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Effect of land use and low impact development measures on urban flood hazard: a case study in the Black Creek watershed

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Abstract

The number of flooding events in Canadian urban environments continues to increase, causing major environmental, economic and social consequences. This research uses the Black Creek watershed, located in southern Ontario, to investigate the impacts of urbanization on flood hazard and evaluate the effectiveness of various low impact development practices at reducing this hazard. A land use analysis indicates extensive urban growth between the periods 1949 and 2015, with extremely high imperviousness percentages existing today in the majority of the watershed. Historical hydrological simulations in PCSWMM show a significant increase in peak flows since 1949, but have now stabilized due to the limited land available for further development. Stormwater management ponds in the northern region of the watershed have helped control runoff from densely developed areas. Minimal stormwater management exists in the southern region, with low potential for implementation of large stormwater management features. Low impact development practices such as bioretention cells, infiltration trenches, permeable pavement, rain barrels, and vegetative swales were simulated in various scenarios to investigate their effects at reducing flood hazard. Results demonstrate that low impact development measures can effectively reduce peak runoff reduction rates by as much as 76% in smaller subcatchments for a 2-year storm event. This thesis provides insight into the capabilities of low impact development measures to improve flood hazard management and decrease flood hazard in urban environments.

Keywords

Urbanization, flood hazard management, low impact development, land use analysis, hydrologic modeling, PCSWMM.

Co-Authorship Statement

The work of this thesis is a collaborative effort by the present author, Dr. Andrew D. Binns, and Dr. Slobodan P. Simonovic. Chapter 2 contains sections that have been accepted for publication as a Water Resources Research Report conducted by the Facility for Intelligent Decision Support, with the present author as primary author and Dr. Simonovic and Dr. Binns providing significant feedback on the manuscript.

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List of Abbreviations

BC	Bioretention Cell
CL	Commercial Lot
CVC	Credit Valley Conservation
HSPF	Hydrologic Simulation Program-Fortran
IT	Infiltration Trench
IBC	Insurance Bureau of Canada
LID	Low Impact Development
PCSWMM	Personal Computer Storm Water Management Model
RB	Rain Barrel
RD	Residential Driveway
RG	Rain Garden
SWMM	Storm Water Management Model
TRCA	Toronto and Region Conservation Authority
VS	Vegetative Swale

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Chapter 1

1 Introduction

1.1 Motivation

In the past century, flood events have been the most frequently occurring and costliest natural disaster in Canada (Sandink et al. 2010; Public Safety Canada 2015). Flooding continues in Canadian urban environments, specifically, resulting in significant economic damages to homeowners, insurers, and municipalities. From 2009-2014, annual insured losses due to water damage have been close to $1 billion, with the exception of the year 2013 which experienced approximately $3.5 billion in insured losses (IBC 2016). In June of 2013, snowmelt and intense rainfall caused severe flooding in Alberta, creating significant damage to infrastructure and impacting the lives of thousands of people. Damages from the storm were estimated to exceed $6 billion (Environment and Climate Change Canada 2014a), with insured losses of approximately $1.8 billion (IBC 2016). In July of 2013, Toronto experienced a high-intensity short-duration rainfall event that exceeded the capacity of storm sewers and caused serious flooding. Several roads and underpasses were under water creating significant disruptions to transit and society (Environment and Climate Change Canada 2014b). Insured losses were estimated at approximately $1 billion (IBC 2016), making it the most expensive natural disaster ever experienced in Ontario (Environment and Climate Change Canada 2014b). More recently in June and July of 2016, intense rainfall in Vancouver and Toronto caused sewers and local creeks to overflow, resulting in more flood damages (CBC 2016a; CBC 2016c).

