A RISK-BASED EVALUATION OF THE LONG-TERM PERFORMANCE OF
STORMWATER INFILTRATION FACILITIES

by

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Infiltration facilities are source control mechanisms that are implemented in urban developments with reduced natural permeable surfaces. Despite the development of design criteria for infiltration facilities, these systems continue to fail due to headloss development, overflow, or chemical breakthrough. The limited research on the long-term performance of these systems has emphasized the role of physical filtration mechanisms within porous media filters to address concerns surrounding system failure, namely filter clogging. A continuous macroscopic depth filtration model was developed to investigate the clogging potential of the underlying sand filter. This continuous model furthers the understanding of temporal and spatial changes in system performance for the development of more appropriate design criteria and more suitable maintenance regimes. The characterization of long-term system performance by defining three different failure modes and a probabilistic approach comprises a comprehensive methodology by considering several performance criteria rather than assuming that one criterion dictates the overall system performance.
I would foremost like to thank my supervisor, Professor Barry Adams for his support, patience, and wisdom throughout this process. I have grown professionally by being challenged to think independently and have gained confidence in my own judgment. I would also like to thank Professor Brent Sleep for reading and providing helpful comments as a second reader for this thesis.

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To my parents, who first suggested that I pursue engineering more than seven years ago; I am forever grateful for your encouragement and belief in me which has lead to some of the best years of my life. To my two brothers, Ian and Eric, and sister-in-law, Marion, I thank you for your love, good humour, and most of all your friendship.

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<th>Description</th>
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<tr>
<td>BMP</td>
<td>Best Management Practice</td>
</tr>
<tr>
<td>EMC</td>
<td>Event Mean Concentration</td>
</tr>
<tr>
<td>LU</td>
<td>Land Use</td>
</tr>
<tr>
<td>PDE</td>
<td>Partial Differential Equation</td>
</tr>
<tr>
<td>PDF</td>
<td>Probability Density Function</td>
</tr>
<tr>
<td>RSF</td>
<td>Rapid Sand Filtration</td>
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<tr>
<td>SSF</td>
<td>Slow Sand Filtration</td>
</tr>
<tr>
<td>SUDS</td>
<td>Sustainable Urban Discharge System</td>
</tr>
<tr>
<td>SWMM</td>
<td>Stormwater Management Model</td>
</tr>
<tr>
<td>USDA</td>
<td>United States Department of Agriculture</td>
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<td>VBA</td>
<td>Visual Basic Application</td>
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Chapter 1

BACKGROUND AND THESIS OBJECTIVES

1.1 URBAN DEVELOPMENT

The development of urban centres has resulted in major adverse effects on the natural environment. The continual alteration of rural landscapes into urban centres has resulted in an increase in the quantity and decrease in the quality of urban runoff that is discharged into receiving waters. In addition, natural permeable surfaces that promote infiltration of precipitation are substituted with impermeable surfaces such as roads, parking lots, roofs, driveways, and sidewalks (Roesner et al., 2001).

Non-point source or diffuse pollution is a major environmental issue that poses a threat to surface and groundwater quality (Muthukrishnan et al., 2006b). Urban runoff is often laden with potential contaminants such as organic compounds, suspended solids, nutrients, heavy metals, and bacteria that may pose acute or chronic threats to receiving water quality. Rapid urbanization over the past 100 years has degraded the quality of water courses and driven the need for more stringent water regulations and policies.

As a result of degraded water quality of receiving waters, best management practices (BMPs) have been implemented in urban watersheds to control this pollution by removing, reducing, retarding, or preventing urban stormwater runoff quantity and pollutants from reaching receiving waters (Strecker et al., 2001). Common structural BMPs applied in urban watersheds include: detention and retention ponds, stormwater wetlands, vegetative biofilters, infiltration practices, sand and organic filters, and other technologies (Muthukrishnan et al., 2006a). Non-structural BMPs for urban stormwater runoff management may also be implemented such as: street and storm drain maintenance, public education and regulatory controls to reduce potential pollutants near the source before
CHAPTER 1. BACKGROUND AND THESIS OBJECTIVES

contact with stormwater (Muthukrishnan et al., 2006a). Table 1.1 summarizes common non-structural and structural BMPs that may be implemented to reduce the volume of urban runoff and also provide a degree of treatment prior to discharge into receiving waters.

Table 1.1 Nonstructural and Structural BMP Categories (Muthukrishnan et al., 2006a)

<table>
<thead>
<tr>
<th>Non-structural Best Management Practices</th>
<th>Structural or Treatment Best Management Practices</th>
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<tr>
<td>Public Education</td>
<td>Ponds</td>
</tr>
<tr>
<td>Planning and Management</td>
<td>Stormwater Wetlands</td>
</tr>
<tr>
<td>Materials Management</td>
<td>Vegetative Biofilters</td>
</tr>
<tr>
<td>Street/Storm Drain Maintenance</td>
<td>Infiltration Practices</td>
</tr>
<tr>
<td>Spill Prevention and Cleanup</td>
<td>Sand and Organic Filters</td>
</tr>
<tr>
<td>Illegal Dumping Controls</td>
<td>Treatment technology Options and Others</td>
</tr>
<tr>
<td>Illicit Connection Controls</td>
<td></td>
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<tr>
<td>Stormwater Reuse</td>
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Typically, infiltration facilities and stormwater management ponds have attracted more attention as viable BMP practices compared to other alternative practices listed above (Ellis et al., 1996). For example, early detention ponds were efficient at reducing downstream flooding; however, they had minimal influence on water quality impacts presented by the increased urban development. Therefore, groundwater infiltration practices have been introduced and viewed as advantageous towards achieving better receiving water quality as they maintain the natural local hydrology and water table levels, and reduce pollution discharges to receiving waters (Duchene et al., 1992).

Specifically, infiltration practices are source control alternatives, in contrast to end-of-pipe control measures that are located near the point of discharge, and are commonly referred to as sustainable urban drainage systems (SUDS). Although infiltration practices possess many benefits, Galli (1992) revealed that historically, high rates of failure has been associated with infiltration facilities, particularly infiltration trenches when compared to other BMPs.
1.2 INFILTRATION PRACTICES

Infiltration practices include infiltration trenches, infiltration land use (LU) curbs, soakaways, biofilters, infiltration inlets, and permeable pavements (Furumai et al., 2005). Conventionally, LU curbs are used to collect urban runoff from gutters, where runoff may be infiltrated into a crushed stone and sand-fill layer (Furumai et al., 2005). When these urban runoff management practices are in proper operation, they have the long-term potential to remove sediment, phosphorus, nitrogen, trace metals, biological oxygen demand, and bacteria (Duchene et al., 1992) from infiltrated runoff through various filtration mechanisms, which is discussed in Chapter 2. The present research specifically focuses on the long-term performance of infiltration trenches.

Infiltration trenches are storage reservoirs for temporary detention of urban runoff that are constructed with an excavated trench (typically 0.5 to 3 metres in depth) backfilled with a coarse porous material, commonly a clear stone fill (Government of British Columbia, 2006; Schueler 1987). The underlying subsoil, also referred to as the sand filter, removes solid particles and associated pollutants from the fluid within the porous media through surface filtration, straining, and other physical-chemical removal mechanisms such as adsorption (McDowell-Boyer et al., 1986). A filter fabric may also be used in place of the sand filter. Typically, this sand filter or permeable layer is placed at the bottom of the trench and is approximately 15 to 30 cm in depth (Schueler, 1987). The design filter depth assumes consolidation of media during installation and no accumulation of deposited sediments on the top segment of the filter (i.e., cake layer). Particle removal throughout the infiltration trench is mostly influenced by the relative size of the particle with respect to the grain size distribution of the porous medium. Small particles transported in the inflowing fluid may reduce the permeability of the filter media by clogging the pores and eventually reducing the flow discharging into the underlying aquifer. The gradual reduction in permeability of the subsoil increases the risk of overflow events and the need for major rehabilitation of the infiltration trench.

Figure 1.1 illustrates the configuration of a typical infiltration trench (Schueler, 1987).
A by-pass system is commonly used to re-direct large event flows that exceed the capacity of the infiltration trench (Barr Engineering Company, 2001) and to prevent ponding at the surface of the trench. This by-passed urban runoff is then discharged directly into nearby surface water courses. Trench configuration is based upon the design volume, trench geometry, drawdown time, and site soil permeability (Barr Engineering Company, 2001).

To ensure the proper performance of infiltration trenches, regular maintenance is required as well as the use of pre-treatment facilities (i.e., vegetative buffer strips) or filter fabrics which remove large coarse particulates that result in rapid clogging of the porous material or subsoils. A layer of filter fabric may be placed along the walls of the trench to increase the soil stability and to reduce the risk of contamination in the surrounding soil. Filter fabrics may also be placed at the base of the trench; however, this placement is not always necessary. In addition, a layer of filter fabric may be located at approximately 0.3 m below the ground surface to prevent large debris and particles from entering the trench and clogging the surface of the bottom filter.

Figure 1.1 Schematic of a typical infiltration trench (Source: after Schueler, 1987)
The underdrain can also redirect large flows that exceed the capacity of the system and in cases where the drainage capacity of the underlying sand layer has been exceeded. When present, underdrain systems become advantageous once the system becomes clogged and are typically connected to the existing drainage system or another suitable outlet. Further details on design specifications for infiltration trenches are discussed in Chapter 3.

While design criteria for infiltration trenches have been specified in BMP manuals such as the *Minnesota Small Urban Sites BMP Manual* (Barr Engineering Company, 2001), Furumai et al. (2005) state that there is insufficient establishment of a design calculation approach that quantifies the effects and reliability of storage and infiltration practices. In addition, available research on the performance of infiltration facilities has not yet emphasized the role of physical filtration mechanisms within porous granular filters.

### 1.3 Problem Statement

In 1996, Switzerland was the only documented country that legally imposed infiltration practices for urban runoff management (Mikkelsen et al., 1996). This practice has been known to be economically cheaper to implement than the equivalent number of detention basins to achieve the same level of flood protection (Fujita and Koyama, 1990).

Acceptance of infiltration trenches (and other infiltration practices) has been impeded due to several concerns including but not limited to groundwater quality impacts, lack of design guidance, and concerns with maintenance and longevity of these systems (Ellis and Marsalek, 1996). In recent investigations, it has been confirmed that the average event infiltration rate and, therefore, the longevity of these systems, reduces asymptotically with time, which is likely due to clogging of the overlying and underlying soils. Galli (1992) revealed from an investigation of approximately 60 infiltration trenches that less than 30 percent of the trenches studied were functioning properly after 5 years of operation. System failure may result in contamination of groundwater sources, clogging, and overflow with subsequent discharge of harmful pollutants into surface water courses.
Existing standards for designing and operating infiltration systems do not adequately address the uncertainties and variability in subsurface strata (specifically permeability), clogging, and infiltrated volume, nor are long-term changes in the system over the lifetime of operation considered. As a result, infiltration devices are subject to failure.

1.4 Thesis Objectives

Failure of infiltration facilities is mainly due to clogging of the filter media, despite pre-treatment mechanisms (Le Coustumer and Barraud, 2007) and maintenance practices such as street sweeping. Infiltration models such as the Green-Ampt or Horton equations have been used in the past to simulate the performance of infiltration facilities; however, these models lack the ability to simulate filter clogging due to particle straining and bio-growth over time. Few studies have applied existing filtration models, commonly implemented in water and wastewater treatment design, to urban runoff problems with intermittent wet and dry periods.

The principal goals of this thesis are (1) to assess the level of risk of system failure, including the probability of overflow and groundwater quality exceedence and the effects of such occurrences by considering clogging mechanisms within the filter media; and (2) to rationalize their design criteria and suggest maintenance regimes for infiltration facilities through this risk-based approach. This thesis provides a comprehensive methodology for assessing the long-term performance of infiltration facilities which contributes to better engineering decision-making.

The specific objectives of this thesis include the following:

a. To perform continuous simulation using historical meteorological data and a macroscopic filtration model to better understand the physical clogging and resulting failure of BMP filter beds;
CHAPTER 1. BACKGROUND AND THESIS OBJECTIVES

b. To employ a probabilistic analysis of macroscopic depth filtration theory to establish the long-term risk of system failure associated with filter bed clogging including groundwater contamination and trench overflow events; and

c. To rationalize infiltration facility design criteria and operation and management regimes such that an acceptable level of risk of system failure results.

1.5 THESIS OUTLINE

By assessing the probability of system failure caused by design uncertainties and long-term filter behaviour, more suitable design criteria and better maintenance regimes may be suggested to improve the future performance of these systems. Incorporation of uncertainty analysis in design may further the acceptance of these systems in water management regulations and policies.

A brief introduction, including background information and the problem statement are provided in this chapter, including the specific thesis objectives.

Chapter 2 investigates the history behind filtration and specifically recent studies in the field of stormwater filtration. Also, mechanisms of urban runoff filters including hydraulic behaviour and particle removal are discussed.

In Chapter 3, a summary of common failure mechanisms that have inhibited the acceptance and further development of infiltration technologies is provided. The modelling approaches commonly taken for particle transport and removal are introduced, including the use of microscopic and macroscopic filtration models.

Chapter 4 presents and discusses the macroscopic depth filtration model in detail, including cake filtration theory, followed by an illustration of the proposed modelling approach including boundary conditions and model limitations.
In Chapter 5, simulation results which illustrate the long-term physical behaviour of infiltration stormwater filters are presented and discussed. Chapter 6 assesses the timing of failure events on a long-term basis through the application of Monte Carlo simulation. Chapter 7 incorporates macroscopic theory and results from the probabilistic analysis into recommended infiltration trench design and maintenance regimes. Overall conclusions and recommendations for this study are presented in Chapter 8.
Chapter 2

FILTRATION TECHNOLOGY & PARTICLE TRANSPORT THROUGH POROUS MEDIA

2.1 HISTORY OF FILTER TECHNOLOGY

Early development of filtration originates from both Scotland and England. Slow sand filtration was preceded by the Lancashire filter which is speculated to have been developed prior to the end of the 18th century (Baker, 1949). However, the first patent issued by the British Patent Office that was related to filtration was in 1790 and was issued for the composition and manufacturing of household filters from earthenware (Baker, 1949). Later, in 1791, James Peacock was issued a filter patent in England (Peacock, 1799 as cited in Baker, 1949) and subsequently published a renowned expository pamphlet (Peacock, 1793 as cited in Baker, 1949). However, Paisley, Scotland was the first town to receive filter-supplied water in 1804 (Baker, 1949). By the early 19th century, two companies in Glasgow were involved in the early piping of filtered water to consumers by building half a dozen filter plants, however, none of these plants succeeded (Baker, 1949).

It was not until 1827 that the slow sand filter (SSF) was designed by Robert Thom and was implemented in Greenock, Scotland. Shortly after, James Simpson designed a similar filter in 1829 with a one-acre SSF for the Chelsea Water Company in London, UK (Hendricks, 2005) which became the model for the English SSF throughout the world (Baker, 1949). Thom’s model introduced rapid filter features including false bottoms and reverse flow backwash which were later used in the development of the rapid sand filter (RSF) in the United States during the 1880s (Baker, 1949; Hendricks, 2005). The RSF which evolved from propriety innovations became the most common filtration technology, described as rapid rate filtration, mechanical filtration, and more recently, depth filtration (Hendricks, 2005). Throughout the 1900s, depth or rapid filtration design evolved as an empirical practice and by the 1980s a
better understanding of process theory developed and began to influence design and operation (Hendricks, 2005).

As cited in Li and Davis, 2008b, mathematical filtration expressions were first derived by Iwasaki (1937) to model and predict the behaviour of various filtration systems. Referred to as Iwasaki’s Equations, theoretical expressions were published in a paper titled “Some Notes on Sand Filtration.” These equations describe the removal of suspended particles within the filter depth of a granular sand bed (Hendricks, 2005). Iwasaki’s filtration models have been widely accepted as the foundation for modern filtration theory (Hendricks, 2005), including both slow and rapid sand filtration.

First, the rate of accumulation of particles within a minute slice of the total filter depth is described by the net rate of advection of particles in and out of the filter (Hendricks, 2005; Tien and Ramarao, 2007). Equation (2.1) presents a simplified partial differential equation (PDE) under the assumption that the deposition of suspended solids within the deposit porosity is negligible (Tien and Ramarao, 2007; Li and Davis, 2008b):

\[
0 = \frac{\partial}{\partial Z} \left( u_s C \right) + \frac{\partial}{\partial t} \sigma = 0
\]

(2.1)

where \( u_s \) is the superficial filtration (approach) velocity (L/T), \( C \) is the particle concentration in the fluid (M/V), \( Z \) is the total filter depth (L), \( \sigma \) is the specific deposit of particles (M/V), and \( t \) is the time of filtration. Equation (2.1) also assumes a mono-disperse distribution of deposited particles in addition to one-dimensional flow, neglecting dispersal effects (Hendricks, 2005).

Experimental data from SSFs demonstrated an exponentially decaying profile of particle concentration through the filter depth (Metcalf and Eddy, 2003; Hendricks, 2005; Tien and Ramarao, 2007). Iwasaki (1937), as cited in Li and Davis (2008b), expressed the kinetic profile of particles by:
\[
\frac{\partial C}{\partial Z} = -\lambda C
\]  

(2.2)

where \( \lambda \) is the filter bed coefficient (L\(^{-1}\)). This coefficient is a measure of the extent of particle removal within the filter bed depth (Hendricks, 2005).

As cited by Li and Davis (2008b), Ives (1963) demonstrated that the vertical profile of a volumetric specific deposit is approximately the same as the vertical particle concentration profile in the fluid phase (Tien and Ramarao, 2007). This relationship is expressed by:

\[
\frac{C}{C_o} = \frac{\sigma_v}{\sigma_{vo}}
\]

(2.3)

where \( C_o \) is the influent particle concentration (M/V); \( \sigma_v \) is the volumetric specific deposition (V/V); and \( \sigma_{vo} \) is the volumetric specific deposition at depth=0. Note that \( \sigma = \sigma_v \rho_s \) and that \( \rho_s \) is the particle density of the deposited solid (M/V).

The filter bed coefficient presented in Equation (2.2) is the clean bed filter coefficient, \( \lambda_o \), and describes the performance of the filter bed in the early stages of filtration. The clean bed coefficient is a function of the properties of particles entering the filter and the properties of the filter media (Hendricks, 2005). As particles are deposited in the void spaces of the filter, the particles clog the filter pores. The dynamic behaviour of filter clogging is expressed by the filter bed coefficient as a function of the cumulative specific deposit. Equation (2.4) linearly relates the filter bed coefficient to the specific deposit:

\[
\lambda = \lambda_o + c\sigma_v
\]

(2.4)

where \( c \) is an empirical constant (positive or negative). Variations of Equation (2.3) have been derived through empirical studies and are summarized by Tien and Ramarao (2007).

The present study investigates the various forms of Equation (2.4) and selects a relationship that best describes the expected aging behaviour of a stormwater filtration system. In this
expression, the accumulation of particles creates more contact opportunities for particle removal (i.e., increasing the likelihood of particle removal within the filter bed) (Hendricks, 2005; Li & Davis, 2008b).

More recently, the theoretical understanding of depth filtration has been enhanced by Chi Tien and colleagues including prediction of the dynamic behaviour of filters over their entire operational life. Tien et al. (1979) describes two consecutive stages in the filtration process:

a. the early phase of filtration occurs as a smooth and uniform coating mode of deposited solids; and

b. the specific deposit reaches a transition value, and subsequent deposition occurs by the constriction clogging mode.

Tien et al. (1979) developed a deep filter bed modelling approach, from the macroscopic material balance equations, which uses theoretical calculations to estimate changes in the filter bed coefficient, and emphasizes the significance of the morphological and evolving nature of deposition within the filter. Specifically, Tien et al. (1979) use both spherical and constricted tube configurations throughout a unit bed element (a series of unit bed elements comprise the entire filter depth) to determine both the rate of particle deposition and the effect of particle deposition, respectively.

Much of the developing research and modelling approaches on stormwater filtration systems have applied theory from SSF and RSF models. The following section investigates research contributions to the field of stormwater filtration, particularly those which provide background for the present study.

### 2.2 Recent Advancements in Stormwater Filtration Theory

Hatt et al., (2007a; 2007b) studied the performance of stormwater filtration systems, specifically filters that are comprised of biofiltration, gravel, sand, or soil media. In these
studies, the authors modelled the long-term hydraulic and pollutant removal performance of fine media stormwater filters at the laboratory scale using column experiments. In particular, Hatt et al. (2007a) focused on the wet-dry periods and the implication of these periods on hydraulic performance and contamination removal. In addition, Hatt et al. (2007a) affirm that mechanical straining is the primary removal mechanism for sediment removal rather than biological processes which are critical for removal of organic nitrogen compounds. As a result, Hatt et al. (2007a) found that sediment removal through the stormwater filters was not affected by varying wet-dry periods. In addition, heavy metals and phosphorous, which are typically particulate-bound, are removed by the same physical processes and are subsequently not affected by wet-dry cycles (Muthukaruppan et al., 2002; Taylor, 2006; Hatt et al., 2007a).

Furthermore, Hatt et al., (2007b) brought focus to the threat of contamination that stormwater infiltration systems pose on the underlying soils and groundwater system. Pollutants that are not captured in the filter or are remobilized in the filter media may exfiltrate through the infiltration system. Through column experiments, Hatt et al. (2007b) determined that adequate removal of sediment and heavy metals may be achieved by gravel filters with a depth of 0.5 m. In addition, it is anticipated that chemical breakthrough will not be of concern since physical clogging is likely to occur prior to chemical breakthrough (Hatt et al., 2007b).

Li and Davis (2008a; 2008b) focused on the long-term behaviour of bioretention systems. Unlike sand or gravel filters, bioretention systems utilize a mixture of media (often stratified) including but not limited to soil, sand, and mulch which act as collectors for urban runoff particles. Typically, bioretention media are smaller than sand filter media, and undergo more dynamic behaviour between wet and dry periods (Li and Davis, 2008a). Research conducted by Li and Davis used theory from SSF and RSF processes and applied these concepts to the dynamic behaviour of bioretention systems. Similar to Hatt et al. (2007b), Li and Davis (2008a; 2008b) studied the performance of bioretention systems by conducting column experiments. Specifically, these experiments focused on the capture of particulate matter, and the development of a cake layer and its ability to enhance particle removal. Minimal research
on cake layer formation has been conducted; however, Li and Davis (2008b) have provided more insight to modelling approaches that simulate both cake formation and depth filtration simultaneously. Figure 2.1 illustrates the theoretically predicted long-term reduction in hydraulic conductivity in a bioretention filter using a simplified form of the O’Melia and Ali (1978) clogging model. This illustration shows the long-term performance for deposited particles of various grain sizes that result in different clogging factors ($\gamma$) (further discussed in Section 2.6).

![Figure 2.1](image.png)

*Figure 2.1* Long-term reduction in hydraulic conductivity ($K_e/K_o$) in a bioretention filter using different clogging factors (Source: after Li and Davis, 2008b)

Based on experimental data, Li and Davis (2008b) concluded that a cake layer on the surface of the bioretention filter will form when the effective hydraulic conductivity ($K_e/K_o$) decreases to a value of 0.3 to 0.4. In addition, assuming a continuous and constant rate of specific deposition across the filter, the hydraulic conductivity approaches zero as time tends to infinity for varying clogging factors (further discussed in Section 2.6). The rate at which the hydraulic conductivity decreases is dependent upon the mean particle size being deposited. For example, when the particle size is small (smooth line), this corresponds to a high clogging factor and causes the hydraulic conductivity to decrease quickly with respect
to time. Conversely, when larger particles are deposited, the clogging factor becomes smaller and results in a more gradual decrease in hydraulic conductivity.

Large particles are expected to be removed at the surface of the filter, whereas smaller particles that are capable of penetrating the surface become trapped in the interstices of the porous media. Although the larger particles result in clogging at the surface, the smaller particles result in stratified clogging throughout the depth of the filter (referred to as particle straining in this study). Lamm et al., (2007) explain that small particles pose clogging issues when they are present in large quantities. Given that the small particles are able to penetrate the filter’s surface as individual particles, they may be inclined to flocculate and attach to organic residues forming larger particles in the depth of the filter, resulting in clogging (Lamm et al., 2007).

Additional modelling of stormwater infiltration practices includes work completed by David and Sousa (2008). This research evaluated the retrofit potential of retention and infiltration facilities (specifically bioretention facilities) with the Lisbon rainfall pattern (Sousa, 2008). David and Sousa (2008) investigated an urban catchment using a long-term rainfall series in a continuous modelling approach; however, this study did not consider the clogging effects of the sub-surface filters. Common amongst many infiltration studies, the modelling approach taken to design storage facilities tends to consider only the risk of system failure associated with the volumetric runoff, neglecting the influence of runoff constituents such as suspended solids. The volumetric model applied in this study follows the continuity approach:

\[ \frac{dS}{dt} = Q_{in} - Q_{out} \]  \hspace{1cm} (2.5)

where \( S \) is the volumetric storage in the basin (\( L^3 \)); \( Q_{in} \) is the runoff inflow (\( L^3/T \)); and \( Q_{out} \) is the outflow through the bottom of the basin (\( L^3/T \)).

The macrofiltration modelling equations, presented above, represent a modelling approach that goes beyond volumetric issues and considers the influence of filtration layers (i.e., deep
filters and cake layer), urban runoff constituents, and the media depth of solids removal (Li
and Davis, 2008b). The modelling approach used in this study combines deep bed filtration
theory taken from Tien and Ramarao (2007) and advances made by Li and Davis (2008a,b)
relating depth filters and cake development, and considers dynamic changes in volumetric
storage.

