0822	0.0500	1.693	0.361	0.0472	0.035	2.789	311.7
1.743	89.5	4.7	5.7	1.5609	0.0822	0.1000	1.743	0.342	0.0478	0.035	2.2559	150.27
1.793	87.1	4.6	8.4	1.5609	0.0822	0.1500	1.793	0.323	0.0482	0.0342	1.917	73.3
1.843	84.7	4.5	10.9	1.5609	0.0822	0.2000	1.843	0.305	0.049	0.0332	1.6923	36.95
1.893	82.5	4.3	13.2	1.5609	0.0822	0.2500	1.893	0.286	0.0501	0.032	1.5543	19.95
1.943	80.3	4.2	15.4	1.5609	0.0822	0.3000	1.943	0.267	0.0514	0.0307	1.4647	11.29
1.993	78.3	4.1	17.6	1.5609	0.0822	0.3500	1.993	0.248	0.053	0.0291	1.4136	6.83
2.043	76.4	4.0	19.6	1.5609	0.0822	0.4000	2.043	0.229	0.0547	0.0276	1.3875	4.46
2.093	74.6	3.9	21.5	1.5609	0.0822	0.4500	2.093	0.210	0.0568	0.0261	1.3796	3.1
2.143	72.8	3.8	23.3	1.5609	0.0822	0.5000	2.143	0.191	0.0591	0.0249	1.3838	2.29
2.193	71.2	3.7	25.1	1.5609	0.0822	0.5500	2.193	0.172	0.0615	0.0235	1.403	1.8
2.243	69.6	3.7	26.7	1.5609	0.0822	0.6000	2.243	0.154	0.0641	0.0225	1.4304	1.48
2.293	68.1	3.6	28.3	1.5609	0.0822	0.6500	2.293	0.135	0.0672	0.0214	1.4681	1.27
2.343	66.6	3.5	29.9	1.5609	0.0822	0.7000	2.343	0.116	0.0705	0.0205	1.5117	1.11
2.393	65.2	3.4	31.3	1.5609	0.0822	0.7500	2.393	0.097	0.0742	0.0197	1.5634	0.99
2.443	63.9	3.4	32.7	1.5609	0.0822	0.8000	2.443	0.078	0.0779	0.019	1.6211	0.91
2.493	62.6	3.3	34.1	1.5609	0.0822	0.8500	2.493	0.059	0.082	0.0183	1.6818	0.85
2.543	61.4	3.2	35.4	1.5609	0.0822	0.9000	2.543	0.040	0.0865	0.0177	1.7469	0.8
2.593	60.2	3.2	36.6	1.5609	0.0822	0.9500	2.593	0.022	0.0911	0.0171	1.8113	0.77
2.600	60.0	3.2	36.8	1.5609	0.0822	0.9570	2.6	0.019	0.0918	0.017	1.819	0.77

Table E-2 ROSETTA model input value and predicted soil hydraulic characteristics value of saturated hydraulic conductivity, residual liquid volume fraction and van-Genuchten parameter α and n (sandy loam layer).

ROSETTA variables for sandy loam												
ROSETTA input value				calculation					Predicted soil hydraulic characteristics value			
bulk density [g/cm ³]	Sand [%]	Silt [%]	Clay [%]	sand mass [g]	silt mass [g]	clay mass [g]	total mass [g]	Porosity [-]	θ_r [-]	α [1 cm ⁻¹]	n [-]	Saturated hydraulic conductivity [cm/day]
1.6165	60.0	40.0	0.0	0.9699	0.6466	0.0000	1.6165	0.390	0.0249	0.0469	1.3818	42.44
1.6665	58.2	38.8	3.0	0.9699	0.6466	0.0500	1.6665	0.371	0.0268	0.0444	1.3474	27.89
1.7165	56.5	37.7	5.8	0.9699	0.6466	0.1000	1.7165	0.352	0.0287	0.0429	1.3125	18.31
1.7665	54.9	36.6	8.5	0.9699	0.6466	0.1500	1.7665	0.333	0.0306	0.0422	1.2771	11.95
1.8165	53.4	35.6	11.0	0.9699	0.6466	0.2000	1.8165	0.315	0.0323	0.0423	1.2434	7.83
1.8665	52.0	34.6	13.4	0.9699	0.6466	0.2500	1.8665	0.296	0.0339	0.0428	1.2133	5.14
1.9165	50.6	33.7	15.7	0.9699	0.6466	0.3000	1.9165	0.277	0.0354	0.0431	1.1882	3.39
1.9665	49.3	32.9	17.8	0.9699	0.6466	0.3500	1.9665	0.258	0.0367	0.0436	1.1689	2.29
2.0165	48.1	32.1	19.8	0.9699	0.6466	0.4000	2.0165	0.239	0.038	0.044	1.1545	1.6
2.0665	46.9	31.3	21.8	0.9699	0.6466	0.4500	2.0665	0.220	0.0394	0.0439	1.1439	1.14
2.1165	45.8	30.6	23.6	0.9699	0.6466	0.5000	2.1165	0.201	0.0408	0.0436	1.1366	0.86
2.1665	44.8	29.8	25.4	0.9699	0.6466	0.5500	2.1665	0.182	0.0423	0.0428	1.1318	0.68
2.2165	43.8	29.2	27.1	0.9699	0.6466	0.6000	2.2165	0.164	0.0437	0.0416	1.1298	0.57
2.2665	42.8	28.5	28.7	0.9699	0.6466	0.6500	2.2665	0.145	0.0454	0.04	1.1295	0.49
2.3165	41.9	27.9	30.2	0.9699	0.6466	0.7000	2.3165	0.126	0.0471	0.038	1.132	0.44
2.3665	41.0	27.3	31.7	0.9699	0.6466	0.7500	2.3665	0.107	0.0492	0.0355	1.1374	0.41
2.4165	40.1	26.8	33.1	0.9699	0.6466	0.8000	2.4165	0.088	0.0513	0.0329	1.1461	0.38
2.4665	39.3	26.2	34.5	0.9699	0.6466	0.8500	2.4665	0.069	0.0537	0.0303	1.1595	0.37
2.5165	38.5	25.7	35.8	0.9699	0.6466	0.9000	2.5165	0.050	0.0563	0.0281	1.1783	0.36
2.5665	37.8	25.2	37.0	0.9699	0.6466	0.9500	2.5665	0.032	0.059	0.0263	1.2045	0.35
2.5995	37.3	24.9	37.8	0.9699	0.6466	0.9830	2.5995	0.019	0.0609	0.0252	1.2257	0.35