Multiple types of floods occur in Canadian urban environments and can be classified as coastal, fluvial, or pluvial flooding (Jha et al. 2012; IBC 2015). Coastal flooding can occur from storm surges, hurricanes, and tsunamis created by high winds and tidal waves from oceans and lakes. Fluvial, or riverine flooding, occurs when large river systems exceed their capacity and begin to overflow their banks. This can be caused by heavy rainfall, ice jams, snowmelt runoff, or failure of dams. Pluvial, or urban flooding, occurs when water ponds on the surface due to heavy rainfall or local creeks overflow. Ponding can be caused by urban drainage systems reaching their maximum capacity or groundwater levels rising due to maximum saturation (Sandink et al. 2010; Buttle et al. 2016). Canadian cities are specifically susceptible to "flash floods" as a result of high levels of imperviousness, capacity of drainage systems, decreased vegetation, and localized intense rainfall. Flash floods occur with little to no warning and prove to be the most devastating due to their rapid response and unpredictability. Urban floods caused by extreme rainfall have the capability of backing up sewers and creating significant damages to basements of Canadian homeowners, which can sometimes be uninsurable (Sandink 2009; Sandink 2015). In 2014, approximately 82% of Canada's total population resided in urban areas and it is projected that this number will rise to approximately 88% by the year 2050 (United Nations 2014). This emphasizes the paramount importance of flood hazard management in Canadian urban environments in order to protect society from the adverse effects of flooding, now and in the future.

Several types of flood control measures are integrated into urban environments for reducing urban flooding hazards. These are generally broken down into structural and non-structural measures. Non-structural measures include insurance, land use regulations, forecasting, education, source controls, recycling, and proper maintenance. Structural measures are engineered or constructed systems and consist of river channelization, dikes, and dams for controlling riverine flooding. Basement flooding is controlled through backwater valves, sump pumps, and stormwater management (SWM) measures. SWM approaches used to reduce urban flood hazard consist of various measures that are incorporated into the environment to aid in the reduction of peak flows and stormwater volume while also improving water quality. These include wet ponds, dry ponds, and constructed wetlands. Ponds allow for large volumes of water to be stored and gradually released after storm events, minimizing the amount of stormwater runoff being sent to drainage systems. Wet ponds contain a permanent pool of water whereas dry ponds fill up with stormwater and then are capable of fully draining. Constructed wetlands promote natural infiltration of rainwater and are able to sustain a diverse ecosystem. However, many Canadian cities have minimal opportunities for implementation of large-scale SWM measures due to a lack of available land required for these measures. Thus, there is a need for the design and installation of smaller-scaled SWM measures that can manage runoff in the urban environment.

In response to the 2016 Vancouver flooding, University of British Columbia professor Hans Schreier stated (CBC 2016b, para. 4):

> And what we're doing in the urban environment, we make everything impervious: that means the water can no longer infiltrate and as a result it runs over the surface into creeks and then the creeks are no longer able to handle the extra water.

Schreier emphasized the push towards more innovative stormwater management moving forward such as permeable pavement, rain gardens, and swales to promote more infiltration of rainwater (CBC 2016b). These are better known today as low impact development (LID) measures. LID measures are one promising alternative to traditional stormwater practices as a result of their small-scale and cost-effective approach. They provide an excellent solution for cities due to their retrofit capabilities and their ability to be integrated into developed environments. LID measures are capable of receiving and infiltrating stormwater runoff that would otherwise increase urban flooding hazards. However, due to their size, they are more commonly seen as lot-level measures suitable for treating smaller storm events. Their flood control capabilities in a large-scaled urban environment are not well understood (Ahiablame and Shakya 2016). This highlights the importance of evaluating the effectiveness of LID measures at reducing flood hazard on a large-scale and in highly developed cities. It should be mentioned that this thesis focuses solely on the hazard side of risk and not the consequences associated with floods (see Eq. 1). Flood hazard, as it relates to this thesis, is defined as any flood event that could cause damage to property, humans, or the environment.

$$Risk = Hazard\ x\ Consequence \tag{1}$$

1.2 Goal and objectives of the thesis

The goal of this thesis is to investigate the effects of urbanization on flood hazard and quantify the effectiveness of LID stormwater management measures to reduce this hazard. This thesis will use the Black Creek watershed, located in southern Ontario, as a case study. To accomplish this goal, the following two objectives will be satisfied:

(1) investigate the effect of land use change and modifications to natural waterways on flood hazard; and

(2) evaluate the effectiveness of LID measures to mitigate flood hazard.

A cost-benefit analysis will compare the cost of implementing LID measures and their expected impact on reducing flood hazard. Through these two objectives, recommendations will be provided for potential retrofit of LID features in urbanized Canadian environments.

1.3 Organization of the thesis

This thesis is organized in the classical monograph format. Following the introduction in Chapter 1, six chapters are included and are organized as follows:

Chapter 2 provides a comprehensive literature review characterizing Canadian cities, reviewing tools to assess flooding in urban environments, and describing common urban stormwater models being used today. A discussion on the gaps in literature is also provided.

Chapter 3 provides a complete description of the study area.