2.3 FILTRATION TECHNOLOGIES

Filtration mechanisms remove organic and inorganic colloidal or suspended solids from
inflowing fluids and may be categorized by the following three filter types: depth filtration,
surface filtration, and membrane filtration (Metcalf and Eddy, 2003). Typically in urban
runoff filtration, sand filters, which are classified as depth filters, are employed and function
appropriately under intermittent meteorological conditions in contrast to water or wastewater
treatment applications with continuous and relatively steady inflowing fluid streams.

2.3.1 Deep Bed Filters

Deep bed filters (or depth filters) remove particulate material that is suspended in liquid by
passing the liquid through a filter bed that is comprised of a granular or compressible filter
medium (Metcalf and Eddy, 2003) or rigid plastic filtration media.

Li and Davis (2008a) affirm that urban stormwater filtration installation, including
bioretention facilities and conventional sand filters, commonly encounter depth filtration
mechanisms. Over time, fine suspended solids from urban runoff accumulate in the
interstices of the granular medium (Metcalf and Eddy, 2003) and the headloss (h) across the
filter bed increases beyond its initial value (h₀). Most of the suspended solids in the inflowing
fluids are deposited within the first 5 to 20 cm; consequently, the filter media are described
as clogging limited rather than breakthrough limited (Li and Davis, 2008b; Hatt et al., 2008).
Clogging limited filters clog before the suspended solids penetrate the filter and achieve
contaminant breakthrough (Li and Davis, 2008b).
2.3.2 Surface and Membrane Filters

Surface filters remove larger suspended particles from the inflowing liquid through particle exclusion at the filter bed surface rather than throughout the entire depth of the filter. Membrane filters follow a similar particle removal mechanism. Membrane filtration is a pressure-driven filtration process with the application of membranes as the separation medium; however, it differs from other filtration technologies as membranes include the separation of both particulate matter and soluble components (Gutman, 1987). The separated components of the fluid are removed by selective permeation through the membrane (or interphase) (Gutman, 1987). In order for a membrane separation process to work effectively, the membrane must be semi-permeable with a driving force, such as concentration gradient or hydrostatic pressure, which moves molecules from one side of the membrane to the other (Gutman, 1987). Although both surface and membrane filters may effectively remove particulates and soluble components, deposition at the surface of the media (membrane or granular) forms a cake layer which decreases the permeability of the filter media over time (Bradford et al., 2006). This mechanical filtration mechanism is not ideal under urban runoff treatment conditions, such as infiltration trenches and basins, due to clogging problems (Bradford et al., 2006) which result in demanding and costly maintenance programs.

Both surface filters and membrane filters act similar to cake filtration where the filter medium acts as a screen for retaining particles in suspension (Tien, 2006). Figure 2.2 illustrates a simplified schematic of both cake filtration (particle screening) and depth filtration.
CHAPTER 2. FILTRATION TECHNOLOGY AND PARTICLE TRANSPORT THROUGH POROUS MEDIA

![Figure 2.2](image.png) (a) surface/membrane filters; (b) depth filters (Source: after Tien, 2006)

2.4 PARTICLE REMOVAL MECHANISMS

The transport and deposition of suspended particles into porous media is essential in various science and engineering applications including: transport of colloidal contaminants in soil and groundwater, municipal and industrial processes, and chromatographic separation (Song and Elimelech., 1993). Specifically, (1) transport and (2) attachment of particles within the filter bed are the two sequential steps for particle removal (Hendricks, 2005; Metcalf and Eddy, 2003). A transport coefficient, $\eta$, describes the coagulation of inflowing particles into flocs (Hendricks, 2005) and the transport of particles to or near the surface where they will be removed (Metcalf and Eddy, 2003). Transport mechanisms that drive particle removal are illustrated in Figure 2.3 and include: diffusion, gravity settling, and interception (Hendricks, 2005). First, *Brownian diffusion* refers to the random motion of particles due to thermal energy that is superimposed on the advective motion in the filter medium (Hendricks, 2005). A diffusing particle within a porous medium takes a tortuous diffusion path since only part of the travel area is available (Mackay, 2001). Diffusion becomes increasingly important for particles less than 1 μm in diameter since the viscous drag of the fluid restricts random movement and the mean free path is typically only 1 to 2 particle diameters for particles greater than 1 μm (Jegatheesan and Vigneswaran, 2005). Diffusion will likely not be a significant driving mechanism in the following stormwater filtration application and, therefore, is not considered in the present models. Next, *gravity settling* (or sedimentation)
applies Stoke’s law to describe the transport mechanism where a gravitational velocity vector is added to the advective velocity vector (Hendricks, 2005). The resultant particle trajectory (including gravity) causes the particle to settle on the filter media. Last, interception is the transport of particles along a streamline via advection that come in contact with filter media (Hendricks, 2005).

For attachment to occur, collisions must first take place in the filter medium (Hendricks, 2005; McDowell-Boyer et al., 1986). The attachment coefficient (or attachment efficiency), \( \alpha \), describes the fraction of collisions that result in attachment. Hendricks (2005) specifies that the surface characteristics of a particle, such as surface charge or pH, influence whether or not attachment occurs. In addition, attachment depends on both forces and torque that act on the particle (Bradford et al., 2007). Forces that act on suspended particles include: gravity, buoyancy, hydrodynamic drag and lift, electrical double-layer repulsion (or attraction), and London Van Der Waals interaction (Bradford et al., 2007).

![Figure 2.3 Dominant filtration transport mechanisms of suspended particle to collector particle (Source: after McDowell-Boyer et al., 1986)](image)

As inflowing particles are transported to the collector medium and collisions take place, there are three filtration mechanisms, not to be confused with the previously discussed transport or collision mechanisms that describe how particles are removed within porous media. The
primary filtration mechanisms are: surface particle removal, particle straining, and physical-chemical particle removal (Hendricks, 2005). These mechanisms are depicted in Figure 2.4.

![Figure 2.4](image)

**Figure 2.4** (a) surface (cake) particle removal; (b) particle straining; (c) physical-chemical particle removal  
(Source: after McDowell-Boyer et al., 1986)

### 2.4.1 Surface Filtration

The following section elaborates on particle removal through surface or membrane filters. The particle removal mechanism, surface filtration, (also known as cake filtration or vacuum filtration) refers to the accumulation of suspended particles that are too large to penetrate the filter media and, therefore, aggregate as a layer above the porous media (McDowell-Boyer et al., 1986). In surface filtration, particle exclusion occurs with the use of separating mechanisms, such as energy or physical matter (Tien, 2006) which act as the filter. Furthermore, in surface filtration, energy separating mechanisms (i.e., gravitational energy) induce the flow of suspension and filtrate, whereas, physical matter refers to the filter media itself (Tien, 2006). As particles are continually retained above the porous medium, and the pore volume in the filter medium decreases due to deposition of solids, the cake layer thickens and the porosity decreases due to compression; thus, the cake layer is known to become rapidly impermeable (McDowell-Boyer et al., 1986). The permeability of the filter cake layer is a direct function of particle aggregation at the surface of the filter medium. The cake layer may also be described as the clogging layer which limits flow through the filter medium.
Typically, the practice of surface or cake filtration is used for the separation or recovery of suspended particles from suspensions with relatively high solids content, such as dewatering processes in wastewater treatment. Deep bed filtration may be preferable for stormwater or groundwater recharge applications since displacement between the suspending liquid and the suspended particles may occur throughout the entire depth of the filter media, circumventing a clogging-limited filter due to formation of an impermeable cake layer. Although infiltration facilities are designed to promote and encourage deep bed filtration, Hatt et al. (2008) studied the hydraulic and pollutant removal performance of stormwater filter systems and concluded that the primary cause of hydraulic failure was the formation of the clogging layer at the surface of the filter media.

2.4.2 Straining Filtration

Bradford et al., (2007) describe straining as “the retention of colloids in the smallest regions of the soil pore space formed adjacent to points of grain-grain contact.” Straining filtration is a common particle removal mechanism in depth filters. Under unfavourable attachment conditions, pore spaces play a significant role in particle deposition (Bradford et al., 2007). Early understanding of straining as the mechanical removal of particles in small pore spaces was supported by Herzig et al. (1970) who claimed that straining was purely a geometric process. More recently, solution chemistry and hydrodynamic forces have been found to influence straining (Bradford et al., 2007) and, therefore, both physical and chemical mechanisms may be involved (particularly in colloidal straining). To better model deposition profiles, straining should consider the potential influence of pore structure, grain-grain junctions, and surface roughness (Bradford et al., 2006).

Although both physical and chemical mechanisms can influence straining, the most critical factor that likely determines straining (particularly under stormwater filtration applications) within porous media is the ratio between the collector media diameter ($d_M$) and the suspended particle diameter ($d_P$) (Li and Davis, 2008b; McDowell-Boyer et al., 1986). McDowell-Boyer et al. (1986) suggest that under conditions where the ratio between $d_M$ and $d_P$ is less than 10, or for large deposited particles, it is likely that no penetration into the
media will occur (i.e., the suspended particle is too large for penetration). However, one trial conducted by Li and Davis (2008b) found significant particle removal through the depth of the filter, prior to cake filtration, when $d_{M}/d_{P}$ ranged between 1 and 5. Typically, porous filters have an upper limit of the total mass of deposited particles than can be retained (McDowell-Boyer et al., 1986). This upper limit of retained particles results in a lower permeability limit (McDowell-Boyer et al., 1986). In addition, larger particles that are retained in the porous medium may also act as strainers for smaller inflowing suspended solids (McDowell-Boyer et al., 1986); however, this is more likely for a large range in particle size distribution. Given that particles found in stormwater are typically less than 25 μm (Randall et al., 1982) and are to be infiltrated into granular sand filters with particles ranging from 0.30 to 5.0 mm (Hendricks, 2005), straining is believed to be the driving filtration mechanism and primary cause of physical clogging.

### 2.4.3 Physical-Chemical Filtration

Physical-chemical processes describe the removal of small particles from suspension through physical-chemical forces between solution and the porous media. Particle-particle or particle-media interactions in both depth and surface filters are established by either electrostatic or London-van der Waals forces (McDowell-Boyer et al., 1986). Differences in surface charges (i.e., negatively charged particles and positively charged media or vice versa) determine whether particle-media interactions can occur (McDowell-Boyer et al., 1986).

Attachments between particles and media result when attractive forces overcome repulsive forces upon collision. Typically, particulate pollutants, which are in the order of micrometers or nanometers, (i.e., bacteria, viruses, and asbestos fibers) are removed from inflowing water via physical-chemical filtration mechanisms. Given that the filter at the bottom of the infiltration trench may be comprised of a sand material, physical-chemical interactions are assumed to be relatively minimal in this application compared to straining and are, therefore, not considered as a significant factor in media clogging and subsequent failure of the infiltration trench compared to the influence of particle straining as previously discussed.
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Recall that Hatt et al., (2007a) claim mechanical straining to be the primary particle removal mechanism for sediment removal.

### 2.5 CLEAN BED PARTICLE REMOVAL

The design of a filtration system for particle removal is most often based upon the application of clean-bed filtration models (Logan et al., 1995). Logan et al. (1995) describe clean-bed models as mass balance expressions that consist of a collector particle, a single-collector collision coefficient, and an empirically derived sticking coefficient. These models only consider particle removal efficiencies during the early phase of the filtration cycle, known as the ripening stage, which is characterized by improved filtration performance (Tien & Ramarao, 2008) as deposited particles become collectors. Clean bed filtration theory implies that the media grains are homogeneous and have not collected enough deposited suspended solids to affect the deposition of subsequent deposited suspended solids (Logan, 1999).

Equation (2.6) illustrates the Yao filtration equation which is characteristically used to calculate a steady-state concentration profile under clean-bed conditions (Vigneswaran and Chang, 1986; Logan et al., 1995):

\[
\frac{C}{C_o} = \exp \left[ -\frac{3}{2} \frac{(1-\varepsilon)}{d_M} \alpha \eta Z \right]
\]  

(2.6)

where \( C \) is the particle concentration (M/V) at depth \( Z \) (L); \( C_o \) is the influent particle concentration (M/V); \( \varepsilon \) is the bed porosity (unitless); \( d_M \) is the diameter of filter grain (L); \( \alpha \) is the sticking coefficient describing the fraction of successful attachments; and \( \eta \) is the single collector collision efficiency.

Models such as the Yao equation describe the microscopic behaviour of a filtration system requiring significant input of empirical design values (Vigneswaran and Chang, 1986). Such detailed models are limited to specific applications and are not suitable for a variety of sites.
and filtration designs. The present study aims to move beyond the theory of clean-bed filtration and its limitations by considering the effects of filter clogging on the subsequent deposition of particles.

2.6 CLOGGED BED PARTICLE REMOVAL

Clogging is the physical reduction in permeability caused by the deposition of particulates in porous media (Mays and Hunt, 2005). As the pore spaces in a granular filter become clogged, the flow through the filtration system is reduced and the presence of deposited particles upon subsequent deposition changes the filtration behaviour of the system. Clogged filter media should be addressed in modelling system performance as it may lead to the following changes: increase in effective size of the filter grains, decrease in the effective permeability of the media, continual growth of deposited-particle aggregation in the void space of the media, and formation of filter cake on the surface of the filter medium (Tien and Ramarao, 2007). Numerous existing models that simulate clogging of stormwater filters are not suitable for predictive behaviour (i.e., predicting permeability) due to the structural complexity of colloidal deposits and the difficulty in quantifying the feedback between flow and deposited particles (Mays & Hunt, 2005). Typically, available clogging models become less predictive of filtration performance as particles accumulate within the pore spaces due to the coupling between particle retention and permeability reduction (McDowell et al., 1986). Furthermore, clogging models that have been developed from the Kozeny-Carman equation require many empirical parameters (Mays and Hunt, 2005) which limit their use in specific problems and applications. The Kozeny-Carman equation refers to clean bed headloss and assumes that the porous medium is a bundle of capillaries of equal length and diameter (Tien and Ramarao, 2007). This expression is shown in Equation (2.7) and assumes that flow is laminar and that pressure drop occurs as a result of form-drag loss (Tien and Ramarao, 2007):

\[
\frac{(-\Delta P)}{Z} = k_1 \frac{(1-\varepsilon)^2}{\varepsilon^3} \frac{\mu \cdot u_s}{d_M^2} \quad (2.7)
\]
where \(-\Delta P\) is the pressure drop across the filter (Pa); \(k_i\) is a constant; and \(\mu\) is the fluid viscosity (Pa/T).

O’Melia and Ali (1978) observed the changes in filter permeability and the dependence on surface area within the filter (Mays and Hunt, 2005). In addition, the O’Melia and Ali (1978) clogging model assumed that as the number of deposited particles within a filter bed increased, the bed volume increases accordingly (Mays and Hunt, 2005). It is assumed that the bed volume increased as more particles are deposited since particles accumulate on the collector and proportionally increase the size of the collector particle. The number of particles deposited per filter grain, of a given depth of filter bed, describes the filter performance if deposited particles are allowed to act like collector particles. Equation (2.8) determines the time-dependent quantity of particles deposited per collector particle, calculated from mass balance (Tien and Ramarao, 2007):

\[
\overline{N} = \frac{u C_o \pi d^3}{6 Z (1 - \varepsilon)} \int_0^\theta \left( 1 - \frac{C}{C_o} \right) d\theta
\]

where \(\overline{N}\) is the average of the number of deposited particles per collector; and \(\theta\) is the corrected time for the initial filtration at different depths (T). The filtration time must be corrected to consider the effects of the porous material within the filter (further discussed in Chapter 4).

A rearranged and simplified form of the O’Melia and Ali (1978) clogging model in Equation (2.8) is presented by Mays and Hunt (2005) and was applied for an investigation of particle clogging in porous media:

\[
\overline{N} = \frac{\pi d^3}{6(1 - \varepsilon)} V_p \sigma
\]

where \(V_p\) is the volume per deposited particle (L^3).
Assuming that the bed volume increases with specific deposition of suspended solids, Mays and Hunt (2005) derive Equation (2.10) from O’Melia and Ali (1978):

$$\frac{\Delta H}{\Delta H_o} = \left[ 1 + \beta' \frac{N A_p}{\pi d_m^2} \right]^2$$  \hspace{1cm} (2.10)

where $\Delta H$ is the headloss across the filter (L); $\Delta H_o$ is the clean bed headloss (L); $\beta'$ is an empirical coefficient representing the fraction of retained particles contributing to the increased specific area (Mays and Hunt, 2005) and is a function of fluid velocity; $A_p$ is the surface area per average deposited particle (L$^2$).

Furthermore, by defining a filter bed clogging factor that relates the geometry of the deposited particle to the size of the filter bed media gives a simplified form of a headloss expression in terms of specific deposition of particles (O’Melia and Ali, 1978; Mays and Hunt, 2005; Li and Davis, 2008b):

$$\gamma = \frac{\beta' d_m}{6(1 - \varepsilon)} \frac{A_p}{V_p}$$ \hspace{1cm} (2.11)

where $\gamma$ is the clogging factor (as depicted in Figure 2.1).

Equation (2.11) assumes that finer particles result in higher clogging factors and further clogging throughout the entire depth of the filter. Conversely, large deposited particles do not penetrate the filter and, therefore, do not result in clogging throughout the depth filter. In this study, it is assumed that larger deposited particles are to be removed through upstream pre-treatment mechanisms and through the overlying filter fabric, whereas, finer particles result in clogging of the depth filter.

Equation (2.9) and (2.11) can be combined to derive the following expression (Mays and Hunt, 2005; Li and Davis, 2008b):
Equation (2.12) may also be expressed in terms of hydraulic conductivity by applying Darcy’s Law (Li and Davis, 2008b):

\[
\frac{\Delta H}{\Delta H_o} = (1 + \gamma \sigma_s)^2
\]

(2.12)

where \( K_e \) and \( K_o \) are the effective and clean bed hydraulic conductivities, respectively (L/T).

Mays and Hunt (2005) adjusted the single clogging parameter in Equations (2.12) and (2.13) to fit experimental data from Veerapaneni (1994) with the simplified O’Melia and Ali (1978) model. Results from fitting the model with three sets of experimental data are illustrated in Figure 2.5.

This figure illustrates both slight and severe clogging. Slight clogging is defined by a linear scaling of normalized increase in headloss with an increase in specific deposit. This linear scaling is expressed in the form of \( \Delta H/\Delta H_o - 1 < 10 \) by Mays and Hunt (2005). Similarly,
severe scaling is defined by a transition from linear to quadratic scaling with specific deposit and is taken as $\Delta H/\Delta H_o - 1 > 10$ (Mays and Hunt, 2005). This expression, $\Delta H/\Delta H_o - 1$, is a normalized increase in headloss and represents the transition from linear to quadratic dependence on specific deposition (Mays and Hunt, 2005). As a result, clogging profiles may be non-linear.

2.7 SUMMARY

Chapter 2 discusses the early development of filtration models and highlights recent advancements in stormwater filtration theory. In addition, column experiments conducted by Hatt et al. (2007a) reveal that sediment removal through stormwater filters is not affected by varying wet-dry cycles. Furthermore, macrofiltration modelling equations were presented, which go beyond volumetric storage issues, and consider the influence of filtration layers, urban constituents, and the media depth of solids removal.

Two filtration technologies that are capable of separating particles from suspension were identified: deep bed filters, and surface or membrane filters. Through the application of these two filtration technologies, particle removal is achieved through surface filtration, straining filtration, or physical-chemical filtration. However, Hatt et al. (2007a) identify particle straining to be the primary particle removal mechanism in stormwater applications.
3.1 HYDRAULIC BEHAVIOUR OF STORMWATER FILTERS

As previously discussed, the development of urban stormwater filtration systems is based upon theory from both SSFs and RSFs which are commonly employed in water treatment plant applications. Although there are similarities in filtration behaviour between water treatment plant and stormwater applications, due to dry inter-event periods, the behaviour of stormwater filters is typically more dynamic than water treatment filters, which operate continuously. In addition to dry periods, stormwater filters must be designed to handle the stochastic nature of urban runoff volumes (Urbonas, 2002).

3.1.1 Stormwater Filtration Configuration

Infiltration facilities are designed and implemented to reduce surface runoff directly from a site and redirect flow into the subsurface to recharge the underlying aquifer as depicted in Figure 1.1. The following section investigates in greater detail the existing design recommendations for stormwater filtration systems.

Volume Control

Urbonas and Ruzzo (1986) suggested after a thorough investigation of infiltration practices throughout the United States that the capacity of storage facilities above infiltration sand filters should be sized to capture a runoff volume between the mean and “maximized” runoff volumes (Guo and Urbonas, 1996; Urbonas et al., 1996). Guo and Urbonas (1996) define the maximized runoff volume as the “point where diminishing returns begin to occur in either the long-term runoff volume capture or treatment or in the number of runoff events captured and
treated”. In addition, the drain time should be short enough such that there is a high probability that the storage volume is completely drained prior to the next rainfall event.

In this study, the design configuration of the infiltration trench is based upon the expected mean volume of runoff to be infiltrated and the capacity of the local soil to drain the stored volume of runoff. The design volume specifically refers to the backfilled storage volume overlying the filter. The design area of the trench is derived by applying the continuity expression in Equation (3.1) such that there is no accumulation of runoff in the trench:

\[
V_{in} = V_{out} = V
\]  

(3.1)

where \(V_{in}\) and \(V_{out}\) are the design volumes (L^3) of runoff in and out of the trench, respectively. Note that \(V\) simply represents the design volume of runoff to be infiltrated under steady-state and equilibrium conditions.

Recalling that flow from the trench follows: \(Q = vnA\) and \(Q = V_{out} / \bar{t}\), where \(v\) is the percolation rate of the native soil (L/T), \(n\) is the fraction of voids in the overlying storage media in the trench, \(A\) is the area of the trench (L^2), and \(\bar{t}\) is the retention time (T) (assumed to be the average time between storm events). The void fraction in the trench must be considered since the entire trench storage volume is not available due to the backfilled stone media. The area of the trench is determined as follows (Barr Engineering Company, 2001):

\[
A = \frac{V}{\bar{t}vn}
\]  

(3.2)

In the present study, for the general case scenario, initial design parameters are calculated using mean rainfall statistics taken from Pearson International Airport (Adams & Papa, 2000). In addition, the rainfall volume to be infiltrated is calculated by assuming a catchment area less than 0.9 ha (2 acres) (Barr Engineering Company, 2001). Catchment areas that are too large (i.e., greater than 0.9 ha) may result in early clogging due to a considerable volume of infiltrating urban runoff and associated suspended solids (Barr Engineering Company,
The initial depth of trench is determined based on design guidelines which suggest trench depths range between 0.5 and 3 m (Government of British Columbia, 2006).

**Time to Drain**

As mentioned previously, infiltration trenches should be sized such that there is a high probability that the volume of runoff is drained prior to the next rainfall event. The recommended retention time (as presented in Equation 3.2) to ensure adequate drainage is between 6 and 72 hours (Barr Engineering Company, 2001). The mean rainfall interevent time is recommended as a design detention time for initial infiltration trench sizing.

**Suspended Solids Control**

Typically, an estimate of the mass of solids that are strained from the inflowing fluid over time is determined by using the average volume of runoff per year, the average event mean concentration, and an expected removal efficiency through the filter (Urbonas, 2002). However, this approach requires a detailed and relatively accurate understanding of the expected filter performance which in most cases is unknown or is simply an estimate based on ideal conditions.

Filter performance models are applied to simulate and approximate the expected removal efficiencies herein. This study aims to incorporate the natural variation in mass loadings during different rainfall events rather than applying a single loading rate over a long-term basis to the model. The event mean concentration (EMC) represents the average concentration of pollutant in the runoff volume during one rainfall event. Figure 3.1 illustrates varying EMCs during four different rainfall events.

This study varies the influent suspended solids EMC for each rainfall event based on its probability distribution function (discussed further in Chapter 4).
Emergency By-pass and Drain Valve

If the design capture volume of the facility is exceeded, runoff may be redirected to by-pass the filter and prevent ponding at the surface of the catchment or infiltration facility. This by-pass can be achieved through a flow controlled outlet and may be configured in two ways: to by-pass only the filter, or to by-pass both the storage volume and the filter. Figure 3.2 illustrates both of these configurations.

Figure 3.1 Event Mean Concentration Time Series (Source: Adams & Papa, 2000)

Figure 3.2 (a) By-pass only the filter; (b) By-pass both the storage volume and the filter (Source: after Urbonas, 2002)
Typically, infiltration trenches are designed to manage the storm “first flush” or the first 25 mm of runoff (Barr Engineering Company, 2001). A runoff volume that exceeds this threshold (25 mm) is diverted around the infiltration trench through the use of a by-pass system.

**Pre-treatment Mechanisms**

To reduce the clogging by sediment in infiltration trenches, pre-treatment mechanisms are recommended. Typically, pre-treatment devices remove 25 to 30 percent of infiltrating sediment loads (Barr Engineering Company, 2001). Pre-treatment systems are particularly important in highly urbanized sites with significant sediment loads such as roadways, but are deemed less critical when infiltration trenches are installed to treat rooftop runoff. Common pre-treatment mechanisms include the following: grit chambers, swales with check dams, filter strips, and sediment forebays/traps (Barr Engineering Company, 2001).

In addition to pre-treatment mechanisms, filter fabrics are also recommended to remove sediments to reduce clogging of the subsurface materials. A protective layer of non-woven filter fabric may be located approximately 0.3 metres below the ground surface to reduce the risk of clogging of the underlying storage and filter media (Barr Engineering Company, 2001). During laboratory testing of woven and non-woven filter fabrics, Pitt et al. (2006) determined approximate removal efficiencies of these filters. For a non-woven fabric material (150 μm openings) and a median sediment particle of 20 μm, testing found a removal efficiency of 73 to 99% (Pitt et al., 2006). Filter fabrics may require frequent maintenance to remove captured sediment, depending on sediment loading rates.

A geotextile filter fabric may also be placed along the walls of the trench to reduce the risk of contaminating the adjacent soils. It is important that urban runoff be cleared of as much sediment as possible prior to discharging through the underlying filter to avoid clogging and poor drainage. Maintenance considerations are required to investigate the accumulation of sediment in the infiltration system and the resulting performance.
Maintenance Considerations

To ensure proper performance of infiltration trenches, regular maintenance and inspection regimes are required. Typical maintenance considerations include regular inspections, cleaning of inlets, mowing of grass at observation wells, and removal of sediment from pre-treatment practices (Government of British Columbia, 2006). Upon installation and once the infiltration trench is in operation, inspection of the water level in the trench should take place (via observation wells) after each storm event to ensure proper drainage (Barr Engineering Company, 2001). This regimen should continue for the first several months of operation (Barr Engineering Company, 2001). After the preliminary inspection period, it is recommended that the infiltration trench be monitored twice a year; however, it is preferable to monitor the system each month (Barr Engineering Company, 2001). Regular maintenance events are critical to avoid costly rehabilitation of the trench. Specific maintenance actions include removal of accumulated sediment, leaves and debris from pre-treatment facilities as well as checking for clogging at any inlet and outlet pipes and ponding inside or at the surface of the infiltration trench (Barr Engineering Company, 2001).

Clogging of the surface materials requires that the first layer of stone aggregate as well as the filter fabric be replaced. However, indications of poor drainage through the infiltration trench, such as ponding, require further corrective actions. In the event that clogging at the bottom of the trench has occurred, all stone aggregate, filter fabrics, and sand filters should be replaced (Barr Engineering Company, 2001).

3.2 Failure Mechanisms

Failure to properly maintain the upstream pre-treatment facilities as well as replace the top layer of stone aggregate and filter fabric once early signs of clogging have been identified can lead to failure of the entire infiltration trench.

The present study identified three different failure modes that indicate the need for rehabilitation of the trench, specifically the underlying sand filter layer. In particular, given
that the requirement for major rehabilitation is associated with clogging at the bottom of the trench, this study specifically investigates failure modes associated with the underlying sand filter.

The length of filter run is determined by whichever is achieved first: the designated terminal headloss or the breakthrough limit (Hendricks, 2005) defined by violation of an existing groundwater quality standard. In addition, this study evaluates failure due to overflow events. Figure 3.3 illustrates the length of filter run based on headloss across the filter bed and breakthrough (after Metcalf and Eddy, 2003).

![Figure 3.3](image)

**Figure 3.3** Length of filter run based on headloss and effluent turbidity (Source: after Metcalf and Eddy, 2003)

In depth filtration, the time rate of head-loss increase is linear, whereas surface straining results in an exponential head-loss increase (Hendricks, 2005).

Failure that is attributable to excessive headloss or effluent water quality violation may not occur simultaneously and this occurrence is depicted by the red-dashed effluent quality curves in Figure 3.3. In good filter design, the breakthrough limit should occur first; however, it is often the case that stormwater filters are characterized as being clogging limited rather than breakthrough limited (Li and Davis, 2008b).

The following section describes each failure mode in detail and the specific approach followed in this study to determine each time of failure.
3.2.1 Clogging of Porous Material

The first failure mode entails clogging of the underlying sand filter. Clogging, as discussed in Chapter 2, results when deposited solids occupy the pore spaces between filter grains and results in a decrease in hydraulic conductivity, buildup of hydraulic head, and subsequent decrease in flow through the filter. Figure 3.4 illustrates progressive clogging in the underlying filter medium.

During the early phase of deposition, it is anticipated that solids are removed through particle straining and begin to deposit throughout the depth of the sand filter. However, as the hydraulic conductivity of the filter is reduced, larger solids begin to accumulate on the surface of the filter forming the cake layer. As more suspensions accumulate on the surface of the filter, compression occurs and a low hydraulic conductivity cake layer develops. Subsequently, the flow through the subsurface is reduced from its initial value.

This failure mode poses concern since clogging of the bottom filter material results in a costly rehabilitation of the entire system. In addition, as flow through the filter decreases, drainage between storm events becomes inadequate and this may lead to larger and more frequent overflow events.
3.2.2 Overflow or Spills

The second mode of failure is the overflow of stored runoff at the surface of the trench. Poor siting or design of infiltration trenches can lead to overflow events if the trench is not adequately designed to hold and/or drain significant volumes of runoff. In addition to poor siting, gradual clogging of the trench can lead to increased frequency of overflow events as well as ponding at the surface of the trench. Ponding at the trench surface can be minimized with an added drain at the bottom of the stone aggregate, and above the sand filter, to remove excess water that is partially treated once the drainage capacity of the sand filter has been compromised by clogging. Overflow events are simulated by modelling the dynamic changes of the free water surface within the trench in response to rainfall events and dry periods between events. An overflow event is defined once the free water surface surpasses the depth of the trench (including the overflow berm).

Overall, this failure mode poses concern since spilling of untreated water results in the discharge of potentially harmful pollutants into nearby surface waters, threatening the overall health of the surrounding environment, inhabiting species, and local residents.

3.2.3 Groundwater Contamination

The final failure mode identified in this study is the exceedence of groundwater quality standards by contaminants of concern. Infiltration trenches are designed to remove suspended solids as well as associated contaminants. The use of filter media in infiltration trenches is more suited for removal of contaminants in particulate form rather than the effective removal of soluble fractions (DeBusk et al., 1997). In addition, dissolved pollutants pose a significant threat to the environment as they are transported in the groundwater flow. This study specifically investigates the potential threat that dissolved contaminants pose to the underlying groundwater quality.

Typically, in the event that a contaminant breakthrough occurs and the concentration in the groundwater exceeds the *Soil, Ground Water and Sediment Standards for use Under Part*
3.3 MODELLING SUSPENDED SOLIDS REMOVAL IN STORMWATER FILTERS

Despite the application of pre-treatment facilities, infiltration systems continue to clog (Bouwer, 2002). As discussed in Chapter 2, several carefully conducted experiments have confirmed the development of the clogging layer and the gradual reduction in permeability in the surface layers of the stormwater filter. To achieve a representative prediction of this failure mode during the operational phase of the infiltration trench, consideration of clogging in the mathematical model is required. However, few predictive models have simulated this behaviour in stormwater applications.

3.3.1 Existing Models

The following section investigates a variety of modelling approaches commonly used to simulate stormwater infiltration devices. This summary of models is not exhaustive but is intended to provide a general overview of the types of filtration models that have been applied to stormwater infiltration devices.

Infiltration Processes

Infiltration models such as the Green-Ampt equation predict the cumulative volume of infiltrated runoff in the subsurface and are a function of time and soil type (Bedient and
Chapter 3. Urban Stormwater Filtration

Huber, 2002). Application of this model requires input parameters such as hydraulic conductivity, porosity, and the wetting front soil suction head (Bedient and Huber, 2002).

Darcy’s Law is another infiltration model that describes the movement of water through a homogeneous porous medium by considering the saturated hydraulic conductivity, cross-sectional area through the porous medium perpendicular to the flow, and the hydraulic gradient. In this model, flow is determined by the difference in total head between two points in a soil mass (Ferguson, 1994).

Although models such as the Green-Ampt infiltration equation and Darcy’s equation are commonly applied to filtration applications, these models assume that clean bed conditions are maintained and a clogging layer is not formed (Le Coutumier and Barraud, 2007). Such infiltration models are not recommended as a stand alone method in this study since they do not address the transport and deposition of suspended solids and their subsequent impact on the permeability throughout the filter bed.

Trajectory Models

The removal efficiency of suspended particles through porous media may also be determined through particle trajectory analyses. Trajectory analyses predict the movement of suspensions influenced by forces that are in dynamic equilibrium (Darby et al., 1992). The removal efficiency of suspended particles is determined with a trajectory analysis combined with suitable geometric models for a porous medium (Darby et al., 1992). Although experiments that were carefully conducted to determine initial removal efficiencies (e.g., Fitzpatrick and Spielman, 1973; Chiang and Tien, 1985a,b; Rajagopalan and Tien, 1977; Tobiason and O’Melia, 1988; Ghosh et al., 1975 as cited in Darby et al., 1992) indicated the success of particle trajectory analyses, this approach is challenging to solve due the complexity of coupling forces, geometry and boundary conditions (Darby et al., 1992).

Network Models

More recent random network models have investigated permeability reduction in filter beds based on changes in geometry such as the constricted tube model (Mays and Hunt, 2005)
which was first discussed almost 30 years ago (Tien and Ramarao, 2007). In constricted-tube geometry, a porous medium is described as a collection of pore spaces that are connected by constrictions. The basic flow channel through the porous medium consists of two half pores that are joined by a constriction and are aligned along the direction of the main flow (Payatakes et al., 1973). Although the constricted tube model realistically describes the process of filter clogging, geometry-based models are primarily used for interpreting and correlating experimental data (Tien and Ramarao, 2007) and may be exceedingly complex for design applications.

**O’Melia and Ali Clogging Model**

Several depth filtration studies have been performed using laboratory column experiments to derive equations that appropriately describe the performance of depth filters and clogging models. O’Melia and Ali (1978) were responsible for the early development and application of clogging models and were able to capture the increase in media clogging within the upper portion of the filter bed. Semi-empirical models initially developed by O’Melia and Ali predict the transient stage removal under macroscopic considerations; modifications have been made to these models over the years (Jagatheesan and Vigneswaran, 2005). This approach assumes that particle removal is due to filter grains and particles that are already deposited onto the filter (Jagatheesan and Vigneswaran, 2005).

**3.3.2 Macroscopic vs. Microscopic Models**

The accumulation or removal of suspended solids in porous media may be determined via microscopic or macroscopic models. Generally, microscopic models make use of trajectory analyses or convective diffusion equations (Jagatheesan and Vigneswaran, 2005) to determine initial particle removal efficiencies and assume favourable filtration conditions. Semi-empirical formulas are applied to determine the initial removal efficiencies under unfavourable conditions (Jagatheesan and Vigneswaran, 2005). Favourable conditions transpire if particle and filter media charges are opposite (Jagatheesan and Vigneswaran, 2005). Typically, microscopic models deal with calculations at the pore-scale, whereas macroscopic models treat the fluid and solid matrix together as a continuum, which is
characterized by macroscopic properties including but not limited to porosity, permeability, and filter grain size (Tien and Ramarao, 2007).

The present study applies depth filter models derived by Tien and Ramarao (2007) but does not emphasize the geometric changes in the collector media due to clogging. Modelling the details of morphological changes within a filter bed due to specific deposit can be demanding and require many empirical input parameters. A macroscopic modelling approach minimizes the number of input parameters as well as the complexity of the calculations. Mays and Hunt (2005) used clogging models modified from the original O’Melia and Ali (1978) clogging models. The modified Mays and Hunt (2005) clogging model, as presented in Equations (2.8) to (2.11), is applied in this study to determine the long-term performance of infiltration trenches. Further details on the macroscopic modelling approach are discussed in Chapter 4.

3.4 SUMMARY

In Chapter 3, a brief overview of stormwater filtration configuration was provided which included an investigation of the following design considerations: design volume control, time to drain, suspended solids control, an emergency by-pass and drain valve, pre-treatment mechanisms, and maintenance considerations. Failure to properly maintain upstream and pre-treatment facilities as well as the top layer of stone aggregate and filter fabric may result in failure of the entire infiltration trench. In the event of improper long-term maintenance of infiltration trenches, three failure mechanisms were identified in this chapter including: clogging of the underlying porous material, overflow and/or spills, and contamination of the underlying groundwater.

After an investigation of existing modelling approaches commonly used to simulate stormwater infiltration devices, a macroscopic modelling approach was selected to determine the long-term performance of infiltration trenches. Macroscopic modelling approaches minimize the number of input parameters as well as the complexity of the selected mathematical expressions.
4.1 MODELLING GRANULAR FILTRATION

Partial differential equations (PDEs) are used to express the changes in mass concentration across a filter, given that influent mass varies in time and space. The temporal and spatial removal of suspended solids in granular sand filters is governed by the following factors (Metcalf and Eddy, 2003):

- Grain size of the filter medium;
- Rate of infiltration;
- Influent particle size distribution; and
- Floc strength (when floc is present).

Suspended solids are used as an indicator pollutant for many urban water quality models and are the focus of the filtration model described in the present study. In urban centres, suspended particulates are frequently generated and deposited onto impermeable surfaces through atmospheric deposition or by automobile traffic.

4.1.1 Comprehensive Macroscopic Equations

The accumulation of suspended solids in the filter bed deteriorates particle removal efficiencies. A modelling approach that considers how fluid flow is affected by the presence of deposited suspended solids is applied. This study attempts to address the transient stages of deep bed filtration that consider particle accumulation when clean bed filtration conditions are no longer a valid assumption. An analytical solution is derived in Tien and Ramarao (2007) and was developed from Iwasaki’s (1937) mathematical filtration expressions, also
cited in Tien and Ramarao (2007), that model particle concentration in the suspension and is presented in Equation (4.1). The concentration profile is given as:

\[
\frac{C}{C_o} = \frac{\exp[u_o \lambda_o C_o k \theta]}{\exp[\lambda_o Z] + \exp[u_o \lambda_o C_o k \theta] - 1} \tag{4.1}
\]

where \( k \) is a positive empirically derived parameter (unitless) and \( \theta \) is a “corrected time” (T) for granular filtration and is calculated using Equation (4.2) (Tien and Ramarao, 2007):

\[
\theta = t - \int_0^Z \frac{\varepsilon}{u_s} \, dz \tag{4.2}
\]

The filtration time must be corrected to account for the porous media within the filter. The distance traveled by a given deposited particle is influenced by the porous material within the filter; therefore, the filter depth is believed to be an underestimate of the actual traveled distance. Equation (4.1) assumes that the deposition of suspended solids in the filter principally results in filter clogging (Ornatski et al., 1955 as cited in Tien and Ramarao, 2007). Degradation of the filter bed is considered by using a filter bed coefficient, \( \lambda \), which is a function of specific deposition (Iwasaki, 1937; Stein, 1940 as cited in Tien and Ramarao, 2007) as follows:

\[
F = 1 - k\sigma_v \tag{4.3}
\]

where \( F=\lambda/\lambda_o \). This empirically derived equation is similar to Equation (2.3); however, it assumes that \( k=1/\varepsilon_o \) (Shekhtman, 1961; Heertjes and Lerk, 1967 as cited in Tien and Ramarao, 2007) and that the possibility of further attachment of suspended solids onto the media grains decreases as a function of solid deposition. Note that \( \varepsilon_o \) is the clean bed porosity.

The influent and effluent concentrations of solids in the suspension are calculated at incremental depths within the filter as illustrated in Figure 4.1.
The specific deposition of solids at incremental depths throughout the filter is calculated using Equation (4.4), which is an overall mass balance equation presented by Tien and Ramarao (2007):

\[
\bar{\sigma} = \frac{1}{Z} \int_{0}^{Z} u_s (c_o - c) dz
\]  

(4.4)

where \( \bar{\sigma} \) (cm\(^3\)/cm\(^3\)) is the average particle deposition within a filter bed with a depth, \( Z \).

Considering a time increment \( \Delta t \), the average specific deposition of solids (cm\(^3\)/cm\(^3\)) at any time within a small filter slice can be developed as:

\[
\bar{\sigma}_{zj} = \frac{u_s (c_{z-1} - c_z) \Delta t}{z \rho_s} + \bar{\sigma}_{z,j-1}
\]  

(4.5)

Tien and Ramarao (2007) confirm that the assumption of uniform deposition becomes more valid as the filter depth decreases. Equation (4.1) incorporates a term (filter bed coefficient) that accounts for the degradation of filtration performance in the filter as the number of deposited particles increases. Based on a thorough investigation, it is believed that this analytical solution underestimates the degree of clogging and development of the cake layer within the upper layers of the filter.
To achieve a hydraulic conductivity profile similar to the results presented by Li and Davis (2008b), Equation (4.5) is used to establish a long-term rate of specific deposition. This long-term rate represents the cumulative mass loading of particles on the filter with the assumption that once the maximum capacity for the filter to retain particles is reached, subsequently deposited particles will further clog the filter, and form a cake layer on its surface. For instance, Figure 4.2 shows the cumulative deposition of suspended solids within the depth filter and Figure 4.3 shows the subsequent decrease in hydraulic conductivity based on a constant loading of suspended solids (EMC = 150 mg/L), similar to the profile depicted by Li and Davis (2008b).

\[ \text{Cumulative Specific Deposition} (\text{kg/m}^3) \]

**Figure 4.2** Specific deposition under clogging and non-clogging conditions

In Figure 4.2, two clogging conditions are shown: one where clogging is limited by the maximum capacity of the filter to retain suspended solids (blue curve), and a second condition where the rate of specific deposition is projected, under a linear assumption, to portray the long-term clogging and development of the cake layer (red curve). The maximum specific deposition is governed by the filter media properties. Considering the simulation shown in Figure 4.2, the maximum specific deposition for a filter bed with porosity of 0.50 is approximately 0.50 kg/m\(^3\). A higher capacity for a filter bed to retain particles is achieved by increasing the porosity.
The simulated time series in Figure 4.2 assumes that the deposition of sediment and subsequent clogging is only achieved during rainfall events. It is believed that interevent rainfall periods have minimal influence on deposition or clogging and, therefore, the simulated time series does not consider deposition during these dry periods. As a result of omitting deposition during dry periods, a moderate level of noise is observed in the long-term deposition profile.

The long-term rate of specific deposition is estimated by projecting the early removal efficiency of suspended solids by fitting a linear relationship: \( y = mx + b \). A constant rate of suspended solids removal was also applied in Li and Davis (2008b) to achieve a long-term reduction in hydraulic conductivity. This study uses Li and Davis’ (2007b) linear relationship as the methodology to arrive at long-term clogging conditions. From the profile in Figure 4.2, the fitted intercept \( b \) is slightly less than zero; this is the result of the moderate amount of noise observed in the profile. The projection of solids deposition over an extended period of time leads to a hydraulic conductivity profile (depicted in Figure 4.2) that approaches \( K/K_0 = 0 \) asymptotically as time tends to infinity (similar to Figure 2.1).

![Figure 4.3 Reduction in hydraulic conductivity due to clogging conditions](image)

The hydraulic conductivity is calculated using Equations (2.8) and (2.11) assuming initial filtration parameters, infiltration trench parameters, and clogging parameters. These initial values are summarized in Table 4.1.
4.1.2 Macroscopic Input Parameters

The input parameter values listed in Table 4.1 were obtained from the literature pertaining to stormwater filtration and were selected based on acceptable and recommended ranges.

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Filtration Parameters</strong></td>
<td></td>
</tr>
<tr>
<td>Filter depth</td>
<td>20 cm</td>
</tr>
<tr>
<td>Superficial velocity</td>
<td>1.5 cm/hour</td>
</tr>
<tr>
<td>Filter porosity</td>
<td>0.50</td>
</tr>
<tr>
<td>Clean bed filter coefficient</td>
<td>0.50</td>
</tr>
<tr>
<td>Particle density (deposited particle)</td>
<td>1.050 g/cm³</td>
</tr>
<tr>
<td>Clean bed hydraulic conductivity</td>
<td>20 cm/hour</td>
</tr>
<tr>
<td>Filter cake porosity</td>
<td>0.40</td>
</tr>
<tr>
<td><strong>Trench Design Parameter</strong></td>
<td></td>
</tr>
<tr>
<td>Catchment area</td>
<td>0.5 ha</td>
</tr>
<tr>
<td>Surrounding soil percolation rate</td>
<td>1.5 cm/hour</td>
</tr>
<tr>
<td>Trench void space</td>
<td>0.40</td>
</tr>
<tr>
<td>Trench area</td>
<td>90 m²</td>
</tr>
<tr>
<td>Trench depth</td>
<td>2 m</td>
</tr>
<tr>
<td><strong>Clogging Parameter</strong></td>
<td></td>
</tr>
<tr>
<td>Collector particle diameter</td>
<td>2.0 mm</td>
</tr>
<tr>
<td>Deposited particle diameter</td>
<td>15 μm</td>
</tr>
<tr>
<td>Clogging factor</td>
<td>~200</td>
</tr>
</tbody>
</table>

Regarding filtration parameters, a preliminary filter depth, 20 cm, and superficial velocity, 1.5 cm/hour, were based on recommended values from the design criteria for infiltration trenches in the *Minnesota Small Urban Sites BMP Manual* (Barr Engineering Company, 2001). The initial porosity of the sand filter, 0.50, is a typical value for clean uniform sand (Holtz and Kovacs, 1981; Ferguson, 1994). Next, the clean bed filtration coefficient was
initially assumed to be 0.50 which falls within the range of values (approximately 0.3-0.8) calculated by Li and Davis (2008b). The clean bed hydraulic conductivity, 20 cm/hour, is in agreement with values suggested by the United States Department of Agriculture (USDA) soil class for sand media under saturated soil conditions (Ferguson, 1994). The density of the suspended particle, 1.050 g/cm\(^3\), falls within the range of particle densities reported by Hendricks (2005). Lastly, the filter cake porosity, 0.40, was selected based on results from Li and Davis (2008b) and is associated with the development of a silt layer on the surface of the granular filter media. This preliminary cake porosity was selected to account for compaction of the cake layer resulting from pressure of the overlying volume of stored runoff.

The trench design parameters were based upon the *Minnesota Small Urban Sites BMP Manual (2001)* and were estimated using Equations (3.1) and (3.2) as a guide. The surrounding soil percolation rate was selected to be 1.5 cm/hour which satisfies the minimum recommended exfiltration rate from the trench. The initial selection of a larger percolation rate greatly underestimated the size of the infiltration trench, which results in accelerated failure events. Therefore, a smaller value was selected as a starting point for the design of the infiltration trench and illustration of the modelling methodology.

Randall et al. (1982) indicated that over 80% of particles found in stormwater runoff are less than 25 \(\mu\)m; therefore, a mean deposited particle size of 15 \(\mu\)m was selected based upon such studies which suggest the majority of particles in stormwater runoff are predominately fine. However, given that conflicting studies indicate that deposition from street surfaces is coarse (Sartor and Boyd, 1972; Shaheen, 1975; Sansalone et al., 1998), a mid-range deposited particle size was selected to avoid any biases associated with selecting extremely small or large particles. It is also understood that the grain size distribution of deposited particles is highly dependent on the specific site. A uniform collector particle size of 2.0 mm was based upon the recommended range of granular bed particle sizes reported by Hendricks (2005) and represents a mid-range size sand particle. Finally, the initial clogging factor was calculated using Equation (2.8). A calculated clogging factor of approximately 130 (\(d_{\text{m}}/d_p > 10\)) results in penetration of the filter where fine urban particles become trapped in the interstices of the porous media throughout the depth of the filter rather than surface particle removal. It is
assumed that the larger sediment particles which result in a clogging factor less than 10 and are typically trapped on the surface of the filter (surface filtration) are likely to have been previously removed by upstream pre-treatment mechanisms. Therefore, particles that penetrate the sand filter primarily experience depth filtration.

The sensitivity of various filtration and design parameters are evaluated in Chapter 6 and 7.

4.2 CONVENTIONAL CAKE FILTRATION THEORY

Mays and Hunt (2005) developed a constitutive equation for filter cake performance to depict the evolution of deposit permeability as filtration occurs; however, this method was developed separately from depth filtration.

Cake and depth filtration are jointly considered in Equation (4.6) which is a simplified analytical expression that was derived from the differential equation for mass balance and describes the partitioning of deposited solids amongst the various filter bed layers (Li and Davis, 2008b):

\[ M = Z_D A \sigma + (1 - \varepsilon_c) \rho_s AZ_C \]  

(4.6)

where \( M \) is the cumulative captured mass; \( Z_D \) is the sand filter depth (not including cake layer) (L); \( \varepsilon_c \) is the porosity of the filter cake layer; and \( Z_C \) is the time dependent cake layer depth (L).

Equation (4.6) is rearranged to solve the evolving depth of the cake layer. Equation (4.7) estimates the filter cake growth as solids deposit, as depicted by \( Z_C \) (Li and Davis, 2008b):

\[ Z_C = \frac{M - Z_D A \sigma}{A(1 - \varepsilon_c) \rho_s} \]  

(4.7)
At time = 0, when the filter bed is void of deposited suspended solids, there is no development of the filter cake layer. It is only after some period of time, with the deposition of solids, that some depth of filter cake is formed. Figure 4.4 illustrates the evolution of the filter bed as suspended solids penetrate through and collect on the surface of the filter through particle exclusion.

\[ Z_T = \frac{Z_C}{K_C} + \frac{Z_D}{K_D} \]  

(4.8)

where \( Z_T \) is the total depth of both the cake layer and the depth filter (L); \( K_e \) is the equivalent hydraulic conductivity associated with the total depth (L/T); \( K_C \) is the hydraulic conductivity of the cake layer (L/T); and \( K_D \) is the hydraulic conductivity of the depth filter (L/T).