Chapter 4 describes information about the data and model used. The proposed methodology for the research is also outlined.

Chapter 5 focuses on objective (1) of this thesis. The results from a land use analysis conducted on the Black Creek watershed between the periods 1949 and 2015 is presented. Results from hypothetical historical simulations are discussed and combined with the land use analysis results.

Chapter 6 focuses on objective (2) of this thesis. The results from current time hydrologic simulations of various LID scenarios based on different locations and different investments is presented. A cost-benefit analysis between the effectiveness of various LID scenarios and the total investment required is also discussed.

Chapter 7 provides a discussion on the results from Chapters 5 through 6 and summarizes the main contributions and conclusions of the thesis. Recommendations and opportunities for future research, as well as a discussion on the applicability of this research to other situations is also presented.

Chapter 2

2 Literature review

The chapter provides on overview of the characteristics of Canadian cities as well as the tools available for flood hazard management. The impact of climate change on Canadian urban infrastructure and common stormwater management measures for mitigating flood hazard will be discussed. Computational tools used for flood hazard management and gaps in literature will also be outlined.

2.1 Characterization of Canadian cities

This section provides a discussion on the characterization of Canadian cities. Specifically, their development over time, common types of floods, the effects of climate change on infrastructure, flood mitigation measures, and the concept of LID measures.

2.1.1 Urbanization of Canadian cities

Canadian cities are complex environments that have been evolving for many decades. The rapid advancement in technology, continuously growing populations, and the demands of society are the main reasons Canadian cities have transformed from fertile agricultural land into heavily developed and impervious land. These cities commonly originated near bodies of water as this provided early sources of transportation, water supply, and power. The structure can be commonly seen today as consisting of an older "inner-city" (or downtown core) surrounded by newer suburbs and rural land, with natural rivers and streams flowing through (Bunting and Filion 2006). Urban sprawl is largely responsible for this structure due to residents becoming increasingly attracted to the low-density suburban lifestyle over time (Stone and Gibbins 2002). This can be accredited to the post World War II boom of the automotive industry and the continuous investments in expressways which has created very efficient commute times (Bunting and Filion 2006).

The suburbs are continuously expanding outwards into fertile agricultural land with the construction of subdivisions and shopping centres (Gurin 2003). Separate sewer systems which consist of sanitary sewers and storm sewers are commonly integrated into all new development. Sanitary sewers convey all wastewater collected from residential, commercial, and industrial buildings to water treatment facilities. Storm sewers, however, convey only excess rainfall from parking lots, roads, roofs, and sidewalks directly to rivers and streams. SWM measures are incorporated into the environment, such as wet ponds in subdivisions, permeable pavement in parking lots and driveways, green roofs on top of structurally capable buildings, and vegetative swales along roads. The combination of these measures help to reduce the volume of water entering storm sewers, reducing the risk for sewer overflow and flooding in local channels.

In comparison, the downtown core is a condensed region consisting of a greater population density, lower income, high-rise buildings, smaller homes, and a lack of pervious land. Aging infrastructure is also very common as development dates back to the 19th and 20th centuries (Bunting and Filion 2006). Combined sewers which convey both stormwater and wastewater to treatment facilities are still in operational use which creates problems for many Canadian cities. These types of sewers increase the risk for combined sewer overflows (CSOs) and bypasses to occur as the sewer system is inadequate to handle today's more frequent and intense precipitation events in combination with highly impervious surfaces. Waterways running through these areas have been engineered or channelized in some form in the past, with minimal natural stormwater management features, in order to improve hydraulic conveyance and reduce bank erosion. These techniques, however, have proven to be unable to handle today's rapidly changing environmental and hydrological conditions.

The development of Canadian cities has had a significant impact on all processes within the hydrological cycle. Infiltration rates are substantially reduced with the implementation of more impervious surfaces such as parking lots and buildings. This directly reduces groundwater recharge, and depending on the watershed characteristics, has the potential to greatly impact the base flow of rivers and streams. Evapotranspiration and interception also decrease as vegetation is cleared for urban development (Karamouz et al. 2010). Runoff is substantially increased due to the presence of impervious surfaces and drainage systems rapidly delivering water to local creeks and rivers. Altogether, these effects have caused components of the hydrologic cycle to change in relative magnitude, considerably increasing flood hazard. However, numerous SWM practices exist today that can adapt to these dynamic, ever-changing conditions by mimicking natural hydrologic processes.