Li and Davis’ (2008b) approach to estimate the hydraulic conductivities for each layer within the filter bed considers a “working zone” at the top of the filter bed which is assumed to capture 95 percent of the deposited solids. A pristine zone at the bottom of the filter remains
void of deposited suspended solids. The modelling approach applied in this study did not separate the sand filter into a working zone and a clean or pristine filter zone since clogging is expected to occur in the first 5 to 20 cm (Li and Davis, 2008b; Hatt et al., 2008) and the sand filter in the infiltration trench is expected to be less than 30 cm. Division of the filter into a working zone and pristine zone is more suited for the assessment of bioretention filters since these filters tend to approach greater depths of 1 metre (Li and Davis, 2008b).

4.3 MODELLING METHODOLOGY

The macroscopic filtration modelling approach presented above is used to address the first research objective outlined in Chapter 1 which is to understand the physical clogging and resulting failure of BMP filter beds on a long-term basis. The following section expands on commonly accepted macroscopic theory to establish an appropriate modelling approach that specifically characterizes all three modes of infiltration trench failure.

4.3.1 Meteorological Behaviour

A long-term rainfall series was simulated using rainfall statistics for the meteorological station at Toronto International Airport commonly referred to as Lester B. Pearson Airport. Rainfall statistics correspond to the interevent rainfall period, event rainfall duration, and event rainfall intensity which follow exponential probability distributions. Meteorological characteristics for the exponential probability density function (PDF) statistics are summarized in Table 4.21 (Adams and Papa, 2000).

These rainfall statistics are based upon an hourly rainfall database from the Atmospheric Environment Service (AES), now Meteorological Services Canada (MSC), of Environment Canada. Statistics are derived from a 2-hour interevent time definition, which is the minimum temporal separation between two discrete storm events (Adams and Papa, 2000). A shorter interevent time definition was selected due to the catchment characteristics which consist of a small storage reservoir and a relatively small urban catchment (Adams and Papa,
2000). In addition, Adams et al. (1986) suggest that interevent time definitions should be between 1 and 6 hours for urban catchments.

Table 4.2 Rainfall statistics for a meteorological station at Pearson Airport, Ontario

<table>
<thead>
<tr>
<th>Rainfall Parameter</th>
<th>1/Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interevent Time, $\beta$ (hr$^{-1}$)</td>
<td>0.758</td>
</tr>
<tr>
<td>Rainfall Duration, $\lambda$ (hr$^{-1}$)</td>
<td>0.282</td>
</tr>
<tr>
<td>Rainfall Intensity, $\psi$ (hr/mm)</td>
<td>0.0230</td>
</tr>
</tbody>
</table>

A long-term simulated rainfall series, as presented in Figure 4.5 was achieved by randomly sampling each rainfall event characteristic from its respective exponential PDF. Although rainfall volume and duration show some degree of statistical dependence, meteorological characteristics in this simulation are treated as statistically independent (Charles Howard and Associates, 1979; Adams et al., 1986).

Figure 4.5 shows a long-term rainfall series based on a single Monte Carlo simulation. Since this study assumes that dry periods do not contribute to further clogging the underlying filter, the interevent rainfall events (i.e., when rainfall intensity is zero) were removed from the long-term rainfall series in Figure 4.5. The simulated rainfall series merely shows periods with rainfall intensities greater than zero.

Shown in the simulated long-term rainfall series, small rainfall intensities of less than 2 mm/hour occur most frequently, whereas, extreme rainfall intensities occur much less frequently.
4.3.2 Clogging Profile

As discussed in Chapters 2 and 3, clogging of the trench subsurface may be characterized by hydraulic conductivity, headloss, and flow rate reduction through the underlying filter. The assessment of clogging in this study is accomplished through an investigation of flow rate reduction due to the mechanical straining of suspended solids. A filter is characterized as “clogged” once the flow through the subsurface is approximately 10% of the initial outflow. This is the characterization of a clogged filter followed by Hatt et al. (2007b) while conducting a series of experiments to determine filter performance in a controlled environment.

Flow through the base of the trench is calculated with respect to time. However, prior to determining the outflow, the development of total headlosses at both the top of the sand filter and the top of the filter cake are determined, as shown in Figure 4.6. The datum is taken as \( H_0 \), whereas \( H_1 \) is the hydraulic head at the top of the sand filter, and \( H_2 \) is the hydraulic head at the top of the filter cake.
As a preliminary estimate, saturated soil conditions are assumed. From a clogging perspective, as discussed in Section 2.2, this assumption is deemed reasonable since dynamic wet-dry cycles are not believed to have a significant influence on sediment removal within the filter (Hatt et al., 2007a). In addition, given that mechanical straining is the primary cause of clogging (Hatt et al., 2007a), it is believed that further clogging will not transpire during the dry soil conditions between rainfall events. Darcy’s Law is used to calculate the headloss while assuming the superficial filtration rate remains constant as follows:

$$q = K \frac{dH}{dz}$$  \hspace{1cm} (4.9)

where $H$ is the headloss development ($H_1$ and $H_2$). It is noted that $q/\varepsilon = v_p$, and $u_s/\varepsilon = v_p$, where $v_p$ is the pore velocity and $q$ is the infiltration rate. Therefore, the infiltration rate may be approximated as the superficial velocity ($q \approx u_s$).

An approximate solution to Equation (4.9) is used to determine both $H_1$ and $H_2$ in Equation (4.10) and (4.11), respectively:

$$H_1 = u_s \frac{1}{K_D} Z_D + H_o$$  \hspace{1cm} (4.10)

$$H_2 = u_s \frac{1}{K_c} Z_C + H_1$$  \hspace{1cm} (4.11)

In this study, flow reduction results from the rising hydraulic headloss associated with specific deposition within the porous media. The filter layer with the smallest hydraulic
conductivity is treated as a “controlling layer” for flow reduction. Early in the simulation, the sand filter is treated as the controlling layer. The cake layer is only treated as the controlling layer in the model once formation of this layer becomes significant since it is expected to have a much lower permeability than the underlying sand filter. Thus, as time proceeds, the following relationship describes the temporal reduction in flow:

$$Q_i < Q_{i-1}$$  \hfill (4.12)

where $Q$ is the flow through the bottom of the trench (L$^3$/T) across a time interval $\Delta t$.

Subsequently, Equation (4.13) profiles the flow reduction due to rising hydraulic headloss through the controlling layer (layer with the lowest hydraulic conductivity):

$$Q_i = Q_{i-1} - \eta A_i \left[ \frac{H_i - H_{i-1}}{\Delta t} \right]$$  \hfill (4.13)

where $H_i$ and $H_{i-1}$ are the total hydraulic heads (L) in the controlling layer across a time interval $\Delta t$, $A_i$ is the effective filtration area of the trench changing through time $t$, and $\eta$ is the fraction of voids since the entire volume of the trench is not available for storage due to the stone fill.

To ensure an accurate and conservative estimate of flow reduction, as the effective filtration area gradually decreases due to filling of the void spaces with deposited particles, the cross-sectional area of the porous medium is recalculated at each time step using the flow from the previous time step. The gradual filling of the void spaces within the underlying sand filter also results in a gradual reduction in porosity. Porosity at each time step is expressed as a function of specific deposition (Tien and Ramarao, 2007); however, the porosity of the deposited suspended solids within the filter is neglected in this analysis (Li and Davis, 2007b):

$$\varepsilon_i = \varepsilon_0 - \sigma_v$$  \hfill (4.14)
where $\varepsilon_t$ is the porosity at time, $t$. The change in porosity is calculated to simulate the gradual increase in pore velocity (recall $u/\varepsilon = v_p$).

Once development of the cake layer is observed, the cake layer porosity is used in Equation (4.14) rather than the porosity of the sand filter. This study assumes that once the maximum capacity to retain particles in the filter bed is reached (shown in Figure 4.2), particles begin to accumulate on the surface of the filter, forming the cake layer.

### 4.3.3 Mass Balance

The frequency and magnitudes of overflow events are modelled by considering both the dynamic changes in flow through the bottom of the trench as well as the volume of runoff per rainfall event and the dry period between storm events. The runoff volume directed into the trench depends on the runoff coefficient, the catchment area, and the event rainfall intensity. The inflow during a single rainfall event is calculated by applying the rational formula:

$$Q_i = CiA_c$$  \hspace{1cm} (4.15)

where $Q_i$ is runoff rate that flows from the catchment into the trench for a rainfall event (L$^3$/T); $i$ is the average rainfall event intensity (L/T); and $A_c$ is the area of the catchment (L$^2$).

In this study, a runoff coefficient of 0.60 is used which is a typical value for a residential area with a return period of 2 to 10 years (Bedient and Huber, 2002). An inflow into the trench is calculated for each rainfall event.

The volume of storage in the infiltration trench at each time step is calculated by simulating the dynamic changes at the free water surface. The mass balance equation (Equation (4.16)) is expressed in finite difference form (Equation (4.17)) to solve for the changing water level in the trench:
CHAPTER 4. MACROSCOPIC FILTRATION MODELLING APPROACH

\[ \frac{dS}{dt} = Q_i - Q_o, \]  \hspace{1cm} (4.16)

where \( \frac{dS}{dt} \) is the change in storage volume in the trench with respect to time and

\[ h_t = \frac{(Q_i - Q_o)(\Delta t)}{\eta A} + h_{t-1}, \]  \hspace{1cm} (4.17)

where \( h_t \) and \( h_{t-1} \) are hydraulic depths measured from the free water surface to the bottom of the trench.

Once the free water surface surpasses the depth of the trench, including the overflow berm, an overflow event is identified. Infiltration trenches are generally designed to accommodate the “first flush” or up to the first inch of rainfall per event (~25 mm) (Barr Engineering Company, 2001). Therefore, runoff volumes that exceed this threshold of 25 mm by-pass the infiltration trench to prevent exceedence of the storage capacity. In addition, in a situation where the capacity of the trench is exceeded due to a declining drainage capacity through the subsurface, an additional drainage valve may remove partially treated runoff to prevent ponding at the surface of the trench.

4.3.4 Contaminants of Concern

Estimates of the influent concentrations of various contaminants associated with suspended solids may be calculated using correlation coefficients. Correlations among contaminants commonly found in urban runoff are provided in Adams and Papa (2000). Cases with a strong positive correlation between the contaminant and total suspended solids were used to estimate levels of the associated contaminants. As a preliminary evaluation, these concentrations were compared to available groundwater standards such as the Soil, Ground Water and Sediment Standards for use Under Part XV.1 of the Environmental Protection Act (Ministry of the Environment, 2007). This approach indicates which contaminants have the potential to threaten the underlying groundwater. An estimate of the contaminant load of potentially threatening species through the base of the filter is estimated by (Adams and Papa, 2000):
CHAPTER 4. MACROSCOPIC FILTRATION MODELLING APPROACH

\[ L_j = \sum_{t=1}^{n} Q_t C_{jt} \]  \hspace{1cm} (4.18)

where \( L_j \) is the annual load of contaminant \( j \) passing through the underlying filter, \( Q_t \) is the filter flow at time \( t \), and \( C_{jt} \) is the concentration of contaminant \( j \) entering the trench at time \( t \).

Table 4.3 summarizes the groundwater contaminant data as well as correlations among various stormwater contaminants and total suspended solids.

Table 4.3  Soil, Ground Water and Sediment Standards for use Under Part XV.1 of the Environmental Protection Act (Ministry of the Environment, 2007) and correlation data (Adams & Papa, 2000)

<table>
<thead>
<tr>
<th>Contaminant</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Correlation (with TSS)</th>
<th>Groundwater Standards</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Deviation</td>
<td></td>
<td>Background</td>
</tr>
<tr>
<td>TSS</td>
<td>133</td>
<td>116</td>
<td>1.0</td>
<td>-</td>
</tr>
<tr>
<td>Aluminum</td>
<td>1.37</td>
<td>1.46</td>
<td>0.67</td>
<td>-</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.13</td>
<td>0.08</td>
<td>0.58</td>
<td>0.042</td>
</tr>
<tr>
<td>Iron</td>
<td>3</td>
<td>2.65</td>
<td>0.70</td>
<td>-</td>
</tr>
<tr>
<td>TKN</td>
<td>2.84</td>
<td>2.24</td>
<td>0.77</td>
<td>-</td>
</tr>
<tr>
<td>Total Phosphorous</td>
<td>0.62</td>
<td>0.45</td>
<td>0.74</td>
<td>-</td>
</tr>
<tr>
<td>Copper</td>
<td>0.04</td>
<td>0.02</td>
<td>0.60</td>
<td>0.0086</td>
</tr>
</tbody>
</table>

*aesthetic water quality objective
Units in mg/L

The highest correlations were identified between total suspended solids (TSS) and heavy metals and total kjeldahl nitrogen (TKN). The contaminants listed in Table 4.3 were estimated by linearly relating the correlated contaminant to the total suspended solids.

The slope, \( m \), and intercept, \( b \), of the linear relationship are estimated by using the Pearson Product-Moment Correlation Coefficient:

\[ r = \frac{\sum_{i=1}^{n} (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_{i=1}^{n} (x_i - \bar{x})^2} \sqrt{\sum_{i=1}^{n} (y_i - \bar{y})^2}} \]  \hspace{1cm} (4.19)
where $x_i$ and $y_i$ are the values of variables $x$ and $y$ in the $i^{th}$ observation, $\bar{x}$ and $\bar{y}$ are the mean values of variable $x$ and $y$ and $n$ represents the total number data points in the sample. Correlation coefficients presented in Table 4.3 are given in Adams and Papa (2000).

Typically, the slope between two data sets is calculated using Equation (4.20), however, considering the correlation coefficient and standard deviations of each data set, a simplified expression, Equation (4.21), is used:

$$m = \frac{\sum_{i=1}^{n} (x_i - \bar{x})(y_i - \bar{y})}{\sum_{i=1}^{n} (x_i - \bar{x})^2}$$  \hspace{0.3cm} (4.20)

$$m = r \frac{S_y}{S_x}$$  \hspace{0.3cm} (4.21)

where $S_x$ and $S_y$ are the standard deviations of variables $x$ and $y$, respectively.

Lastly, the intercept is calculated by using the slope between each data set and the mean of each sample and is as follows:

$$b = \bar{y} - m\bar{x}$$  \hspace{0.3cm} (4.22)

Given the known slope, intercept, and mean concentration of total suspended solids, an estimate of correlated contaminants may be calculated to determine which species pose the greatest threat to the underlying groundwater system.

4.4 **Boundary Conditions**

Application of the macroscopic equations presented in Section 4.1 must first consider the following initial boundary conditions as outlined in Tien and Ramarao (2007):

1. $C=C_0$ at the surface of the filter, $Z=0$, and when $\theta>0$;
2. \( C=0 \) and \( \sigma=0 \) for any depth \( Z \geq 0 \) and when \( \theta \leq 0 \);

3. Long-term rainfall series begins with a randomly sampled interevent time;

4. \( \varepsilon=\varepsilon_0 \) for any depth \( Z \geq 0 \) and when \( \theta \leq 0 \);

### 4.5 Long-Term Computer Simulation

To minimize the likelihood or frequency of each failure mode, an iterative approach is followed.

Figure 4.7 presents a flowchart of the iterative process to improve the performance and longevity of the infiltration trench.

![Figure 4.7 Flowchart of model simulation for all failure modes and design criteria rationalization](image)
Preliminary steps in this framework consist of defining the input parameters (STEP 1) and profiling the concentration and specific deposition (STEP 2) on a long-term basis. After completion of these preliminary steps, performance criteria may be calculated and characterizations of the three modes of failure are determined; this includes incorporation of the simulated long-term rainfall series (STEP 3). A user-defined number of replications are assigned for the Monte Carlo simulation that is programmed in Microsoft Excel using Visual Basic Application (VBA). STEPS 1 to 3 are carried out for the assigned number of replications and results are outputted to perform STEP 4. Each replication establishes an estimate of the time to fail for each failure mode. In STEP 4, by considering the uncertain parameters (inter-event time, rainfall intensity, rainfall duration, input concentration of total suspended solids, and input of contaminants of concern), probability distributions for each mode of failure are determined. Lastly, as part of STEP 4, by perturbing the infiltration trench design criteria, an attempt is made to extend the longevity and overall performance of these systems by reducing the likelihood of each failure event.

It is understood that there is a high rate of failure of infiltration trenches within the first 5 years of operation due to clogging (Galli, 1992); therefore, this study provides direction for improved trench design to reduce the likelihood of early failures.

### 4.6 SUMMARY

In Chapter 4, the methodology developed for characterizing the long-term performance of infiltration trenches while considering physical clogging is presented. A series of comprehensive macroscopic equations are used in this study that attempt to address the transient stages of deep bed filtration by applying the following: an analytical solution derived by Tien and Ramarao (2007), the O’Melia and Ali (1978) clogging model, and Darcy’s Law. A long-term rate of specific deposition is estimated by projecting the early removal efficiency of suspended solids by fitting a linear relationship. Initial macroscopic
model input parameters were defined based on literature pertaining to stormwater filtration and were selected based on acceptable and recommendation ranges.

An iterative framework is illustrated that consists of the defined input parameters and the comprehensive macroscopic equations. The model framework indicates the use of a Monte Carlo simulation programmed in Microsoft Excel using Visual Basic to minimize the likelihood and frequency of each failure mode.
Chapter 5

ASSESSMENT OF FILTRATION PERFORMANCE

5.1 MODELLING FAILURE MECHANISMS IN DEPTH FILTRATION

As outlined in the first objective of this study, through the use of a continuous long-term simulation, the physical clogging and resulting modes of failure were simulated and are presented herein. The continuous long-term simulation was conducted for a 30 year period which ensures that the expected lifespan of the infiltration trench (8 to 19 years) is captured (Tahoe Basin Interagency Roadway Maintenance and Operations Committee, 2001), including failure events that occur after this projected lifetime.

Assessing system performance from long-term data series or single design events are two approaches for analyzing stormwater conveyance or storage systems. Long-term continuous simulations capture temporal variations in meteorological data based upon actual long-term rainfall data or artificially generated rainfall time series from stochastic methods (Adams and Papa, 2000) as well as model the response of engineered storage systems throughout the expected lifetime of operation. Single storm events do not consider antecedent soil moisture conditions or storage volume that has not drained from a previous event (Adams and Papa, 2000). Certain runoff problems, including water quality problems, are better analyzed by methods such as continuous analysis (Adams and Papa, 2000).

This chapter presents and discusses results from continuous long-term simulations. The results presented in this section are drawn from a single replication in the Monte Carlo simulation; recall that model parameters (e.g., rainfall intensity, interevent rainfall time, rainfall duration, and EMCs) are sampled from their respective probability distributions based on known statistics to simulate long-term rainfall and runoff patterns. Although overall trends are maintained amongst replications, the plots presented in this section may vary.
slightly since the sequence of sampled input parameters changes due to random sampling of the probability distributions. The following outputted results were selected from the second replication in the Monte Carlo simulation completed using VBA code via Microsoft Excel. The implications of parameter uncertainty are presented and discussed in Chapter 6.

### 5.1.1 Reduction in Hydraulic Conductivity and Headloss Buildup

The long-term clogging of the underlying granular sand filter was modelled using hydraulic conductivity, headloss development due to accumulation of particles in the filter, and the overall reduction in flow through the bottom of the trench. The model was run using the input parameter values presented in Table 4.1, and the results represent long-term performance profiles based on the continuous loading of suspended solids calculated from the sampled EMCs and rainfall statistics.

Simulation results of the long-term hydraulic conductivity profile are shown in Figure 5.1.

![Figure 5.1 Profile of hydraulic conductivity (K) as a function of long-term solids deposition (clogging)](image)

As demonstrated in these results, the equivalent hydraulic conductivity decreases exponentially from the initial clean bed hydraulic conductivity, 20 cm/hr, and gradually approaches the abscissa asymptotically as the simulation time approaches 30 years. In the results shown, over the 30 year simulation, the equivalent hydraulic conductivity approaches
a value of approximately 0.3 cm/hr, where the highest rate of decrease occurs during the first
4 years of simulation. This rapid rate of decrease early in the simulation is due the
mathematical form of Equation (2.11). It can be seen in Equation (2.11),
where \( K / K_o \propto 1/\sigma_r^2 \), that the hydraulic conductivity varies inversely with the inverse square
of specific deposition. As the cumulative specific deposition increases with time, the
hydraulic conductivity approaches zero asymptotically.

Subsequently, headloss across the sand filter is determined from the previously calculated
hydraulic conductivities. Figure 5.2 shows headloss development across the underlying sand
filter at the base of the infiltration trench due to accumulation of deposited particles.

![Figure 5.2](image)

**Figure 5.2** Headloss development across the granular filter as a function of long-term solids deposition

These simulation results demonstrate an exponential increase in headloss across the depth of
the filter. An exponential increase in headloss is typical in surface particle removal where
headloss development occurs more rapidly and filter runs tend to be shorter (Hendricks,
2005; Cheremisinoff, 1995). In addition, from Darcy’s equation, hydraulic conductivity and
headloss are inversely related: \( \Delta H_o / \Delta H \propto K / K_o \) (Mays and Hunt, 2005; Li and Davis,
2008b). Recall that headloss development in depth filtration tends to increase linearly
(Cheremisinoff, 1995) as illustrated in Figure 3.3.
Upon completion of the 30 year simulation, the equivalent depth of headloss across the sand filter is much less than the total depth of the trench. Once long-term profiles of hydraulic conductivity and headloss are achieved, the gradual decrease in flow rate through the sand filter is modelled.

Figure 5.3 shows a flow reduction profile across the underlying filter associated with headloss development and hydraulic conductivity reduction. The initial flow through the filter (0.270 m$^3$/hr) is dependent on the initial parameters from Table 4.1 and represents clean filter conditions where it is assumed that there are no deposited particles within the filter at the start of the simulation.

This simulation shows that once specific deposition commences, the flow rate decreases relatively steadily until it reaches the point of clogging which is referred to as the “failure threshold” for the first mode of failure. The gradual decline in filter flow is likely attributable to the decrease in the cross-sectional area of the filter that is available for perpendicular flow. As presented in Equation (4.13), the difference in filter flow between two time steps (1 hour) is assumed to be dependent on the cross-sectional area; therefore, as this area decreases due to clogging, the rate of change in filter flow (i.e., the slope in Figure 5.3) diminishes. Recall that the failure threshold represents only 10 percent of the initial flow. Therefore, based on
the simulation results shown, the filter at the base of the trench reaches the point of clogging after approximately 12 years of operation.

### 5.1.2 Spills or Overflow Events

Dynamic changes in the free water surface within the infiltration trench were modelled. Two simulations were completed to determine the dynamic changes in the water level within the storage trench as runoff drains through the subsurface. First, a simulation was conducted which considers an underdrain at the bottom of the clear stone backfill material, whereas, the second simulation does not include this underdrain. The objective of both of these simulations is to determine and quantify whether the underdrain provides any added benefit to the overall performance of the infiltration trench and specifically to the reduction in frequency and severity of overflow events.

Simulation results of the fluctuating free water surface with the underdrain are shown in Figure 5.4.

![Fluctuating free water surface with an underdrain](image)

**Figure 5.4** Fluctuating free water surface with an underdrain

The red-dashed line in Figure 5.4 indicates the depth of the infiltration trench including the depth of the overflow berm (approximately 30 centimetres). Failure occurs once the free water surface surpasses this depth. As a safety feature to reduce the likelihood of exceeding
the capacity of the trench (common to both simulations), the model only infiltrates up to the first 25 mm of rainfall per event and by-passes any excess volume of runoff around the trench (as indicated in Figure 3.2b). Peaks in the free water surface profile in Figure 5.4 that are greater than the depth of the trench surface denote an overflow event. In theory, the underdrain provides additional drainage capacity for the filter once the subsurface begins to clog with deposited particles. The model simulates this behaviour by draining the infiltration trench if a significant volume of runoff infiltrates resulting in an overflow event (i.e., there is no residual volume in the trench at the beginning of the next rainfall event). Urbonas (2002) illustrates this type of by-pass system in Figure 3.2b where filtration through the underlying subsurface is by-passed during extreme rainfall events.

From the results shown in Figure 5.4, the free water surface after each rainfall event remains below the total depth of the trench during the first 7 years of operation; therefore, no overflow failure events occur during this time. However, after almost 8 years of simulation, the first overflow failure event occurs. The frequency and magnitude of overflow events increase as a function of simulation time and filter clogging.

Figure 5.5 shows simulation results of the fluctuating free water surface without the underdrain located at the bottom of the trench.
Similar to Figure 5.4, the simulation results presented in Figure 5.5 show that the capacity of the infiltration trench to store and drain urban runoff between rainfall events functions adequately during the early years of operation. Without the presence of the drain, the first overflow failure event occurs after the 6th year of simulation. As a result, the minimum hydraulic depth in the trench begins to rise due to inadequate drainage capacity between rainfall events; this trend is observed between 16 and 20 years of simulation time. The preceding results suggest that after a prolonged period of inadequate drainage, the stored runoff volume will begin to pond at the surface of the trench. An increase in ponding depths as the infiltration system begins to clog may become hazardous since some areas may become prone to mosquito breeding (Davis et al., 2009). The influence of evapotranspiration in the overall mass balance of urban runoff was not addressed in this preliminary analysis, however, is recommended for consideration in future studies as discussed further in Chapter 8.

Both simulation results suggest that overflow events occur more frequently with greater magnitudes (i.e., the volume of spill increases) once the underlying granular filter becomes clogged. However, the use of an underdrain may extend the first overflow event occurrence by almost 2 years, as seen in the above results. Different times to fail were achieved between simulations since in the first case (Figure 5.4) the trench is drained once the trench depth of 2 metres is exceeded; however, a failure event is not identified until the capacity including both the trench and the overflow berm depth are exceeded. Hence, there may be some overflow events that exceed the depth of 2 metres; however, they do not exceed the capacity including the overflow berm and are consequently not considered as failure events. Lastly, in both cases, although the first failure event does not occur until years 8 and 6, respectively, proper drainage as per the design guidelines prior to these failure times is not achieved.