While these hydrologic processes operate on a global scale, other local processes exist in cities creating an urban hydrologic cycle. Precipitation, along with extraction of water from lakes and groundwater can introduce water into the system. Drainage systems transport water to and from homes, industries, and businesses, as well as to treatment facilities, and finally to local rivers and streams.

2.1.2 Types of flooding in Canadian cities

The most common types of flood events that occur in Canadian cities are coastal, riverine (fluvial), and urban (pluvial) flooding. Coastal flooding is common in some Canadian cities due to Canada’s proximity to many bodies of water. Coastal flooding can occur from storm surges, hurricanes, and tsunamis created by high winds and tidal waves from bodies of water. This type of flooding is less common but still poses a serious threat to Canadian citizens living near the coasts of Canada. Riverine flooding affects many Canadian cities such as Winnipeg, Calgary, and Vancouver. Large river systems in cities such as these are prone to large-scale flooding due to ice jams, heavy snowmelt, and extreme precipitation events. Urban flooding, however, has become a severe issue to homeowners in almost every city in Canada. The effects of urban development, watercourse modifications, and climate change has severely increased flood hazard in cities. The abundance of impervious surfaces has created excess amounts of stormwater runoff being directed to drainage systems. These systems are at a high risk of reaching their full capacity due to aging infrastructure and current use of combined sewer systems in older parts of cities. This may lead to street flooding and sewer backups into basements which is a growing issue in Canada. Local rivers and creeks are also unable to handle large amounts of stormwater runoff being sent from drainage systems. The lack of surrounding natural environments prevents infiltration of water, causing waterways to overflow and flood as well. This thesis focuses on urban flooding and evaluates the abilities of LID measures in being able to infiltrate runoff and reduce this hazard.

2.1.3 Effect of climate change on Canadian urban infrastructure

Urban growth in Canadian cities has put a tremendous amount of pressure on the environment with mass amounts of automobiles and industrial plants emitting harmful greenhouse gases into the atmosphere. These gases trap heat in the atmosphere and contribute to climate change (Statistics Canada 2008). Climate change is creating changes in precipitation patterns throughout the world (Dore 2005; Trenberth 2011; Acharya et al. 2013; Moore et al. 2015; Villafuerte II et al. 2015), including Canada (Ashmore and Church 2001; Statistics Canada 2008). More frequent and intense precipitation events are commonly experienced along with warmer temperatures across Canada. Sea levels are rising due to rapid melting of glaciers, creating much higher risk of storm surge flooding for coastal cities. Extended periods of wet weather, spring snowmelt, and ice-jams are also increasing flood hazard (Ouellet et al. 2012; Abraham 2015).

This changing climate is impacting Canada's water and transportation infrastructure through higher maintenance and operation costs. Extreme precipitation introduces more contaminants from runoff whereas higher temperatures negatively impact the quality of water, increasing the cost of water treatment. The resulting increase in flows challenge municipal water infrastructure by increasing the risk of combined sewer overflows and placing stress on the operational abilities of pumping stations (Andrey et al. 2014). Other examples of the effect of climate change on infrastructure include: failures in permafrost highways in northern communities due to the permafrost thawing from warmer ground temperatures, increased freeze-thaw cycles in southern Ontario which greatly reduce the service life of roadways (Infrastructure Canada 2006), and failures in culverts such as that due to the intense rainfall event in Toronto on August 19, 2005 which resulted in millions of dollars in damage (Environment and Climate Change Canada 2013).

2.1.4 Flood mitigation measures in Canadian cities

Traditional SWM practices consist of structural and non-structural measures which can be broken down into source, lot-level, conveyance, and end-of-pipe controls. Structural measures are engineered systems that are designed to mitigate the impacts of stormwater whereas non-structural measures are practices and approaches that are implemented to reduce the occurrence of stormwater runoff while also controlling pollution at the source. Non-structural measures can be very efficient and cost-effective as they can reduce the need for expensive structural measures at a future time. Examples of non-structural measures include insurance, land use regulations, forecasting, and minimizing soil compaction. These measures depend on public awareness and municipality enforcement.