### 5.1.3 Contaminant Loading

As discussed in Section 4.3.4, a preliminary investigation of the potential for heavy metal contamination of the underlying groundwater was determined using the correlation between suspended solids and heavy metals including: copper, zinc, and iron. Based on estimated
mean concentrations of suspended solids between 0 mg/L and 35 mg/L, considering pre-
treatment and removal through the filter fabric, Figure 5.6 illustrates estimates of the
expected associated ranges in concentration of each of the listed heavy metals in the
infiltration trench. Figure 5.6 shows a linear increase in each heavy metal concentration as
the concentration of inflowing suspended solids increases.

\[
\begin{align*}
\text{(a)} & \quad \text{Zinc (mg/L)} \\
\text{(b)} & \quad \text{Iron (mg/L)} \\
\text{(c)} & \quad \text{Copper (mg/L)}
\end{align*}
\]

**Figure 5.6** Estimated ranges of heavy metal concentrations based on their correlation with suspended solids concentration

In all three graphs, aqueous concentrations of zinc, iron, and copper are greater than their respective groundwater quality standard. Since many urban runoff pollutants are associated with suspended particles, pollutants may be readily removed in infiltration devices through
filtration mechanisms. However, Figure 5.6 suggests that in the case where no suspended solids are deposited throughout the infiltration trench (i.e., complete removal through upstream pre-treatment mechanisms), heavy metals are still likely to be present in the infiltrating urban runoff. This observation suggests that even with the removal of all particulates from the urban runoff prior to infiltrating into the trench, dissolved constituents may be present in the urban runoff.

DeBusk et al. (1997) suggest that heavy metals such as zinc, cadmium and copper are washed from roadways primarily in their dissolved forms. The presence of dissolved metals is a concern since sand filters are not believed to be effective at removing soluble pollutants (DeBusk et al., 1997). Therefore, it is reasonable to conclude that dissolved constituents may pose a risk to the water quality in the underlying aquifer. Furthermore, zinc and copper are treated as conservative metal tracers through the filter material in this study. It is reasonable to assume that these heavy metals act as conservative tracers through the sand filter since the sand media is relatively inert and is not effective at removing soluble pollutants in stormwater applications.

Figure 5.7 illustrates results from a single replication of the cumulative mass loading of zinc and copper released to the groundwater as a function of time and filter flow through the subsurface of the infiltration trench. The cumulative mass loading is normalized with the catchment area. Although groundwater standards are concentration-based, the cumulative mass loading of dissolved metals was examined as a simplified alternative approach to define a groundwater failure event or exceedence of the point of compliance.

The results in Figure 5.7 show that in the early years of simulation, the cumulative mass loading of both copper and zinc increase. However, the mass loading gradually approaches the points of compliance which are denoted as the failure thresholds as the simulation time approaches 30 years. The cumulative mass loadings of copper and zinc that penetrate through the bottom of the infiltration trench are limited by the gradual clogging and subsequent flow reduction through the filter material. In addition, Figure 5.7 demonstrates that a larger cumulative mass of zinc is found to penetrate the trench compared to copper which is attributable to a much higher measured mean concentration of zinc in urban runoff.
The time to fail for both heavy metals in Figure 5.7 depends on assigned mass loading thresholds (or points of compliance). Based on available literature, acceptable points of compliance were not identified; therefore, thresholds for this study are assigned based on reasonable judgment and are referred to as the failure threshold. Once the failure threshold has been exceeded, a failure event is denoted. For copper and zinc, the cumulative mass thresholds used in this study are 0.40 kg/ha and 1.40 kg/ha, respectively. In the results shown above, it is anticipated that the mass loading thresholds for copper and zinc would be exceeded after 2.9 years and 3.2 years, respectively.

 Lastly, in the case where rehabilitation of the entire infiltration trench occurs and the underlying filter is replaced prior to the expected time of clogging, higher cumulative mass loadings of both copper and zinc would result potentially causing further degradation of the groundwater quality. Frequent rehabilitation of the underlying granular filter encourages long-term infiltration of potentially deleterious dissolved contaminants.
5.2 **PARAMETER SENSITIVITY**

Long-term filter performance profiles are subject to change depending on the sensitivity of input parameters such as the clogging factor as well as the level of pre-treatment provided upstream by various mechanisms such as those described in Section 3.1.1. The purpose of this section is to demonstrate the expected variability within the clogging profile based on changes in site specific parameters. First, the clogging factor is selected as a sensitive site parameter since the mean particle size of deposited suspended particles can be quite variable and it is, therefore, necessary to identify the influence of particle size on the overall long-term filter performance. Next, the level of pre-treatment is investigated since without proper monitoring and maintenance of pre-treatment facilities, upstream particle removal rates may decline and the resulting impact on filter performance should be identified. Pre-treatment requirements may also vary based on the selected site for the infiltration trench or type of deposited particle common to the location.

It is evident through current investigations that each mode of failure is highly influenced by the clogging of the underlying filter. In both overflow failure events (i.e., with and without the underdrain) and groundwater failure events, flow reduction through the bottom of the trench plays a significant role in the timing and magnitude of trench failure. Therefore, the following section investigates the influence of both the clogging parameter and the degree of pre-treatment on the filter flow rate profile. In addition, to assist in identifying trends in the clogging profile when sensitive parameters are perturbed, the input concentration is held constant at 120 mg/L and the same time series of rainfall events is used in each sensitivity analysis.

### 5.2.1 Clogging Factor

The clogging factor, $\gamma$, is calculated using Equation (2.11) and relates the filter medium grain size to the size of the deposited particle. In general, finer particles have been found to have a higher tendency to clog filters compared to coarse particles (Boadu, 2000; Li and Davis, 2008b). In this study, by varying the clogging factor, the influence of deposited particle size
on filter clogging is demonstrated. As shown in Figure 2.1, when the mean size of deposited particle decreases (i.e., more fine particles), the clogging factor becomes larger which results in a stronger tendency to clog the filter media. Conversely, as the mean deposited particle size increases (i.e., more coarse particles), the clogging factor decreases and, therefore, results in a lower tendency for the filter to become clogged. Smaller particles are known to clog depth filters due to their swelling behaviour, such as fine clays, which leads to filling of media pores and overall decrease in permeability (Li and Davis, 2008b).

In addition to the base case scenario where a mean deposited particle size of 15 $\mu$m is assigned, three other scenarios are simulated and shown in Table 5.1.

<table>
<thead>
<tr>
<th>Deposited Particle Size, $\mu$m</th>
<th>Clogging Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>645</td>
</tr>
<tr>
<td>15</td>
<td>215</td>
</tr>
<tr>
<td>25</td>
<td>121</td>
</tr>
<tr>
<td>35</td>
<td>92</td>
</tr>
</tbody>
</table>

In Table 5.1, for small deposited particles such as 5 $\mu$m, a larger clogging factor was estimated; whereas, smaller clogging factors resulted in cases where a larger particle was used in the simulation as previously discussed in Section 2.2.

Figure 5.8 shows results from the sensitivity analysis assuming a constant and uniform loading of suspended solids with a range of clogging factors.

Figure 5.8 confirms that when a larger clogging factor results from finer deposited particles, the filter flow decreases quickly, indicative of rapid filter clogging. Specifically, the most rapid filter clogging throughout the depth of the filter is observed when large clogging factors result, leading to a clogged filter prior to year 3 of the simulation. Again, it is speculated that this phenomena is the result of high concentrations of fine particles that tend to floculate together, becoming trapped in the interstices of the porous media throughout the depth filter.
Also observed in Figure 5.8, the filter remained unclogged until almost 15 years when a simulation with coarser particles, 35 μm, was performed.

![Figure 5.8 Influence of clogging factor on the filter flow profile](image)

### 5.2.2 Upstream Removal of Suspended Solids

Pre-treatment requirements may vary depending on the infiltration trench location. Davis et al. (2009) comment on pre-treatment considerations for bioretention systems and confirm that the need, type, and size of pre-treatment required should be analyzed and updated for improved future performance. Therefore, in this study, by varying the level of upstream pre-treatment, its influence on filter performance is determined which may lead to improved overall trench design.

Figure 5.9 shows results from the sensitivity analysis for a constant and uniform loading of suspended solids with varying degrees of sediment removal from upstream pre-treatment mechanisms.
This study confirms that by increasing the level of pre-treatment upstream of the infiltration trench, operation of the filter under non-clogged conditions can be sustained for a longer period of time. In the base case scenario of 25% upstream sediment removal, the clogging threshold is reached after approximately 4 years; however, in the case where no pre-treatment mechanisms are considered, the clogging threshold is reached after 3 years. A significant increase in the longevity of the underlying filter is observed when pre-treatment is increased to 75% and the filter reaches the clogging threshold after approximately 12 years of operation. Although the maximum filter performance is observed by increasing upstream sediment removal to 75%, the literature indicates that typical levels of treatment from pre-treatment mechanisms range from 25 to 30% (Barr Engineering Company, 2001). Longer operating periods of infiltration trenches under non-clogged conditions may be achieved by incorporating treatment methods with higher removal capabilities that are routinely maintained upstream of the BMP in sites where heavy sediment loads are a concern.

5.3 SUMMARY

The first research objective as described in Chapter 1 is presented and discussed in this Chapter. Each failure mode was profiled using a long-term continuous simulation, while also considering the physical clogging of the filter medium due to particle straining in the
interstices of the porous media throughout the depth filter. Time-varying hydraulic conductivity and headloss profiles across the underlying sand filter were established on the basis that both of these parameters vary as a function of the cumulative specific deposition of particles. Flow reduction through the bottom of the infiltration trench was observed and the clogging failure threshold was reached prior to the expected lifetime of the infiltration trench (8 to 19 years). Consequently, as flow through the bottom of the trench decreases as a function of cumulative specific deposition, the frequency and magnitude of overflow events increase. Adequate volumetric performance (i.e., proper drainage resulting in no overflow events), was observed during the first few years of simulation. In the case where no underdrain was added at the bottom of the clear stone fill, the simulation results indicated the earlier development of ponding at the trench surface.

The granular sand filter at the base of the trench is effective in removing particulates and as a result, contaminants in their dissolved forms including zinc and copper may pose risk to the underlying aquifer. The cumulative mass loading of both of these metals was limited by the gradual flow reduction across the filter. By assigning mass loading failure thresholds, a methodology for determining the time of failure for the third failure mode, groundwater contamination, is presented. Based on the results shown above, both zinc and copper reached their respective failure thresholds within the first 5 years of simulation.

The long-term performance profiles presented in this section are based upon site specific input parameters (shown in Table 4.1). To demonstrate that performance profiles change based on varying site conditions, two sensitivity analyses were conducted: the clogging factor and the degree of upstream pre-treatment. In both cases, when the input parameters were varied, the longevity of the underlying filter bed changed. Deposition of fine particles resulted in faster clogging of the filter bed compared to deposition of coarser particles. In addition, the longevity of the underlying filter bed is increased by achieving a higher level of pre-treatment upstream of the infiltration trench.
Chapter 6

PROBABILISTIC ANALYSIS WITH THE MACROSCOPIC FILTRATION MODEL

6.1 OVERVIEW OF UNCERTAIN MODEL PARAMETERS

Chapter 5 presents and discusses results from a long-term continuous simulation which was drawn from a single replication as part of a Monte Carlo analysis. This study considered a user-defined number of replications to establish a distribution of results. These long-term performance profiles reveal the nature of the physical clogging and resulting failure modes of a BMP filter bed. The performance profiles shown in Chapter 5 reveal the gradual transition from a clean and pristine filter media to a clogged filter bed. However, the associated times to fail for each failure mode cannot be fully or accurately characterized by merely observing and drawing conclusions from a single continuous simulation; replications are required.

Computer models are often applied in deterministic approaches to estimate the expected performance of a BMP facility throughout its lifetime of operation. In reality, there is often questionable confidence in a single input parameter value, such as suspended solids concentration, which results in parameter uncertainties. Estimation of expected failure times is more realistically determined with probability density functions since particular input parameter values are uncertain; e.g., interevent rainfall time, rainfall event duration, rainfall event intensity, suspended solids EMC, and copper and zinc EMCs. Therefore, a more comprehensive estimate of the system reliability is achieved through a probabilistic analysis.

This chapter presents and discusses the development of a computer simulation model and the resulting probabilistic analysis that better characterizes the anticipated failure times for each mode of failure.
6.1.1 Rainfall Event Characteristics

The meteorological characteristics used in this study, as outlined in Chapter 4, follow exponential probability distributions and are assumed to be independently distributed. A simulated rainfall series is established by randomly sampling meteorological parameter values from their respective probability distributions. A different simulated rainfall series is established for each Monte Carlo replication. Figure 6.1 presents simulated probability distributions for each of the meteorological parameters used to create a single rainfall series.

![Figure 6.1](image)

**Figure 6.1** Probability density functions of rainfall event characteristics: (a) rainfall event intensity; (b) rainfall event duration; and (c) interevent time

The probability densities presented in this section represent one realization from the Monte Carlo simulation and are illustrated for the purpose of discussion. As discussed in Section 6.2, the Monte Carlo simulation consists of 1,000 realizations (or replications) to characterize
the probability of each mode of failure, where each realization models the trench performance over a 30 year period.

Illustrated in Figure 6.1 are the simulated and theoretical probability densities. The theoretical density functions are derived from the exponential probability distribution expression. In all three plots, both the simulated and theoretical distributions follow similar exponential profiles and are, therefore, in agreement with each other. It is the view through this study that the simulated long-term rainfall series that is generated (shown in Figure 4.4) is representative of a realistic long-term rainfall series that may be observed in the vicinity of Pearson Airport, Ontario. These figures illustrate that rainfall events with low intensities and short durations occur most frequently.

### 6.1.2 Event Mean Concentrations

In addition to meteorological statistics, EMCs for suspended solids, copper, and zinc are randomly sampled from their respective lognormal probability distributions to simulate the quality of urban runoff prior to infiltration into the BMP facility. Figure 6.2 shows the simulated and theoretical probability density functions for each water quality parameter.

In all three plots, the simulated probability distributions as well as the theoretical probability density functions follow a similar lognormal shape. Therefore, it is accepted that these distributions closely resemble a realistic long-term variation in concentrations for each water quality parameter. In addition, the probability plots in Figure 6.2 suggest that extremely high concentrations of each water quality parameter would occur infrequently.
6.2 **VISUAL BASIC APPLICATION**

Given the repetitive nature of randomly sampling each uncertain parameter from their respective probability distributions to produce the long-term rainfall and urban runoff water quality data series, a macro program was coded using VBA in Microsoft Excel. This macro is also used to perform repetitive trials to develop the probability distributions for each mode of failure. The model code is presented in Appendix A.

To establish an appropriate number of simulation replications to be used for each model simulation, the modelling results from an increasing number of replications were calculated.
and are summarized in Table 6.1. Each simulation experiment consists of a defined number of replications to establish probability distributions for each failure mode.

<table>
<thead>
<tr>
<th>No. Replications</th>
<th>Mean (years)</th>
<th>STDEV (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>14.2</td>
<td>3.36</td>
</tr>
<tr>
<td>200</td>
<td>14.2</td>
<td>3.18</td>
</tr>
<tr>
<td>500</td>
<td>14.4</td>
<td>3.54</td>
</tr>
<tr>
<td>750</td>
<td>14.4</td>
<td>3.51</td>
</tr>
<tr>
<td>1000</td>
<td>14.3</td>
<td>3.37</td>
</tr>
</tbody>
</table>

The simulation results presented in Table 6.1 show that by gradually increasing the number of replications from 100 to 1,000, minimal differences amongst the simulated time to clog are observed. In addition, earlier preliminary simulations were also completed using up to 10,000 replications and similar repeatability amongst results was observed. As a result, to achieve stable and reliable modelling results in this study, probability distributions are established by performing 1,000 replications in each simulation experiment. A single replication is completed in approximately 10 to 15 seconds; therefore, limiting simulations to 1,000 replications ensures relatively stable results while avoiding excessive computation times.

The probabilistic approach for determining each failure time requires that random numbers be generated to sample each uncertain input parameter from defined probability distributions. Random number generation can be achieved by using the built-in Microsoft Excel “Rnd” function or by employing a computer module for an existing random number generator in the VBA model. The calculation of pseudo random numbers using a random number generator is based on a recursive mathematical formula. The application of Microsoft Excel for statistical algorithms, including random number generation, has been criticized in the past; as such, an alternative approach for random number generation was investigated in this study. L’Ecuyer (1999) suggests a random number generator with a period length much longer than the built-in “Rnd” function that prevents early repetition of the generated random numbers. Both methods are compared by calculating the sample mean of total suspected solids EMC from several thousands of replications; results are illustrated in Figure 6.3.
The results in Figure 6.3 illustrate minimal differences between the two methods of random number generation when testing sample sizes between 100 and 100,000. Both sample means converge on the population mean, 133, as the sample size increases (the population mean is shown by the blue-dashed line in Figure 6.3). It is likely that the built-in Microsoft Excel function may provide adequate results for this study; however, the alternative approach is used to avoid any concerns that may be associated with the Microsoft Excel “Rnd” function.

### 6.3 Long-Term Probability of Failure

As defined in the second research objective of this study, this section presents and discusses results from the probabilistic analysis that characterizes the long-term probability of each failure mode.

It is the goal of this section to better characterize the long-term performance of infiltration trenches based on current design and maintenance practices. Furthermore, although infiltration trenches may operate within acceptable limits from one perspective (e.g., acceptable drainage rates through the filter) at any given time, a failure event from a different aspect of operation may transpire. For example, the infiltration trench may not have reached
the point of clogging; however, it may have already exceeded a specific groundwater quality standard. For this reason, by investigating long-term system performance in terms of the three different failure modes, this study provides a comprehensive characterization of infiltration trenches by considering several performance criteria rather than assuming that one criterion (e.g., drainage rate) dictates the overall system performance.

### 6.3.1 Probability of Clogging

The long-term probability that the underlying granular sand filter will reach the clogging failure threshold in any given year is modelled as illustrated in Figure 6.4. The probability distribution in Figure 6.4 is first established using the base case input parameters presented in Table 4.1.

![Figure 6.4 Probability density function for failure due to a clogged filter](image)

In addition to identifying the approximately normal probability distribution presented in Figure 6.4, this figure also presents the cumulative distribution function. This cumulative probability curve suggests that as the infiltration trench approaches a life of 14 to 16 years, there is a 50% likelihood that the underlying sand filter would reach the clogging failure threshold (as discussed in Chapters 3, 4, and 5). However, clogging may first appear as early as the 8\(^{th}\) year after installation which indicates that the infiltration system may not be operating as per design conditions long before the end of its expected design lifetime (8 to 19
years). Based on the simulation results summarized in Figure 6.4, a clogging failure event would occur prior to 30 years of operation. However, a major rehabilitation event is likely anticipated prior to reaching this 30 year mark. The considerable spread in the data (approximately 8 years to 30 years) is a result of the large standard deviations associated with the uncertain input parameter values. For instance, the mean EMC for suspended solids concentration in urban runoff is 133 mg/L with an associated standard deviation of 116 mg/L (Adams and Papa, 2000).

As discussed in Chapter 5, the reduction of flow through the bottom of the infiltration trench, due to the accumulation of deposited particles and reduction of void space may lead to other modes of failure such as trench overflow events.

### 6.3.2 Probability of Overflow Events

The probability of the first overflow event occurring in any given year of operation, considering two design alternatives (i.e., a trench with or without an underdrain at the base of the clear stone media), was modelled and is analyzed in this section. Figure 6.5 shows the results from the design alternative with no underdrain, while Figure 6.6 shows the alternative with the underdrain.

![Figure 6.5 Probability distribution for failure due to trench overflow with no underdrain](image-url)

85
Figure 6.5 illustrates that there is 50% likelihood that after 8 to 10 years of operation, the first spill or overflow event would occur; however, trench overflows may be first observed as early as the 4th year of simulation. After an operation period of 16 to 18 years, based on simulation results, Figure 6.5 indicates that the first overflow event would take place prior to this time. As shown in Figure 6.6, a similar probability distribution, with slight differences, is observed when considering the design alternative with an underdrain.

Figure 6.6 also illustrates that an overflow event may be first observed with certainty after 16 to 18 years of simulation, and the first spill may first appear as early as the 4th year. However, when comparing the mean of the simulated results between both design alternatives, the average time with no observed overflow event in cases with and without an underdrain are 8 years and 7.3 years, respectively. Therefore, underdrains delay the occurrence of the first overflow on average by less than 1 year. Furthermore, the cumulative probabilities of overflow failure after 8 years of operation in Figure 6.5 and Figure 6.6 are 0.69 and 0.56, respectively, indicating a slightly higher probability of failure in the case with no underdrain. Minimal difference in the standard deviation between alternatives is observed; the standard deviations with and without the underdrain are 1.98 and 1.88, respectively.
Based on the simulation results presented in this section, in addition to the free water surface profiles illustrated in Figure 5.4 and Figure 5.5, the use of an underdrain at the base of the trench is not likely to extend the timing of the first overflow event. However, the underdrain is likely to be effective at reducing the impacts of the overflow failure event by preventing the occurrence of ponding at the surface.

### 6.3.3 Probability of Groundwater Contamination

As indicated in Chapter 5, the selection of failure thresholds for the mass loading of both copper and zinc are not based on established groundwater quality standards, rather on reasonable judgment. The probability of water quality failure is determined in this section to illustrate the potential utility of this new methodology.

The probability that the defined loading threshold for copper and zinc is exceeded in any given year of operation is illustrated in Figure 6.7 and Figure 6.8, respectively.

Based on the simulation results presented in Figure 6.7 and considering the copper loading threshold (0.40 kg/ha), there is a 50% likelihood that a groundwater quality failure event with respect to dissolved copper occurs prior to 3.5 years of simulation. In addition, as presented in the simulation results, a groundwater failure event with respect to copper would occur within the first 5 years of operation. A similar probability distribution is established for reaching a groundwater quality failure event with respect to dissolved zinc.

The methodology in this study uses 1.40 kg/ha of zinc as the loading failure threshold (greater than the loading threshold for copper). Adams and Papa (2000) report a slightly larger standard deviation for zinc in urban runoff as compared to copper, and consequently, a slightly larger spread in the simulation results in Figure 6.8 is observed. In addition, there is a 50% likelihood that groundwater quality failure with respect to zinc would occur after 3.8 years, which is moderately longer than the 50% failure time observed for copper. After 6.5 years of operation, a zinc failure event would take place with certainty. Longer failure times
are calculated for zinc since a higher failure threshold was assigned and, therefore, more time is required to reach this threshold.

![Probability Distribution](image1)

**Figure 6.7** Probability distributions for the failure mode of exceeding copper loading threshold

![Probability Distribution](image2)

**Figure 6.8** Probability distributions for the failure mode of exceeding zinc loading threshold

In the present study, it is important to assign different loading thresholds for each metal to show that the calculated time of failure is dependent upon the magnitude of the value selected. Therefore, it is evident that poorly defined failure thresholds can result in significantly higher amounts of metals being discharged into the underlying groundwater system before a failure event is identified and remedial actions are initiated.
Overall, the simulation results observed from the probability distributions for the last failure mode, Figure 6.7 and Figure 6.8, demonstrate a much narrower spread in the modelled failure times compared to the clogging and overflow failure probability distributions. The calculated mass loading of each dissolved metal is dependent upon the flow through the base of the infiltration trench as well as the sampled EMCs. It is hypothesized that the narrower spread in the simulation results for the last failure mode is due to smaller standard deviations of the inputted EMC statistics for zinc and copper in comparison to the statistics for suspended solids concentration that has a significantly higher standard deviation.

6.4 SUMMARY

The second research objective as described in Chapter 1 is presented and discussed in this Chapter. As revealed in the probabilistic analyses, clogging of the underlying granular sand filter may occur as early as after 8 years of BMP operation; however, the highest probability of clogging will occur between 14 and 16 of trench operation. Rehabilitation of the entire infiltration trench will be required prior to 30 years of operation, as the clogging threshold will certainly be reached by this time.

Associated with the clogging of the underlying filter, the first overflow event is certain to occur after year 16 of operation; however, the highest probability of overflow is achieved between years 8 and 10. Although a significant delay in the occurrence of the first overflow event may not be achievable with the application of an underdrain, this device may be better suited for reducing the impacts of the overflow events, such as preventing ponding at the surface of the infiltration trench and reducing the total number of overflow events anticipated during the expected lifetime of operation.

The third failure mode of groundwater quality failure is dependent on the specified failure mass threshold where the groundwater standard is exceeded. Based on the defined methodology in this study (i.e., the assigned mass thresholds) the failure thresholds for copper and zinc are reached within the first 5 and 6 years of simulation, respectively.
The distribution of results in each probability distribution presented and discussed in this Chapter indicate significant differences in the anticipated times of failure for each failure mode. In addition, it is observed that when characterizing the overall long-term performance, the anticipated time of failure of the infiltration trench is highly dependent on which performance criterion is being considered. As seen, one type of failure event may be achieved prior to another.
Chapter 7

INCORPORATING MACROSCOPIC FILTRATION THEORY INTO TRENCH DESIGN

7.1 SENSITIVE DESIGN CRITERIA

As outlined in the third research objective of this study, improved design practices and maintenance regimes are established by considering both macroscopic theory and simulation results from the probabilistic analysis in Chapter 6. Through completion of a thorough sensitivity analysis in this section, the influences of varying design parameters on each failure mode are determined. This chapter includes a sensitivity analysis on four different model parameters including: infiltration trench area, infiltration trench depth, thickness of the filter media, and pre-treatment efficiency.

Failure of an infiltration trench often occurs due to poor siting and design as well as neglecting routine maintenance or rehabilitation requirements. Therefore, by bringing attention to how each design criterion contributes to the long-term performance and ultimate failure of the BMP facility, improved environmental protection strategies are suggested in this Chapter. Specifically, this Chapter compares statistics from each simulation to determine how each design criterion influences the long-term system performance.

7.1.1 Area of Infiltration Trench

To rationalize the recommended trench design surface area, this parameter is both increased and decreased from its initial value to determine the appropriateness recommended guidelines and the overall influence on long-term performance. In this first experiment, the surface area is perturbed while all other design criteria are held constant at their initial values.
as per Table 4.1. The surface area of the infiltration trench in this sensitivity analysis is taken to be the same as the cross-sectional area of the granular filter bed (i.e., effective filtration surface area). Current design practices recommend that the area of the trench be sized according to the average volume of urban runoff expected to infiltrate into the system as well as granular media properties in the underlying drainage system (Barr Engineering Company, 2001). However, recognizing that Galli (1993) suggested that many infiltration trenches fail within the first 6 years of operation (mode of failure undefined) and that the performance of these systems decline over the long-term, this study investigates the appropriateness of existing design procedures.

The effective filtration surface area is varied in an attempt to increase the longevity of the infiltration trench. In this analysis, the long-term influence of a range of infiltration trench areas is investigated. Statistical results for model simulations are summarized in Table 7.1.

<table>
<thead>
<tr>
<th>Area m²</th>
<th>Clogging (no underdrain)</th>
<th>Overflow (underdrain)</th>
<th>Copper Loading</th>
<th>Zinc Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stat: Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
</tr>
<tr>
<td>30</td>
<td>14.3 3.4</td>
<td>0.33 0.20</td>
<td>0.55 0.38</td>
<td>25.8 7.0</td>
</tr>
<tr>
<td>60</td>
<td>14.3 3.5</td>
<td>4.3 1.5</td>
<td>5.2 1.4</td>
<td>5.5 3.0</td>
</tr>
<tr>
<td>90</td>
<td>14.3 3.5</td>
<td>7.3 1.9</td>
<td>8.0 2.0</td>
<td>3.1 0.26</td>
</tr>
<tr>
<td>120</td>
<td>14.3 3.5</td>
<td>9.1 2.2</td>
<td>9.7 2.3</td>
<td>2.2 0.18</td>
</tr>
<tr>
<td>150</td>
<td>14.3 3.4</td>
<td>10.3 2.4</td>
<td>10.7 2.5</td>
<td>1.7 0.14</td>
</tr>
</tbody>
</table>

*all statistics are in years of simulation time
Stdev (standard deviation)

Simulation averages and standard deviations are summarized in Table 7.1; these statistics are used to evaluate the effect of varying surface area on each model failure mode. These simulation results are also summarized in Figure 7.1.

First, by decreasing the base case surface area from 90 m² to 30 and 60 m², no changes in the estimated mean time to clog or the associated standard deviation are observed. The time to reach the clogging threshold is dependent on the mass loading of deposited suspended solids to the underlying filter. However, as flow of urban runoff through the sand filter decreases
due to the reduction in surface area while the filtration rate is assumed to remain fairly constant, the mass loading of solids in the underlying sand filter is restricted. To reiterate, as shown with the mass balance expression in Equation (4.13), as flow is calculated from one time step to another, the reduction of flow is a direct result of the decrease in effective filtration area and the headloss development due to deposition of solids in the interstices of the porous media. Although as the design surface area of the trench is increased and more flow is initially transmitted through the trench bottom, a constant volumetric loading rate of suspended solids per unit area in the underlying filter was observed in model simulations. This trend indicates a proportional relationship between flow and area, which results in an unchanged mean time to clog in the underlying sand filter. It is assumed that the volume of suspended solids that settle out from the fluid is insignificant. Therefore, changing the area of the filter does not influence the ratio of specific deposition (cm$^3$ of suspended solids per cm$^3$ of filter media). The same trend is observed when increasing the effective filtration area from 90 m$^2$ to 120 and 150 m$^2$, respectively.

![Figure 7.1](image)

**Figure 7.1** Effect of infiltration trench area on model failure modes – time to failure (years)

When the effective filtration area is decreased to 30 and 60 m$^2$, the modelled mean failure times for the first overflow event with the underdrain are shortened by approximately 93% and 35%, respectively. Modelled mean failure times for 30 and 60 m$^2$ trench areas are also
shortened by approximately 95% and 41%, respectively, when there is no underdrain system. As the trench area is decreased, while holding the depth constant, the storage volume of the trench or reservoir (overlying the sand filterbed) is reduced which results in earlier and more frequent overflow events. Storage volume for the urban runoff (not to be confused with storage of deposited solids in the underlying depth filter) is reduced due to a narrower trench. However, by increasing the trench area to 120 and 150 m\(^2\), the estimated time to reach the first overflow event is delayed by approximately 2 and 3 years, respectively, compared to the base case simulation results. For trench designs with and without the underdrain, these delays represent an increase in mean time to failure by approximately 21% and 25% for areas of 120 m\(^2\) and 34% and 41% for areas of 150 m\(^2\), compared to the base case results. Standard deviations in each simulation change marginally; when the time to fail increases, the standard deviation increases slightly and vice versa.

Finally, estimated times to reach both copper and zinc mass loading thresholds vary depending on the effective size of the filtration area. Decreasing the trench area reduces the mass loading of copper and zinc to the underlying groundwater and consequently results in longer times required to reach these failure thresholds; estimated mean times to reach failure thresholds for copper and zinc increase by approximately 77% and 102%, respectively, with a trench area of 60 m\(^2\). When the trench area is further decreased to 30 m\(^2\), the estimated mean times to reach both copper and zinc failure thresholds increase by over 600% and 700%, respectively. A significant increase in time to fail is observed when the trench area is reduced to 30 m\(^2\) since the mass loading of dissolved metals to the groundwater is significantly reduced and the time to fail occurs after the 30 year simulation limit in some model replications. Conversely, by increasing the design trench area and holding all other design criteria constant, an early higher flow of urban runoff (and washed off metals) through the filter is initially achieved and leads to earlier times to reach the respective groundwater failure thresholds since more mass of metals is transmitted into the aquifer (i.e., groundwater failure). To reiterate, since a larger volume of runoff infiltrates into the groundwater by increasing the design area, following Equation (4.18), the contaminant load of potentially threatening species per rainfall event increases. Compared to the base case simulation, failure times for copper and zinc decrease by approximately 29% and 30%, respectively, with an
area of 120 m², and decrease by 45% and 46%, respectively, with an area of 150 m². Again standard deviations change with increasing and decreasing mean times to fail; more significant changes in model results are observed by decreasing the trench area. Large standard deviations are observed when the trench area is reduced since several of the replications completed for copper and zinc result in failure times after the 30 year simulation limit. This significantly delayed time to failure is observed when clogging is achieved early in the simulation, which prevents a large cumulative volume of metals from penetrating the filter.

### 7.1.2 Depth of Infiltration Trench

The depth of the infiltration trench is typically sized using guidelines described in BMP manuals such as the *Minnesota Small Urban Sites BMP Manual (2001)*. Characteristically, design recommendations for the infiltration trench depth range between 0.5 and 3 metres (Government of British Columbia, 2006). Trench dimensions are often established from generic design recommendations rather than specific meteorological or hydrologic conditions. The depth of the infiltration trench in this sensitivity analysis is varied while holding all other design parameters constant; trench depths ranging from 1 to 3 metres are compared to the base case model, 2 m, to determine the effect of trench depth on the overall long-term system performance. Statistical results for the model simulations are summarized in Table 7.2. Simulation results presented in Table 7.2 are also illustrated in Figure 7.2.

As presented in Table 7.2 and Figure 7.2, whether increasing or decreasing the trench depth, the estimated mean time to reach the clogging threshold remains unaffected. By changing the trench depth, the flow of suspended solids through the underlying filter is not affected. The flow of urban runoff through the filter is a function of both approach velocity ($U_a$) and cross-sectional area ($A$) available for perpendicular flow and is independent of the overlying trench storage volume. No significant changes in the standard deviation are identified. However, based on simulation results, altering the overlying trench storage volume changes the estimated times of the first overflow event.
Table 7.2 Effect of trench depth on model failure modes – time to failure (years)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Clogging</th>
<th>Overflow (no underdrain)</th>
<th>Overflow (underdrain)</th>
<th>Copper Loading</th>
<th>Zinc Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stat:</td>
<td>Mean</td>
<td>Stdev</td>
<td>Mean</td>
<td>Stdev</td>
<td>Mean</td>
</tr>
<tr>
<td>1</td>
<td>14.3</td>
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<td>4.8</td>
<td>1.2</td>
<td>5.6</td>
</tr>
<tr>
<td>1.5</td>
<td>14.3</td>
<td>3.4</td>
<td>6.3</td>
<td>1.5</td>
<td>6.9</td>
</tr>
<tr>
<td>2</td>
<td>14.3</td>
<td>3.5</td>
<td>7.3</td>
<td>1.9</td>
<td>8.0</td>
</tr>
<tr>
<td>2.5</td>
<td>14.3</td>
<td>3.4</td>
<td>8.1</td>
<td>2.0</td>
<td>8.7</td>
</tr>
<tr>
<td>3</td>
<td>14.3</td>
<td>3.5</td>
<td>8.7</td>
<td>2.2</td>
<td>9.3</td>
</tr>
</tbody>
</table>

*all statistics are in years of simulation time

Stdev (standard deviation)

Figure 7.2 Effect of trench depth on model failure modes – time to failure (years)

From Table 7.2 and Figure 7.2, when the depth of the infiltration trench is decreased to 1 and 1.5 m, the estimated mean times of the first overflow event are observed to be earlier by approximately 2.5 and 1 year, respectively, compared to the base case model results. Increasing the trench depth to 2.5 m results in a delay of the first overflow event by less than one year for both trench designs (with and without the underdrain), whereas increasing the trench depth to 3 m appears to delay the first overflow event by 1.3 and 1.6 years in cases with and without the underdrain, respectively. Early and more frequent overflow events occur by decreasing the trench depth since the storage volume available for urban runoff is reduced. Conversely, delayed and less frequent overflow events result by increasing the trench depth due to increased available storage volume. However, attention to the depth of
the water table must be considered when determining an appropriate depth of the infiltration trench.

The expected mean times for reaching the mass loading thresholds are observed to remain unaffected by varying trench depths since flow (and mass of copper and zinc) through the underlying filter is not related to the storage volume.

### 7.1.3 Thickness of Filter Media

Schueler (1987) illustrates in a well-known infiltration trench configuration diagram (shown in Figure 1.1) that the depth of the underlying sand filter at the bottom of the trench should range between 15 and 30 cm. Unlike the design criteria discussed in Section 7.1.1 and 7.1.2, a suitable depth of the underlying sand filter is not dependent on specific meteorological or hydrologic parameters. It is accepted in this study that improved directions are required for the sizing of underlying filter beds. Therefore, the present study highlights the effect that shallow or deep filter beds may have on the overall long-term BMP facility performance. Statistical results for model simulations are summarized in Table 7.3 and Figure 7.3.

Table 7.3 and Figure 7.3 show that by decreasing the depth of the filter bed to 10 and 15 cm, the mean time to reach the clogging threshold is approximately 14% and 8% earlier, respectively, compared to the base case model results. Conversely, increasing the depth of the filter bed to 25 and 30 cm extends the duration of non-clogged filter bed conditions by approximately 25% and 38%, respectively. In addition, increasing the depth of the filter bed improves the filter’s capacity to retain deposited particles as well as extends the time required for the filter to reach its capacity to retain particles. The modelling approach applied in this study assumes that shallow filter depths result in rapid clogging (i.e., a steeper long-term clogging profile in Figure 4.2) and deeper filters allow for more time to reach the defined clogging threshold.
Table 7.3 Effect of filter media thickness on model failure modes – time to failure (years)

<table>
<thead>
<tr>
<th>Filter (cm)</th>
<th>Clogging</th>
<th>Overflow (no underdrain)</th>
<th>Overflow (underdrain)</th>
<th>Copper Loading</th>
<th>Zinc Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Stdev</td>
<td>Mean</td>
<td>Stdev</td>
<td>Mean</td>
</tr>
<tr>
<td>10</td>
<td>12.3</td>
<td>3.7</td>
<td>6.7</td>
<td>2.0</td>
<td>7.2</td>
</tr>
<tr>
<td>15</td>
<td>13.1</td>
<td>3.5</td>
<td>6.9</td>
<td>1.9</td>
<td>7.4</td>
</tr>
<tr>
<td>20</td>
<td>14.3</td>
<td>3.5</td>
<td>7.3</td>
<td>1.9</td>
<td>8.0</td>
</tr>
<tr>
<td>25</td>
<td>17.9</td>
<td>3.2</td>
<td>8.8</td>
<td>1.8</td>
<td>9.5</td>
</tr>
<tr>
<td>30</td>
<td>19.8</td>
<td>3.2</td>
<td>9.5</td>
<td>1.8</td>
<td>10.3</td>
</tr>
</tbody>
</table>

*all statistics are in years of simulation time

Stdev (standard deviation)

Figure 7.3 illustrates how changes in the filter bed depth alter the maximum capacity to retain particles. In addition, to assist in identifying changes in the filter bed’s capacity to retain particles, the input concentration is held constant at 120 mg/L and the same time series of rainfall events is used for each simulation (similar to the sensitivity analysis performed in Chapter 5).
Figure 7.4 illustrates that by increasing the depth of the filter, more time is required to reach the maximum capacity. The maximum capacity to retain particles in the filter bed is indicated by the horizontal line associated with a specific depth of filter in Figure 7.4. In addition, as the depth of the filter increases (i.e., from 10 to 30 cm) a larger cumulative mass of suspended solids can be retained in the filter. Recall that clogging in this study is not defined by the maximum capacity of the filter to retain solids, rather clogging is defined by flow reduction.

As decreasing the depth of the filter results in rapid clogging of the infiltration trench, the estimated mean times for the first overflow event to occur, for designs with and without the underdrain, are shortened. When the filter is reduced to 15 cm, the first overflow event for designs with and without the underdrain occurs earlier by approximately 8% and 10%, respectively. However, when the filter is only reduced to 20 cm, the first overflow events occur earlier by approximately 8% and 5%, respectively, for cases with and without the underdrain. Shorter times until trench overflow are achieved since flow rate through the subsurface decreases more rapidly. Conversely, increasing the depth of the filter enables the trench to operate without overflowing for a longer period. For depths of 30 and 35 cm, the time until the first overflow event is extended by approximately 20% and 30%, respectively, for cases with an underdrain and by 19% and 29% for cases without the underdrain. It is
expected that better performance is achieved here since a greater magnitude of flow rate is maintained for a longer period of time since clogging is delayed.

As previously discussed, decreasing the depth of the filter media results in rapid flow rate reduction through the subsurface of the trench and as a result, earlier clogging. Since flow through the bottom of the trench is anticipated to be reduced earlier in the simulation, mass loading thresholds for both copper and zinc are not reached until a slightly later time compared to the base case model results. Excess water that results from a limited flow rate through the filter will accumulate as stored water in the overlying trench, until an overflow or spill results from exceeding the trench capacity. This accumulation occurs from a steady inflow of runoff into the trench followed by a limited outflow through the filter due to clogging. A 3% and 11% increase in time to fail for copper and zinc are observed when the filter depth is decreased to 10 cm. The observed standard deviations of the copper and zinc failure time are 1.3 and 2.9, respectively, which are relatively high compared to standard deviations estimated from previous simulations. When investigating the simulation results in further detail, it is observed that there are a number of replications where the times to reach the loading thresholds occur later than the 30 year simulation time. Times to fail that occur past the 30 year simulation mark are achieved when the filter clogs prior to the groundwater failure event; this trend occurs in less than 1% of the replications for both simulations. As a result, a large standard deviation is achieved. Increasing the depth of the filter to 30 cm results in similar times to reach the copper and zinc thresholds, 3% and 5%, respectively; these times to fail are marginally earlier. This outcome results since a deeper filter extends the life of the filter under non-clogged conditions and allows for more flow to pass through the subsurface for a longer period. In this simulation, there are no replications that result in clogging prior to groundwater failure. Note that it is likely that assigning different mass thresholds will arrive at vastly different conclusions for this failure mode.

7.1.4 Pre-treatment Efficiency

The last sensitivity analysis presented and discussed in this study is the influence of pre-treatment efficiency on the long-term performance of an infiltration facility. As previously
mentioned in Chapter 5, pre-treatment facilities situated upstream from the infiltration trench are capable of removing approximately 25% to 30% of suspended solids from the infiltrating urban runoff (Barr Engineering Company, 2001). Pre-treatment facilities are recommended at sites with an anticipated heavy loading of suspended solids. In cases where pre-treatment facilities are not maintained regularly, their removal efficiencies degrade and consequently, the loadings of suspended solids to the infiltration trench increase. Therefore, the pre-treatment efficiency is perturbed by varying degrees from the base case value, 25%. The first simulation where the pre-treatment efficiency is operating at 15% represents a situation where the pre-treatment facility is no longer operating under design conditions and conversely, the simulation where the pre-treatment facility is operating at 35% represents a situation where optimal performance is achieved. Statistical results for model simulations are summarized in Table 7.4 and Figure 7.5.

Table 7.4 and Figure 7.5 illustrate that when the pre-treatment facility is not operating under design conditions (i.e., a pre-treatment efficiency of 15%) the estimated mean time to clogging is approximately 1 year earlier than the base case model results which represents a 7% decrease in time. It is possible that earlier clogging of the underlying filter occurs due to an increase in loading of suspended solids from infiltrating urban runoff. However, circumstances where pre-treatment facilities are operating under design conditions (i.e., 35% efficiency), the mean time to clogging is delayed by 1.5 years, which represents a 10% increase in time. As a result, maintaining high removal efficiencies for suspended solids in the pre-treatment facility reduces the loading of solids on the infiltration trench and delays clogging and related failure events. Overall, Figure 7.5 illustrates that as the pre-treatment efficiency increases, the time to clogging gradually increases.
Table 7.4 Effect of pre-treatment efficiency on model failure modes – time to failure (years)

<table>
<thead>
<tr>
<th>Pre-treatment (%)</th>
<th>Clogging</th>
<th>Overflow (no underdrain)</th>
<th>Overflow (underdrain)</th>
<th>Copper Loading</th>
<th>Zinc Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stat: Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
<td>Mean Stdev</td>
</tr>
<tr>
<td>15</td>
<td>13.3 3.3</td>
<td>6.9 1.8</td>
<td>7.4 1.8</td>
<td>3.1 0.29</td>
<td>3.9 1.7</td>
</tr>
<tr>
<td>20</td>
<td>13.8 3.2</td>
<td>7.1 1.7</td>
<td>7.7 1.8</td>
<td>3.1 0.27</td>
<td>3.8 0.92</td>
</tr>
<tr>
<td>25</td>
<td>14.3 3.5</td>
<td>7.3 1.9</td>
<td>8.0 2.0</td>
<td>3.1 0.26</td>
<td>3.7 0.36</td>
</tr>
<tr>
<td>30</td>
<td>15.2 3.6</td>
<td>7.7 1.9</td>
<td>8.3 2.0</td>
<td>3.1 0.24</td>
<td>3.6 0.36</td>
</tr>
<tr>
<td>35</td>
<td>15.8 3.9</td>
<td>8.1 2.1</td>
<td>8.7 2.1</td>
<td>3.0 0.25</td>
<td>3.6 1.2</td>
</tr>
</tbody>
</table>

*all statistics are in years of simulation time
Stdev (standard deviation)

Figure 7.5 Effect of pre-treatment efficiency on model failure modes – time to failure (years)

Similar to the modelling results achieved when the filter depth was decreased, reducing pre-treatment efficiency results in slightly earlier times to reach the first overflow event for both cases with and without an underdrain. The estimated mean time to reach the first overflow failure occurs approximately 0.6 (8% decrease) and 0.4 years (5% decrease) earlier for cases with and without an underdrain when the pre-treatment efficiency is reduced to 15%. In addition, maximizing the pre-treatment efficiency to 35% extends the estimated mean time without overflows by less than 1 year, which represents a 9% and 10% increase for both design alternatives. The standard deviations increase slightly as the mean time increases and vice versa.
Lastly, the results in Table 7.4 and Figure 7.5 demonstrate minimal changes in the estimated times required to reach the defined groundwater mass loading thresholds. The marginal observed changes are believed to be associated with the change in flow rate through the subsurface of the infiltration trench (due to clogging). Similar to the sensitivity analysis where the depth of the filter is decreased to 10 cm, these simulation results indicate that there may be some instances (for zinc) where early clogging of the underlying filter prevents a groundwater exceedence. Generally, as the pre-treatment efficiency increases, the standard deviation decreases since the number of replications where a groundwater failure event occurs after the 30 year mark reduces. However, when the pre-treatment efficiency is increased to 35%, one replication is observed where the zinc failure threshold is exceeded after the 30 year mark, which results in a high standard deviation, 1.2. Also observed in this replication, clogging is reached after approximately 5 years, which results in the late zinc failure and subsequent high standard deviation. This result may be attributable to the sequence of random generated numbers for the model replication.

7.2 RECOMMENDATIONS FOR IMPROVING DESIGN CRITERIA

From the sensitivity analysis completed in Section 7.1, the following section presents recommendations for improved design criteria that may extend the life and enhance the performance of infiltration trenches. The sensitivity analysis reveals that certain design parameter values are better suited to increase the longevity of the BMP facility. This section investigates and discusses these parameters in detail.

7.2.1 Area of the Infiltration Trench

Based on simulation results, although it is observed that the effective filtration surface area does not affect the estimated time of clogging, increasing the surface area does, however, provide better safety and protection against overflow events; the occurrence of the first overflow event is delayed with an increase in surface area. Nonetheless, increasing the trench area from initial design considerations (90 m$^2$) results in earlier exceedence of the mass
loading threshold and may cause earlier contamination of the underlying aquifer. Therefore, a tradeoff is identified between volume and water quality control when considering improved surface area recommendations for the BMP facility.

7.2.2 Depth of Infiltration Trench

No change in the time to achieve the clogging threshold is observed by varying the depth of the trench. Also, an earlier risk of degrading the groundwater quality is not anticipated to occur by changing the depth of the trench. However, an increase to the trench depth did appear to provide better protection against overflow events. Therefore, it is recommended that to provide better volume control without risking earlier degradation of the groundwater quality, the depth of the trench should be augmented rather than changing the effective filtration surface area.

7.2.3 Thickness of Filter Media

Implementing a thicker filter bed is likely to delay a clogging event as well as delay the first overflow event; however, it is anticipated that a slightly earlier groundwater exceedence may result when increasing the thickness of the filter. Although an earlier groundwater exceedence may occur, this difference in time is marginal. Therefore, it is recommended that a thicker filter bed be implemented to ensure a delay in clogging and overflow events as well as impose minimal changes to the groundwater quality compared to the base case simulation results.

7.2.4 Pre-treatment Efficiency

Lastly, by maintaining a higher pre-treatment efficiency (i.e., 35%), the occurrence of filter clogging and timing of the first overflow event are delayed. By improving upstream pre-treatment efficiency, minimal differences are anticipated in terms of additional protection
against groundwater quality failure. Therefore, upstream pre-treatment facilities should be maintained at a high level of efficiency.

### 7.2.5 Improved Design Strategy

Based on the simulation results achieved through the macroscopic modelling approach, the probabilistic analysis, and the sensitivity analysis, to increase the longevity of an infiltration trench, this study offers the following recommendations for trench design:

- Increase the depth of the trench (rather than the surface area) to a maximum depth of 3 m to provide additional volume for urban runoff storage without compromising groundwater quality;
- Maintain improved pre-treatment efficiency (approximately 35%); and
- Implement a thicker sand filter (approximately 30 cm) to delay clogging of the subsurface.

A final long-term continuous simulation was conducted applying the aforementioned recommendations. Compared to the base case simulation results, the estimated mean time of clogging is extended from approximately 14 years to 23.5 years and the first overflow event for design alternatives with and without the underdrain is extended from approximately 8 and 7 years to 14 and 13 years, respectively. Clogging may occur as early as 16 years of operation; however, overflow events are expected after 10 years of operation. There exists less than 1% chance of overflow events occurring prior to 10 years of operation in both cases with and without the underdrain. Lastly, a slight decrease in the estimated times to groundwater failure were observed; for both copper and zinc, failure times decreased by approximately 6% and 7%, respectively. Recommendations for better management strategies are more so based on results from the first two failure modes (clogging and overflow) since the investigation of groundwater failure is carried out in this study to simply demonstrate the capability of this methodology of characterizing the third failure mode.
7.3 RECOMMENDATIONS FOR MANAGEMENT REGIMES

Recommendations for long-term maintenance regimes are dependent on selected design criteria. Infiltration trenches are expected to endure for 8 to 19 years (Tahoe Basin Interagency Roadway Maintenance and Operations Committee, 2001); however, from the present study, it is evident that based on common design practices as well as the improved design recommendations, failure events will occur prior to the end of this expected lifetime. The following section recommends long-term management practices that are based on the improved design strategy in order to extend the longevity of the BMP facility, while avoiding early, frequent failure events.

Based on this study and the suggested design strategy, it is recommended that complete rehabilitation of the infiltration trench be carried out after 10 years of operation. Rehabilitation that takes place after 10 years risks the occurrence of frequent overflow events and potential ponding at the surface for trench designs without an underdrain. Although it is estimated that the underlying filter will not be classified as clogged within 10 years of operation, a reduction in flow rate would likely transpire and reduce the drainage capacity from the trench, increasing the risk of overflow events and ponding.

Upstream pre-treatment facilities should be monitored and maintained frequently (i.e., monthly or quarterly) to ensure that they are operating at design targets, such as 35% removal of suspended solids. Neglecting to maintain the pre-treatment facility may lead to overloading the infiltration trench with suspended solids from the urban runoff and premature clogging. In addition, the non-woven filter fabric that is situated slightly below the surface of the trench should be maintained frequently (monthly or quarterly) to prevent clogging. Removal of sediment from this filter fabric can be done relatively easily given that it is situated near the surface, and minimal stone fill will need to be removed.

7.4 SUMMARY

The third and final research objective is presented and discussed in this Chapter. Considering results from the first two research objectives, modelling the long-term performance and
completing a probabilistic analysis, a sensitivity analysis was completed to improve design
criteria and long-term management regimes. Four design parameters (model inputs) were
selected as sensitive parameters and varied to determine their influence on the long-term
performance of infiltration trenches. These design parameters include: infiltration trench
area, depth of infiltration trench, depth of filter media, and pre-treatment efficiency.

A sensitivity analysis revealed that clogging of the underlying filter media is not affected by
changing the trench area or the trench depth, rather the depth of the filter media and pre-
treatment efficiency appear to have an impact on the longevity of the BMP facility in terms
of clogging. Earlier overflow events occur as the volume of the overlying storage reservoir is
reduced; this may be achieved by either decreasing the surface area or trench depth.
However, decreasing the depth of the trench does not significantly affect the mass loading of
dissolved contaminants to the underlying aquifer. Pre-treatment facilities degrade over time
and reduced sediment removal efficiencies would lead to earlier clogging of the granular
filter which would in turn cause earlier overflow events.

Based on the sensitivity analysis, an improved design strategy is suggested that includes
increasing the depth of the infiltration trench to provide additional storage for urban runoff
rather than increasing the area of the trench. In addition, shallow granular filters should not
be used as this leads to earlier development of the cake layer and subsequent clogging. Pre-
treatment facilities should be monitored routinely to maintain design conditions. Lastly, it is
concluded in this study that infiltration trenches do not endure with adequate performance
past 10 years and, therefore, should undergo a complete rehabilitation during this time to
avoid further risks to the surrounding environment.
Chapter 8

CONCLUSIONS AND RECOMMENDATIONS AND FURTHER RESEARCH

8.1 CONCLUSIONS AND RECOMMENDATIONS

To provide more insight into the long-term performance of stormwater infiltration trenches, this study applied a continuous macroscopic filtration model that incorporates clogging mechanisms within the filter media. Performance criteria were identified and evaluated including: filter flow, frequency and magnitude of overflow events, and mass loading of dissolved contaminants into groundwater. This study defined the following three research objectives: profiling the long-term performance of infiltration trenches, using a probabilistic analysis to determine the time of failure for each failure mode, and carrying out a sensitivity analysis to achieve an improved design strategy that extends the longevity of infiltration trenches. The modelling methodology developed and applied in this study established estimated times of failure for each failure mode while minimizing the number of microscopic input and empirical parameters.

The following conclusions are made based on the developed modelling methodology and completed analyses:

1. Stormwater infiltration trenches can clog prior to an expected lifetime of 8 to 19 years and although adequate volumetric control was observed during the initial years of operation, clogging (i.e., flow reduction) results in early and frequent overflow events. From the base case simulation results presented in Sections 5 and 6, clogging of the underlying sand filter may occur as early as after 8 years of operation;
however, this system has the highest probability of failure between 14 and 16 years of operation.

2. Overflow events can occur as early as after 4 years of operation, with the highest probability of failure after the 8th year of operation. Underdrains are implemented as a safety measure to reduce the occurrence of overflow events once the underlying filter clogs and the drainage capacity is depleted. Although the first overflow event may be delayed by only one year, underdrains are better suited to reduce ponding at the surface of the infiltration trench.

3. Infiltration trenches may employ sand filters to remove suspended solids from urban runoff; however, they are not suitable for the removal of contaminants in their dissolved form, such as zinc and copper. This thesis presents a methodology for characterizing the mass loading of zinc and copper from the bottom of the infiltration trench and reveals that mass loading is restricted by the gradual reduction in flow through the filter.

4. From the preliminary sensitivity analysis completed in Chapter 5, deposition of fine sediment which results in a large clogging factor leads to early clogging of the underlying filter. Conversely, large flows from the bottom of the infiltration trench are maintained when the mean deposited particle is larger and a smaller clogging factor results. Higher flows through the bottom of the trench are also maintained when a high level of pre-treatment is maintained upstream.

5. By further investigating the sensitivity of four design parameters (trench area, trench depth, filter depth, and pre-treatment efficiency), this study show that occurrence of the first overflow event can be delayed by almost 2 years without compromising groundwater quality by increasing trench depth. Although increasing the surface area of the infiltration trench increases the storage volume, earlier groundwater failure may result. Augmenting the depth of the filter bed from 20 to 30 cm delayed clogging of the subsurface by over 5 years as more solids were observed to deposit in deeper filter beds prior to the development of the cake layer. A high level of pre-treatment
efficiency will also delay clogging by reducing the mass loading of suspended solids into the infiltration trench.

6. Implementing an improved design strategy based on results from the sensitivity analysis revealed a significant delay in clogging of the underlying filter, however, early overflow events and groundwater failure may persist. With the improved design strategy, these BMP devices are not likely to operate properly after 10 years of implementation. Without proper maintenance of pre-treatment facilities and filter fabrics, earlier failure is expected to result.

The following trench design and maintenance recommendations are made based on the above analyses and conclusions:

1. Infiltration trenches should not be sited at locations where urban runoff is primarily laden with fine suspended solids as this will cause early failure due to clogging.

2. High pre-treatment efficiencies should be maintained to extend the longevity of the infiltration trench under design conditions. An improved design strategy recommends maintaining upstream removal efficiency at 35%.

3. To increase the storage volume of the trench and consequently delay overflow events, the depth of the trench should be increased rather than increasing the trench area. The suggested improved design strategy recommends increasing the depth of the trench from 2 to 3 m.

4. The depth of the filter bed should be increased to delay development of the cake layer; shallow filter beds result in earlier clogging. An improved design strategy recommends increasing the depth of the filter bed from 20 to 30 cm.


5. Underdrains should be used at the base of the infiltration trench to reduce ponding and frequency of overflow events if the drainage capacity of the underlying filter degrades due to clogging.

6. Filter fabrics situated at 0.3 m below the trench surface should be maintained monthly or quarterly to reduce clogging at the surface of the trench. Regular cleaning of pre-treatment facilities as well as overlying filter fabrics would extend the operating life of the infiltration trench to 10 years.

7. Infiltration trenches should undergo complete rehabilitation every 10 years to avoid clogging and frequent overflow events that may result in the degradation of surrounding water courses. It is evident from this study that assessing the longevity of infiltration trenches should not be carried out by simply investigating one performance criterion.

8.2 FUTURE WORK

Given the limitations and assumptions referenced throughout this document, this study recommends that the following future work be completed to enhance the major findings:

1. Unsaturated flow conditions during interevent rainfall periods should be considered in the drainage model applied in this study. The present study is a preliminary investigation and, therefore, does not fully incorporate drainage of runoff through the filter, but rather assumed a constant flow through the filter during interevent periods.

2. Evapotranspiration of urban runoff from the vegetative material planted above the trench should be incorporated into the model. The present study assumes that all urban runoff infiltrates into the trench; however, Davis et al. (2009) reported on a field study conducted by Sharkey (2006) where evapotranspiration was found to account for the fate of 15 to 20% of all inflow water on an annual basis in Louisburg, North Carolina. In addition, infiltration and evapotranspiration together may account for upwards of 50 to 90% of the fate of urban runoff; however, this depends on the in
situ soil type, media depth and type, and the drainage configuration (Heasom et al., 2006; Hunt et al., 2006) and climate.

3. The present study establishes an improved design strategy, as a result of a preliminary investigation, that can extend the longevity of the infiltration trench. Future work may include the development a mathematical model that optimizes the system performance.

4. An economic analysis is recommended for future work. Frequent maintenance and monitoring as well as long-term rehabilitation of the BMP facility can be costly. Life cycle costs should be investigated and considered while selecting an appropriate design for the infiltration trench. In addition, maintenance plans should be determined to minimize the long-term costs of the infiltration trench, delaying expensive rehabilitation.

5. The long-term performance of infiltration trenches as determined in this study should be compared and validated with existing field data where and when available.


REFERENCES


REFERENCES


REFERENCES


REFERENCES


Sub Continuous_Probability()
'
' Continuous_Probability Macro
' Macro recorded 10/03/2009 by Caitlin
**********************************************************************

'THIS SIMULATION CALCULATES A LONG-TERM SIMULATION OF RAINFALL EVENTS,
INCLUDING EVENT MEAN CONCENTRATIONS BASED ON LONG-TERM
'STATISTICS FROM ADAMS & PAPA (2000)

Dim Iteration As Object
Dim PercentComplete As Single
Dim seedValue As Double

Iteration = 0
PercentComplete = 0
UserForm1.Show() ' Show progress bar
Randomize()
seedValue = 12345

'REFORMATING SPREADSHEET AFTER EACH SIMULATION
Worksheets("Analysis ii_Table 1").Range("A7:F30000").ClearContents()
Worksheets("Analysis i_Model Output").Cells(16, 15).ClearContents()
Worksheets("Analysis i_Table 1").Range("A12:P30000").ClearContents()
Worksheets("Analysis ii_Table 1").Range("A7:C40000").ClearContents()
Worksheets("Analysis i_Table 2").Range("A11:P30000").ClearContents()
Worksheets("Analysis i_Table 3").Range("A12:P30000").ClearContents()
Worksheets("Analysis i_Table 4").Range("A12:P30000").ClearContents()
Worksheets("Analysis ii_Table 1").Range("A8:H20000").ClearContents()
Worksheets("Analysis ii_Table 2").Range("b6:I30000").ClearContents()
Worksheets("Analysis ii_Table 2").Range("q8:AC30000").ClearContents()

'REPEATING SIMULATION FOR X NO. OF TRIALS
For t = 1 To Worksheets("Analysis i_Table 1").Cells(1, 14).Value

  'UPDATE PROGRESS BAR
  UserForm1.Label2.Width = 250 * PercentComplete
  UserForm1.Label4.Caption = Round(PercentComplete * 100, 0) & " % Complete"
  DoEvents()
  Application.ScreenUpdating = False

  'INTRODUCING ALL DEPENDENT AND INDEPENDENT VARIABLES FOR SIMULATION

  'time counters, and code functions
  Dim NumEvents As Object
  Dim Iterations As Integer
  Dim Count As Integer  'COUNTER FOR THE CUMULATIVE HOURS OF RAINFALL
  Dim NewIncr As Integer
  Dim randomNumber As Double
  Dim A() As Object
  Dim B() As Object
  Dim m_slope As Single
  Dim b_intercept As Single
  Dim Events As Single
  Dim ClogTime As Single
  Dim Tstop As Object
  Dim TCumul As Object
Dim TcopperPass As Single
Dim TzincPass As Single
Dim TimeLag As Long
Dim Year As Integer
Dim Tevent As Object

' model design parameters
Dim Density As Single
Dim Pretreatment As Single  ' REMOVAL EFFICIENCY FROM UPSTREAM PRETREATMENT MECHANISMS
Dim FilterFabric As Single  ' REMOVAL EFFICIENCY FROM OVERLYING NON-WOVEN FILTER FABRICS
Dim Ko As Single    ' CLEAN BED HYDRAULIC CONDUCTIVITY OF THE GRANULAR FILTER
Dim Porosity As Single
Dim Area As Single  ' AREA OF THE INFILTRATION TRENCH (DEPENDENT ON RAINFALL AND DRAINAGE)
Dim RunCoef As Single
Dim CatchmentArea As Single
Dim ClogParameter As Single
Dim Velocity As Single
Dim FilterCoefficient As Single
Dim FilterDepth As Single
Dim Kvalue As Single
Dim CleanPorosity As Single
Dim TotalDepth As Single    ' TOTAL DEPTH OF THE GRANULAR SAND FILTER
Dim Depth As Single ' CUMULATIVE DEPTH FROM Z=0 TO Z=TOTALDEPTH
Dim Delta As Single ' INCREMENTAL DEPTH OF FILTER

' uncertain parameters from probability distributions
Dim LambdaInterEvent As Single
Dim LambdaIntensity As Single
Dim LambdaDuration As Single
Dim Intensity As Single
Dim Duration As Single
Dim InterEvent As Single
Dim MuCopper As Single
Dim SigmaCopper As Single
Dim MuZinc As Single
Dim SigmaZinc As Single
Dim EMCaverage As Single
Dim MuEMC As Single
Dim SigmaEMC As Single
Dim RandEMC As Single
Dim InputEMC As Single

' parameters for filter clogging and overflow events
Dim Outflow_start As Single
Dim Outflow_end As Single
Dim HydDepthTotal As Single
Dim DeltaT As Single
Dim Ke As Single
Dim Kd As Single
Dim Kc As Object
Dim CakePorosity As Single
Dim ChangeCakePore As Single
Dim CakeDepth As Single
Dim Head1 As Single
Dim Head2 As Single
Dim TotIncr As Single
Dim TotHydDepth1 As Single
Dim TotHydDepth2 As Single
Dim TrenchDepth As Single
Dim OverflowTime1 As Single
Dim OverflowTime2 As Single
Dim Inflow As Single
Dim HydDepth1 As Single
Dim HydDepth2 As Single
Dim NumOverflow1 As Object
Dim NumOverflow2 As Object
Dim OverflowBerm As Single
Dim CumulDepth As Single
Dim Ke_Ko As Single
Dim Kd_Ko As Single
Dim FilterFlow As Single
Dim VoidFraction As Single
Dim AvgVel As Single

' defining variables for heavy metal breakthrough
Dim ConcSoilCopper As Single
Dim TsoilCopper As Single
Dim MSoilCopper As Single
Dim ConcSoilZinc As Single
Dim TsoilZinc As Single
Dim MSoilZinc As Single
Dim Mcopper As Single
Dim Mzinc As Single
Dim EventZinc As Object
Dim EventCopper As Object
Dim CumulativeZinc As Single
Dim CumulativeCopper As Single

' defining variables for concentration profiles and specific deposition
Dim Ceﬄuent As Single
Dim Ceﬄuent_z As Single
Dim Ceﬄuent_zo As Single
Dim C_Co As Long
Dim SpecificDeposit As Single
Dim SumDeposit As Single
Dim DepthDeposit As Single
Dim WholeDeposit As Single
Dim LongTermDeposit As Single
Dim CFraction As Object
Dim FSpecifc As Object
Dim MassDeposit As Single
Dim SumMassDeposit As Single

' defining variables for arrays to store data for future use - speeds up simulations
ReDim EMC_Array(0) As Single
ReDim Intensity_array(0) As Single
ReDim TimeLag_Array(0) As Single
ReDim DepthDeposit_Array(0) As Single
ReDim TCumul_Array(0) As Single
ReDim WholeFilter_Array(0) As Single
ReDim LongTerm_Array(0) As Single
ReDim CakeDepth_Array(0) As Single
ReDim CumulDepth_Array(0) As Single
ReDim Ke_Array(0) As Single
ReDim Kd_Array(0) As Single
ReDim Kc_Array(0) As Single
ReDim Porosity_Array(0) As Single
ReDim Head1_Array(0) As Single
ReDim Head2_Array(0) As Single
ReDim FilterFlow_Array(0) As Single
ReDim Area_Array(0) As Single
ReDim AvgVel_Array(0) As Single
ReDim CumulativeMassDeposit_Array(0) As Single
ReDim HydDepth1_Array(0) As Single
ReDim HydDepth2_Array(0) As Single
ReDim OverflowTime1_Array(0) As Single
ReDim OverflowTime2_Array(0) As Single
ReDim Copper_Array(0) As Single
ReDim Zinc_Array(0) As Single
ReDim AnnualZinc_Array(0) As Single
ReDim AnnualCopper_Array(0) As Single
ReDim Duration_Array(0) As Single
ReDim EventCopper_Array(0) As Single
ReDim EventZinc_Array(0) As Single
ReDim CwCopper_Array(0) As Single
ReDim CwZinc_Array(0) As Single
ReDim SoilCopper_Array(0) As Single
ReDim SoilZinc_Array(0) As Single
ReDim InterEvent_Array(0) As Single
ReDim Tevent_Array(0) As Single
ReDim CumulativeZinc_Array(0) As Single
ReDim CumulativeCopper_Array(0) As Single
ReDim SumMassDeposit_Array(0) As Single
ReDim SumDeposit_Array(0) As Single
ReDim CakePorosity_Array(0) As Single

'DEFINING INITIAL VARIABLES AS PER EMC SIMULATION DATA
MuEMC = Worksheets("Analysis ii_Table 1").Cells(4, 7).Value
SigmaEMC = Worksheets("Analysis ii_Table 1").Cells(4, 8).Value

'DEFINITION INITIAL VARIABLES AS PER RAINFALL STATISTICS (ADAMS & PAPA, 2000)
LambdaInterEvent = Worksheets("Analysis ii_Table 1").Cells(4, 2).Value
LambdaDuration = Worksheets("Analysis ii_Table 1").Cells(3, 2).Value
LambdaIntensity = Worksheets("Analysis ii_Table 1").Cells(2, 2).Value

'DEFINING STATISTICS FOR COPPER AND ZINC
MuCopper = Worksheets("Analysis ii_Table 1").Cells(4, 16).Value
SigmaCopper = Worksheets("Analysis ii_Table 1").Cells(4, 17).Value
MuZinc = Worksheets("Analysis ii_Table 1").Cells(4, 19).Value
SigmaZinc = Worksheets("Analysis ii_Table 1").Cells(4, 20).Value

'DEFINING INFILTRATION TRENCH PARAMETERS FOR TREATMENT AND GEOMETRY
Pretreatment = Worksheets("Analysis i_Model Output").Cells(14, 15).Value
FilterFabric = Worksheets("Analysis i_Model Output").Cells(15, 15).Value
Area = Worksheets("Analysis i_Model Output").Cells(10, 15).Value
CatchmentArea = Worksheets("Analysis i_Model Output").Cells(6, 15).Value
RunCoef = Worksheets("Analysis i_Model Output").Cells(12, 10).Value
Qeffluent = Worksheets("Analysis i_Model Output").Cells(13, 10).Value

'INITIALLY, THE FLOW AT SUBSURFACE IS EQUAL TO START VALUE WITH NO CLOGGING
CleanPorosity = Worksheets("Analysis i_Model Output").Cells(7, 5).Value
TotalDepth = Worksheets("Analysis i_Model Output").Cells(5, 5).Value
FilterCoefficient = Worksheets("Analysis i_Model Output").Cells(6, 5).Value
Velocity = Worksheets("Analysis i_Model Output").Cells(8, 5).Value
Delta = Worksheets("Analysis i_Model Output").Cells(14, 5).Value
Kvalue = Worksheets("Analysis i_Model Output").Cells(13, 5).Value
DeltaT = Worksheets("Analysis i_Model Output").Cells(12, 5).Value
Density = Worksheets("Analysis i_Model Output").Cells(11, 5).Value
DepthDeposit = 0
CakePorosity = Worksheets("Analysis i_Model Output").Cells(17, 5).Value
ClogParameter = Worksheets("Analysis i_Model Output").Cells(10, 5).Value
Ko = Worksheets("Analysis i_Model Output").Cells(15, 5).Value
VoidFraction = Worksheets("Analysis i_Model Output").Cells(9, 15).Value
TrenchDepth = Worksheets("Analysis i_Model Output").Cells(11, 15).Value
OverflowBerm = Worksheets("Analysis i_Model Output").Cells(7, 15).Value

'DEFINING INITIAL RAINFALL INTENSITY AT TIME=0
Intensity = 0

'DEFINING INITIAL TIME PARAMETERS
SumEMC = 0
Tstop = Worksheets("Analysis i_Table 1").Cells(2, 14).Value * 24 * 365
TCumul = 0
TSum = 0
Tevent = 0

'DEFINING INITIAL COUNTER PARAMETERS
NumEvents = 0
Count = -1
InColumn = 6
NumOverflow = 0

'**********************************************************************
'SAMPLING DISTRIBUTIONS TO SIMULATE RAINFALL AND EVENT MEAN CONCENTRATIONS
'SAMPLING DISTRIBUTION TO SIMULATE INPUT OF DISSOLVED METALS:COPPER AND ZINC
'ALL WATER QUALITY DATA ARE SAMPLED FROM LOGNORMAL DISTRIBUTIONS, WHILE
RAINFALL DATA IS SAMPLED FROM EXPONENTIAL
'**********************************************************************

Do
    NumEvents = NumEvents + 1
    randomNumber = MRG32k3a(seedValue)
    InterEvent = (-1 / LambdaInterEvent) * (Log(1 - randomNumber) / Log(2.718282)) 'SAMPLING INTEREVENT TIME
    Intensity = 0   'ENSURING THAT EACH SIMULATION, RAINFALL INTENSITY IS
ZERO DURING INTEREVENT PERIODS
    InterEvent_Array(UBound(InterEvent_Array)) = InterEvent
    ReDim Preserve InterEvent_Array(UBound(InterEvent_Array) + 1)
    Do
        TCumul = TCumul + 1
        Intensity_array(UBound(Intensity_array)) = Intensity    'RAINFALL
INTENSITY IS 0 mm/hr DURING INTEREVENT TIME
        ReDim Preserve Intensity_array(UBound(Intensity_array) + 1)
        Loop Until TCumul > InterEvent + TSum - 1
        randomNumber = MRG32k3a(seedValue)
        Duration = (-1 / LambdaDuration) * (Log(1 - randomNumber) / Log(Exp(1))) 'SAMPLING DURATION OF RAINFALL
        Duration_Array(UBound(Duration_Array)) = Duration
        ReDim Preserve Duration_Array(UBound(Duration_Array) + 1)
        randomNumber = MRG32k3a(seedValue)
        Intensity = (-1 / LambdaIntensity) * (Log(1 - randomNumber) / Log(Exp(1))) 'SAMPLING INTENSITY OF RAINFALL
        randomNumber = MRG32k3a(seedValue)
        RandEMC = WorksheetFunction.LogInv(randomNumber, MuEMC, SigmaEMC)
        SumIntensity = 0
        InputEMC = RandEMC * (1 - Pretreatment) * (1 - FilterFabric)
        'SAMPLING EMCs OF COPPER AND ZINC DURING RAINFALL EVENTS- CONSERVATIVE CONTAMINANT
randomNumber = MRG32k3a(seedValue)
RandCopper = WorksheetFunction.LogInv(randomNumber, MuCopper, SigmaCopper)
randomNumber = MRG32k3a(seedValue)
RandZinc = WorksheetFunction.LogInv(randomNumber, MuZinc, SigmaZinc)

Do

SumIntensity = SumIntensity + Intensity
TCumul_Array(UBound(TCumul_Array)) = TCumul / 8760
ReDim Preserve TCumul_Array(UBound(TCumul_Array) + 1)
EMC_Array(UBound(EMC_Array)) = InputEMC
ReDim Preserve EMC_Array(UBound(EMC_Array) + 1)
Intensity_array(UBound(Intensity_array)) = Intensity
ReDim Preserve Intensity_array(UBound(Intensity_array) + 1)
Copper_Array(UBound(Copper_Array)) = RandCopper
ReDim Preserve Copper_Array(UBound(Copper_Array) + 1)
Zinc_Array(UBound(Zinc_Array)) = RandZinc
ReDim Preserve Zinc_Array(UBound(Zinc_Array) + 1)

TCumul = TCumul + 1
Count = Count + 1

Loop Until TCumul > (InterEvent + Duration + TSum - 1)

TSum = TCumul

Tevent_Array(UBound(Tevent_Array)) = TCumul / 24 / 365
ReDim Preserve Tevent_Array(UBound(Tevent_Array) + 1)

Loop Until TSum > Tstop - 1

'CREATING AN ARRAY FOR SIMULATION TIME INCLUDING TIME LAG DUE TO POROUS MEDIA

For i = 0 To Count

TimeLag = i - (CleanPorosity / Velocity) * TotalDepth
TimeLag_Array(UBound(TimeLag_Array)) = TimeLag
ReDim Preserve TimeLag_Array(UBound(TimeLag_Array) + 1)

Next

'TCALCULATING THE CONCENTRATION PROFILE OF SOLIDS (C/Co) ACROSS THE DEPTH OF THE FILTER ON A LONG-TERM BASIS
'T************************************************************************************************************

TotIncr = TotalDepth / Delta
ReDim CFraction(0 To Count, 0 To TotIncr)

For i = 0 To Count

InColumn = 6
Depth = 0

For j = 0 To TotIncr

'Depth = Worksheets("Analysis i_Table 1").Cells(10, InColumn).Value

If TimeLag_Array(i) < 0 Then
C_Co = 0
Else
If i > 2000 Then

End If

Next
C_Co = 1
Else
    If EMC_Array(i) * TimeLag_Array(i) < 450000 Then 'TO PREVENT
        TOO LARGE VALUES
        C_Co = Exp(Velocity * FilterCoefficient * (EMC_Array(i) / 1000) * Kvalue * TimeLag_Array(i)) / (Exp(FilterCoefficient * Depth) + Exp(Velocity * FilterCoefficient * (EMC_Array(i) / 1000) * Kvalue * TimeLag_Array(i)) - 1)
    Else
        C_Co = 1
    End If
End If
End If

CFraction(i, j) = C_Co
Depth = Depth + Delta
InColumn = InColumn + 1
Next

'**********************************************************************
'CALCULATING THE SPECIFIC DEPOSITION ACROSS THE FILTER DEPTH ON A LONG-TERM
BASIS (NO CLOGGING FACTORS INCLUDED)
'**********************************************************************

SumDeposit = 0
CumulativeMassDeposit = 0
NewIncr = TotIncr - 1
ReDim FSpecific(0 To Count, 0 To NewIncr)
For i = 0 To Count
    k = 0
    SumMassDeposit = 0
    For k = 0 To NewIncr
        SpecificDeposit = Velocity * (CFraction(i, k) - CFraction(i, k + 1)) * EMC_Array(i) / 1000 * DeltaT * 1 / Delta
        'Total mass (kg) of solids deposited in each filter slice at each
time interval
        MassDeposit = SpecificDeposit * Delta * Area * 100 ^ 2 / 1000 / 1000
        FSpecific(i, k) = MassDeposit
        SumMassDeposit = MassDeposit + SumMassDeposit
    Next
    CumulativeMassDeposit = SumMassDeposit + CumulativeMassDeposit
    CumulativeMassDeposit_Array(UBound(CumulativeMassDeposit_Array)) = CumulativeMassDeposit
    SumDeposit = CumulativeMassDeposit / (TotalDepth / 100 * Area)
    SumDeposit = SumDeposit / Density
    SumDeposit_Array(UBound(SumDeposit_Array)) = SumDeposit
ReDim Preserve SumDeposit_Array(UBound(SumDeposit_Array) + 1)
ReDim Preserve CumulativeMassDeposit_Array(UBound(CumulativeMassDeposit_Array) + 1)
Next
'CALCULATING THE TOTAL (WHOLE) SPECIFIC DEPOSITION ACROSS THE FILTER DEPTH ON A LONG-TERM BASIS- INCORPORATES A CLOGGING MODEL
'(Y=mX+b)

'MUST CALCULATE THE SLOPE AND INTERCEPT OF THE EARLY SPECIFIC DEPOSITION IN THE DEPTH FILTER TO PROJECT THE DEPOSITION INTO THE FUTURE FOR A LONG-TERM SIMULATION

ReDim A(200) 'THE NUMBER OF CELLS IN THE ARRAY MUST BE CHANGED DEPENDING ON THE FILTER DEPTH (10CM=100, 20CM=200, 30CM=550)

ReDim B(200)

For i = 0 To 200
    A(i) = SumDeposit_Array(i)
    B(i) = TCumul_Array(i)
Next

m_slope = Application.WorksheetFunction.Slope(A(), B())
b_intercept = Application.WorksheetFunction.Intercept(A(), B())

For i = 0 To Count
    If TimeLag_Array(i) > 0 Then
        WholeDeposit = m_slope * TCumul_Array(i) + b_intercept
    Else
        WholeDeposit = 0
    End If
    If WholeDeposit < 0 Then
        WholeDeposit = 0
    End If
    WholeFilter_Array(UBound(WholeFilter_Array)) = WholeDeposit
    ReDim Preserve WholeFilter_Array(UBound(WholeFilter_Array) + 1)
Next

'OUTPUTS THE SPECIFIC DEPOSITION PROFILE (COMBINING BOTH DEPTH FILTER AND WHOLE FILTER PROFILES)

For i = 0 To Count
    If i < 200 Then
        LongTermDeposit = SumDeposit_Array(i)
    Else
        LongTermDeposit = WholeFilter_Array(i)
    End If
    If LongTermDeposit < 0 Then
        LongTermDeposit = 0
    End If
    LongTerm_Array(UBound(LongTerm_Array)) = LongTermDeposit
    ReDim Preserve LongTerm_Array(UBound(LongTerm_Array) + 1)
Next i
'**********************************************************************
'CALCULATING THE DEPTH OF FILTER CAKE THAT DEVELOPS ON THE TOP OF THE FILTER MEDIA
'CALCULATING THE HYDRAULIC CONDUCTIVITIES (CAKE AND DEPTH K VALUES)
'**********************************************************************

    i = 0
    For i = 0 To Count

        CakeDepth = (LongTerm_Array(i) - SumDeposit_Array(i)) * TotalDepth / (1 - CakePorosity)

        If CakeDepth < 0 Then
            CakeDepth = 0
        End If

        CakeDepth_Array(UBound(CakeDepth_Array)) = CakeDepth
        ReDim Preserve CakeDepth_Array(UBound(CakeDepth_Array) + 1)

        CumulDepth = CakeDepth_Array(i) + TotalDepth
        CumulDepth_Array(UBound(CumulDepth_Array)) = CumulDepth
        ReDim Preserve CumulDepth_Array(UBound(CumulDepth_Array) + 1)

        Ke_Ko = 1 / (1 + ClogParameter * LongTerm_Array(i)) ^ 2
        Ke = Ke_Ko * Ko
        Ke_Array(UBound(Ke_Array)) = Ke
        ReDim Preserve Ke_Array(UBound(Ke_Array) + 1)

        Kd_Ko = 1 / (1 + ClogParameter * SumDeposit_Array(i)) ^ 2
        Kd = Kd_Ko * Ko
        Kd_Array(UBound(Kd_Array)) = Kd
        ReDim Preserve Kd_Array(UBound(Kd_Array) + 1)

        If CumulDepth = TotalDepth Then
            Kc = Ke
        Else
            Kc = CakeDepth_Array(i) / (CumulDepth_Array(i) / Ke_Array(i) - TotalDepth / Kd_Array(i))
        End If

        Kc_Array(UBound(Kc_Array)) = Kc
        ReDim Preserve Kc_Array(UBound(Kc_Array) + 1)
    Next

'**********************************************************************
'CALCULATING THE HYDRAULIC HEAD DEVELOPMENT AT THE TOP OF THE DEPTH FILTER, AND AT THE TOP OF THE CAKE LAYER
'**********************************************************************

'ASSUME THAT THE HYDRAULIC HEAD AT THE BASE OF THE FILTER IS ZERO (THIS IS DENOTED AS THE DATUM POINT)
'H2 REPRESENTS THE HYDRAULIC HEAD AT THE CAKE LAYER, WHEREAS, H1 REPRESENTS THE HYDRAULIC HEAD AT THE DEPTH FILTER

    For i = 0 To Count

        Porosity = CleanPorosity - LongTerm_Array(i)
        Porosity_Array(UBound(Porosity_Array)) = Porosity
Head1 = Velocity * TotalDepth / Kd_Array(i) 'applies darcy's relationship between H and K.
Head1_Array(UBound(Head1_Array)) = Head1

ReDim Preserve Porosity_Array(UBound(Porosity_Array) + 1)
ReDim Preserve Head1_Array(UBound(Head1_Array) + 1)

If TimeLag_Array(i) < 0 Then
    Head2 = 0
Else
    Head2 = (Velocity * CakeDepth_Array(i) / Kc_Array(i)) + Head1_Array(i)
End If

Head2_Array(UBound(Head2_Array)) = Head2
ReDim Preserve Head2_Array(UBound(Head2_Array) + 1)

Next i

'**********************************************************************
'CALCULATING FLOW REDUCTION, CHANGE IN SURFACE AREA, AND AVERAGE VELOCITY
'**********************************************************************

'IN THE EARLY PHASE OF SIMULATION (PRIOR TO CLOGGING), FLOW THROUGH THE POROUS MEDIA IS DEPENDENT ON Kd.
ONCE THE FILTER BEGINS TO CLOG, THE CLOGGING LAYER DEVELOPS WITH A LOWER K VALUE, AND THIS BECOMES THE CONTROLLING LAYER FOR FLOW REDUCTION.

i = 0

For i = 0 To Count
    If i = 0 Then
        Outflow_start = Worksheets("Analysis i_Model Output").Cells(13, 10).Value
        Area = Outflow_start / Velocity * Porosity_Array(i) * 100
        FilterFlow_Array(UBound(FilterFlow_Array)) = Outflow_start
        Area_Array(UBound(Area_Array)) = Area
        AvgVel = FilterFlow_Array(i) / Area_Array(i) * 100
        AvgVel_Array(UBound(AvgVel_Array)) = AvgVel
        ReDim Preserve FilterFlow_Array(UBound(FilterFlow_Array) + 1)
        ReDim Preserve Area_Array(UBound(Area_Array) + 1)
        ReDim Preserve AvgVel_Array(UBound(AvgVel_Array) + 1)
    Else
        If CakeDepth_Array(i) = 0 Then
            FilterFlow = FilterFlow_Array(i - 1) - Area_Array(i - 1) * VoidFraction * (Head1_Array(i) / 100 - Head1_Array(i - 1) / 100) / DeltaT
            FilterFlow_Array(UBound(FilterFlow_Array)) = FilterFlow
            Area = FilterFlow / Velocity * Porosity_Array(i) * 100
            Area_Array(UBound(Area_Array)) = Area
            AvgVel = FilterFlow_Array(i) / Area_Array(i) * 100
            AvgVel_Array(UBound(AvgVel_Array)) = AvgVel
            ReDim Preserve FilterFlow_Array(UBound(FilterFlow_Array) + 1)
            ReDim Preserve Area_Array(UBound(Area_Array) + 1)
            ReDim Preserve AvgVel_Array(UBound(AvgVel_Array) + 1)
        Else

        End If
    End If

Else

End If
ChangeCakePore = CakePorosity - LongTerm_Array(i)
FilterFlow = FilterFlow_Array(i - 1) - Area_Array(i - 1) * VoidFraction * (Head2_Array(i) / 100 - Head2_Array(i - 1) / 100) / DeltaT
FilterFlow_Array(UBound(FilterFlow_Array)) = FilterFlow
Area = FilterFlow / Velocity * ChangeCakePore * 100
Area_Array(UBound(Area_Array)) = Area
AvgVel = FilterFlow_Array(i) / Area_Array(i) * 100
AvgVel_Array(UBound(AvgVel_Array)) = AvgVel
ReDim Preserve FilterFlow_Array(UBound(FilterFlow_Array) + 1)
ReDim Preserve Area_Array(UBound(Area_Array) + 1)
ReDim Preserve AvgVel_Array(UBound(AvgVel_Array) + 1)
End If
End If
Next

'*****************************************************************************
'FIND CLOGGING TIME- THIS IS REPRESENTED BY THE TIME WHEN FLOW IS REDUCED TO
10% OF IT'S ORIGINAL VALUE
'*****************************************************************************
Iteration = Iteration + 1
Outflow_end = Outflow_start * 0.1 'DEFINING THE MAGNITUDE OF THE FLOW UPON
CLOGGING
For i = 0 To Count
    If FilterFlow_Array(i) > Outflow_end Then
        ClogTime = TCumul_Array(i)
    End If
Next
Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 3).Value = ClogTime
Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 2).Value = Iteration

'*****************************************************************************
'SIMULATING FLUCTUATIONS IN THE HYDRAULIC SURFACE IN THE INFILTRATION TRENCH
'*****************************************************************************
TotHydDepth1 = 0
TotHydDepth2 = 0
i = 0
k = 0
Area = Worksheets("Analysis i_Model Output").Cells(10, 15).Value
For i = 0 To Tstop
    Inflow = Intensity_array(i) * CatchmentArea * RunCoef / 1000
    If Inflow = 0 Then 'RESRESENTS AN INTEREVENT PERIOD WHEN THERE IS NO
    Rainfall (ONLY DRAINAGE THROUGH THE TRENCH)
        SumIntensity = 0
        HydDepth1 = (Inflow - FilterFlow) / Area + TotHydDepth1
        HydDepth2 = (Inflow - FilterFlow) / Area + TotHydDepth2
    End If
Next
TotHydDepth1 = HydDepth1
TotHydDepth2 = HydDepth2

If TotHydDepth1 < 0 Then
    TotHydDepth1 = 0
End If

If TotHydDepth2 < 0 Then
    TotHydDepth2 = 0
End If

Else    'Intensity > 0 (i.e rainfall event)

    k = k + 1
    SumIntensity = SumIntensity + Intensity_array(i)
    
    If SumIntensity < 25.4 Then
        HydDepth1 = (Inflow - FilterFlow_Array(k - 1)) / Area_Array(k - 1) + TotHydDepth1
        HydDepth2 = (Inflow - FilterFlow_Array(k - 1)) / Area_Array(k - 1) + TotHydDepth2
    Else 'VOLUME OF RAINFALL FOR ONE EVENT EXCEEDS 25.4 mm AND MUST BE REDIRECTED
        HydDepth1 = (0 - FilterFlow_Array(k - 1)) / Area_Array(k - 1) + TotHydDepth1
        HydDepth2 = (0 - FilterFlow_Array(k - 1)) / Area_Array(k - 1) + TotHydDepth2
    End If

    TotHydDepth1 = HydDepth1
    TotHydDepth2 = HydDepth2
    
    If TotHydDepth1 < 0 Then
        TotHydDepth1 = 0
    End If

    If TotHydDepth2 < 0 Then
        TotHydDepth2 = 0
    End If

    FilterFlow = FilterFlow_Array(k - 1)
    Area = Area_Array(k - 1)
    HydDepth1_Array(UBound(HydDepth1_Array)) = TotHydDepth1
    HydDepth2_Array(UBound(HydDepth2_Array)) = TotHydDepth2
    ReDim Preserve HydDepth1_Array(UBound(HydDepth1_Array) + 1)
    ReDim Preserve HydDepth2_Array(UBound(HydDepth2_Array) + 1)
    End If

    If TotHydDepth1 > TrenchDepth Then 'NO DRAIN IS USED TO FLUSH OUT THE SYSTEM WHEN CLOGGING OCCURS
        TotHydDepth1 = TrenchDepth
    End If

    If TotHydDepth2 > TrenchDepth Then 'A DRAIN IS USED TO PREVENT POOLING AT THE SURFACE OF THE TRENCH
        TotHydDepth2 = 0
    End If

Next

'**********************************************************************************************
'ESTIMATING TIME THAT RUNOFF IN THE INFILTRATION TRENCH FLOWS OVERFLOWS
'*****************************************************************************

    i = 0
    NumOverflow1 = 0
    NumOverflow2 = 0

    For i = 0 To k  'CALCULATING TIME THAT TRENCH WITH DRAIN OVERFLOWS FOR THE
                    FIRST TIME

        If HydDepth1_Array(i) > TrenchDepth + OverflowBerm Then
            OverflowTime1 = TCumul_Array(i)
            OverflowTime1_Array(UBound(OverflowTime1_Array)) = OverflowTime1
            NumOverflow1 = NumOverflow1 + 1
            ReDim Preserve OverflowTime1_Array(UBound(OverflowTime1_Array) + 1)
        End If

        If HydDepth2_Array(i) > TrenchDepth + OverflowBerm Then
            OverflowTime2 = TCumul_Array(i)
            OverflowTime2_Array(UBound(OverflowTime2_Array)) = OverflowTime2
            NumOverflow2 = NumOverflow2 + 1
            ReDim Preserve OverflowTime2_Array(UBound(OverflowTime2_Array) + 1)
        End If

    Next i

    Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 4).Value = OverflowTime1_Array(0)
    Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 6).Value = NumOverflow1
    Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 5).Value = OverflowTime2_Array(0)
    Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 7).Value = NumOverflow2

'*****************************************************************************

'CALCULATING THE ANNUAL MASS LOADING OF DISSOLVED METALS THROUGH THE BOTTOM OF
THE INFILTRATION TRENCH
'DISSOLVED METALS ARE ASSUMED TO BE A CONSERVATIVE TRACER THROUGH THE BASE OF
THE TRENCH DUE TO THE FAIRLY INERT FILTER MATERIAL
'THE CONSERVATIVE TRACER ASSUMPTION DOES NOT APPLY ONCE THE DISSOLVED METAL
REACHES THE GW MEDIA DUE TO HIGHER ORGANIC CONTENT
'*****************************************************************************

'Note that the unit of time for the following count function is 1 hour. Therefore, the mass
loading represents the total
'mass loading for 1 hour. Also, the Copper and Zinc arrays are the
respective concentration of each metal during rainfall events.

    TotalCopper = 0
    TotalZinc = 0

    For i = 0 To Count  'Recall that Count is the cumulative time (in hours)
                        during which rain events occur.

        Mcopper = Copper_Array(i) * FilterFlow_Array(i) / 1000  'loading for 1
                                                            hour (units kg)
        Mzinc = Zinc_Array(i) * FilterFlow_Array(i) / 1000
        TotalCopper = TotalCopper + Mcopper  'Calculation of total loading per
                                              year of each heavy metal.
        TotalZinc = TotalZinc + Mzinc
        CumulativeCopper_Array(UBound(CumulativeCopper_Array)) = TotalCopper
        CumulativeZinc_Array(UBound(CumulativeZinc_Array)) = TotalZinc
        ReDim Preserve CumulativeZinc_Array(UBound(CumulativeZinc_Array) + 1)
ReDim Preserve CumulativeCopper.Array(UBound(CumulativeCopper.Array) + 1)

Next i

'********************************************************************************
'DETERMINING THE TIME THAT MASS LOADING OF COPPER AND ZINC EXCEED ASSIGNED CRITERIA
'********************************************************************************

'MAXIMUM ZINC LOADING = 0.75 KG
'MAXIMUM COPPER LOADING = 0.20 KG

For i = 0 To Count
    If CumulativeCopper.Array(i) < 0.2 Then
        TcopperPass = TCumul.Array(i)
    End If
    If CumulativeZinc.Array(i) < 0.75 Then
        TzincPass = TCumul.Array(i)
    End If

Next

Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 8).Value = TcopperPass
Worksheets("Analysis ii_Table 2").Cells(Iteration + 5, 9).Value = TzincPass

PercentComplete = t / Worksheets("Analysis i_Table 1").Cells(1, 14).Value
Application.ScreenUpdating = True

Next

Unload(UserForm1) 'close progress bar

'********************************************************************************
'PRINTING VALUES OF FINAL SIMULATION IN SPREADSHEET
'********************************************************************************

i = 0

For i = 0 To Count

    Worksheets("Analysis i_Table 1").Cells(i + 12, 1).Value = TCumul.Array(i)
    Worksheets("Analysis i_Table 1").Cells(i + 12, 2).Value = TCumul.Array(i) * 8760
    Worksheets("Analysis i_Table 1").Cells(i + 12, 3).Value = EMC.Array(i)
    Worksheets("Analysis i_Table 1").Cells(i + 12, 4).Value = i
    Worksheets("Analysis i_Table 1").Cells(i + 12, 5).Value = TimeLag.Array(i)
    Worksheets("Analysis i_Table 3").Cells(i + 12, TotIncr + 3).Value = SumDeposit.Array(i)
    Worksheets("Analysis i_Table 3").Cells(i + 12, TotIncr + 2).Value = CumulativeMassDeposit.Array(i)
    Worksheets("Analysis i_Table 3").Cells(i + 12, TotIncr + 4).Value = WholeFilter.Array(i)
    Worksheets("Analysis i_Table 4").Cells(i + 12, 6).Value = LongTerm.Array(i)
    Worksheets("Analysis i_Table 4").Cells(i + 12, 1).Value =
    CakeDepth.Array(i)
    Worksheets("Analysis i_Table 4").Cells(i + 12, 2).Value =
    CumulDepth.Array(i)

Worksheets("Analysis i_Table 4").Cells(i + 12, 3).Value = Ke.Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 4).Value = Kd_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 5).Value = Kc_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 11).Value = Porosity_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 7).Value = Head1_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 8).Value = Head2_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 12).Value = Area_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 9).Value = FilterFlow_Array(i)
Worksheets("Analysis i_Table 4").Cells(i + 12, 10).Value = AvgVel_Array(i)

Next

i = 0

For i = 0 To k
    Worksheets("Analysis ii_Table 1").Cells(i + 8, 1).Value = HydDepth1_Array(i)
    Worksheets("Analysis ii_Table 1").Cells(i + 8, 2).Value = HydDepth2_Array(i)
Next i

'OUTPUTING CONCENTRATION PROFILES AND EFFLUENT CONCENTRATIONS FOR SIMULATIONS
i = 0
For i = 0 To Count
    j = 0

    For j = 0 To TotIncr
        Worksheets("Analysis i_Table 1").Cells(i + 12, j + 6).Value = CFraction(i, j)
        Worksheets("Analysis i_Table 2").Cells(i + 11, j + 1).Value = CFraction(i, j) * EMC_Array(i) / 1000
    Next j

Next i

'OUTPUTING SPECIFIC DEPOSITION PROFILES
i = 0
For i = 0 To Count
    For j = 0 To TotIncr - 1
        Worksheets("Analysis i_Table 3").Cells(i + 12, j + 2) = FSpecific(i, j)
    Next j

Next i

i = 0

'OUTPUTTING MASS LOADING PER YEAR (HEAVY METALS)
For i = 0 To Count
    Worksheets("Analysis ii_Table 2").Cells(i + 8, 19).Value = CumulativeCopper_Array(i)
    Worksheets("Analysis ii_Table 2").Cells(i + 8, 20).Value = CumulativeZinc_Array(i)
    Worksheets("Analysis ii_Table 2").Cells(i + 8, 18).Value = TCumul_Array(i)
Next i
'PRINTING LABELS
Depth = 0
For j = 0 To TotIncr
    Worksheets("Analysis i_Table 1").Cells(10, 6 + j).Value = Depth
    Worksheets("Analysis i_Table 2").Cells(8, 1 + j).Value = Depth
    Worksheets("Analysis i_Table 3").Cells(9, 1 + j).Value = Depth
    Depth = Depth + Delta
Next
Worksheets("Analysis i_Table 3").Cells(9, TotIncr + 2).Value = "Specific Deposit"
Worksheets("Analysis i_Table 3").Cells(9, TotIncr + 3).Value = "Non-Clog Cumulative"
Worksheets("Analysis i_Table 3").Cells(9, TotIncr + 4).Value = "Clog Cumulative"
End Sub

Private Function MRG32k3a(ByVal seed As Double) As Double
    'Random number generator by P. L'Ecuyer, 1999
    'This function combines 2 multiple recursive number generators of order 3 to return a pseudo-random number with a uniform probability distribution in the open interval (0.0, 1.0).
    'The period length is = 2^191 = 3.1 x 10^57
    'This routine is approx. ~7 times slower than the VB Rnd function
    Static s10 As Double, s11 As Double, s12 As Double
    Static s20 As Double, s21 As Double, s22 As Double
    Dim k As Long
    Dim p1 As Double, p2 As Double
    Const norm As Double = 0.000000000232830654929573
    Const m1 As Double = 4294967087.0#
    Const m2 As Double = 4294944443.0#
    Const a12 As Double = 1403580.0#
    Const a13n As Double = 810728.0#
    Const a21 As Double = 1403580.0#
    Const a23n As Double = 1370589.0#
    If s10 = 0.0# Then 'Initialize first time
        s10 = seed
        s11 = seed
        s12 = seed
        s20 = seed
        s21 = seed
        s22 = seed
    End If
    ' Component 1
    p1 = a12 * s11 - a13n * s10
    k = Int(p1 / m1)
    p1 = p1 - k * m1
    If (p1 < 0.0#) Then p1 = p1 + m1
s10 = s11
s11 = s12
s12 = p1

' Component 2
p2 = a21 * s22 - a23n * s20
k = Int(p2 / m2)
p2 = p2 - k * m2
If (p2 < 0.0#) Then p2 = p2 + m2
s20 = s21
s21 = s22
s22 = p2

' Combination
If (p1 <= p2) Then
    MRG32k3a = (p1 - p2 + m1) * norm
Else
    MRG32k3a = (p1 - p2) * norm
End If

End Function
APPENDIX B
SCREEN CAPTURES OF EXAMPLE SIMULATIONS
**FA 1** Screen capture of concentration profiles at changing depths and time increments

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<th>Years</th>
<th>Hours</th>
<th>Input</th>
<th>Time</th>
<th>Conc. Time, C</th>
<th>C/O</th>
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</table>
Screen capture of long-term specific deposition profile (non-clogging and clogging)
FA 3 Screen capture of dynamic changes in performance criteria (headloss, hydraulic conductivity, and flow)

### TABLE 3: Hydraulic Head Development and Decrease in Hydraulic Conductivity

<table>
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<tr>
<th>Lc</th>
<th>Lb</th>
<th>ke</th>
<th>kd</th>
<th>Ke</th>
<th>ln</th>
<th>Hl</th>
<th>Hc</th>
<th>Q(t)</th>
<th>v(t)</th>
<th>dH(t)/dt</th>
<th>Ap(react)</th>
</tr>
</thead>
<tbody>
<tr>
<td>cm</td>
<td>cm</td>
<td>cm/hr</td>
<td>cm/hr</td>
<td>cm/hr</td>
<td></td>
<td>cm</td>
<td>cm</td>
<td>m^3/hr</td>
<td>m/s</td>
<td>m/hr</td>
<td>ster/day</td>
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<td>20.0</td>
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<td>90.00</td>
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<td>20.0</td>
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<td>90.00</td>
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<td>2.000</td>
<td>3.000</td>
<td>5.000E+01</td>
<td>90.00</td>
</tr>
</tbody>
</table>

### Notes:
- The table shows the development and decrease in hydraulic conductivity over time, with changes in headloss and flow rates.
- Parameters such as headloss, hydraulic conductivity, and flow are key indicators of performance in this context.
- The data reflects dynamic changes that are crucial for understanding the efficiency and stability of the system under investigation.