Structural measures, such as lot-level and conveyance controls can include storage and infiltration techniques (Municipal Program Development Branch 1999; Strassler et al. 1999; MOE 2003), channelization, and placement of dikes. Together, these controls help reduce stormwater quantity and improve stormwater quality by removing contaminants before they can be transported downstream. Examples of these controls include: rooftop or parking lot storage, reduced lot grading, infiltration trenches, and pervious pipe systems. They are generally applied in small drainage areas and away from industrial activity to reduce the risk of failure or clogging. End-of-pipe controls enhance stormwater quality prior to discharge into rivers or streams. These controls are particularly useful for preventing flooding and possibly erosion downstream by controlling the quantity of stormwater and releasing it at predetermined rates. Examples include wet ponds, dry ponds, and constructed wetlands. Wet ponds are commonly installed in new residential areas as they not only control the large amount of stormwater produced, but they provide an aesthetic appearance with vegetation and wildlife habitat. They can also be implemented in commercial or industrial areas where nutrient levels may be higher (Municipal Program Development Branch 1999; Strassler et al. 1999; MOE 2003). Depending on the characteristics of the region, constructed wetlands have the potential to mitigate floods, increase water quality, and sustain a diverse ecosystem (Malaviya and Singh 2012). Natural wetlands have been decreasing in Canada, however, recent research has demonstrated the ability of wetlands to reduce peak flows (Simonovic and Juliano 2001; Qaiser et al. 2012). The success of structural SWM measures depends on numerous factors such as drainage area, soil type, topography, and water table depth (Stephens et al. 2002; MOE 2003).

Channelization of natural waterways is a very common process implemented in Canadian cities in order to reduce riverine flooding hazard. This includes enlarging channels to allow for a higher capacity of flow (Surian 2007), lining banks with concrete to promote efficient drainage (Charlton 2008), and stream alignment to reduce the flow length of channels. However, these artificial approaches have considerable adverse effects on river morphology, hydrology, ecology, and infrastructure due to the loss of natural functions and reduced ability to adapt to rapidly changing conditions (Surian 2007). This is especially important in cities that have undergone intensive urban development and are diverting a substantial amount of runoff to local channels that were channelized in the past for efficient drainage purposes. Today, with the increasing amount of runoff and changing hydrological patterns due to climate change, these channels are now susceptible to flood hazard. Dikes are implemented along water bodies to contain riverine flooding and prevent damage to homes and infrastructure.

The flood mitigation measures discussed in this section are implemented on a large-scale to prevent flooding from severe rainfall events. However, most Canadian cities now lack available land for large-scale implementation of SWM measures and are experiencing more frequent flooding events. Cities are slowly converting to separate sewer systems and restoring engineered rivers to more natural conditions, but the associated costs of doing so are very high. Other measures, such as small-scale LID measures, need to be evaluated in hopes of reducing flood hazard in Canadian cities.

2.1.5 Low impact development

A promising alternative to traditional large-scale SWM practices mentioned in Section 2.1.4 are LID measures. These consist of small-scale structural practices that utilize natural resources and aim to mimic natural hydrologic conditions. Their ability for retrofit in a wide variety of situations makes them an appealing SWM practice. LID measures can reduce stormwater quantity and improve quality through processes such as infiltration, evapotranspiration, and detention. In addition, they also help to reduce impervious surfaces and increase aesthetics in urban settings. LID measures are best applied in combination with traditional structural and non-structural SWM measures to achieve the best results. Common LID approaches incorporated in residential, commercial, industrial, and institutional areas include bioretention, infiltration trenches, permeable pavement, rainwater harvesting, green roofs, downspout disconnection, and vegetative swales.

Bioretention units are excavated depressions that contain layers of vegetation, mulch, an engineered soil mix, and a storage reservoir made of stones (Uda et al. 2013). Bioretention cells, rain gardens, stormwater planters, extended tree pits, and curb extensions are all variations of bioretention (TRCA and CVC 2010). These can be applied in many settings such as residential neighborhoods, parking lots, roadsides, and next to buildings. They can infiltrate runoff as well as provide aesthetic benefits in urban settings. Infiltration trenches are simply rectangular depressions that are completely filled with clean granular stone (Uda et al. 2013). They typically only handle runoff from roofs and walkways but can also be designed to accept overflow from rainwater harvesting systems (TRCA and CVC 2010). Permeable pavements replace traditional impervious pavement and allow water to infiltrate through the surface into a stone reservoir. Porous concrete, porous asphalt, concrete or plastic grid pavers, and permeable interlocking concrete pavers are all different variations of permeable pavement (Uda et al. 2013). These are typically incorporated into areas where traffic volume is low such as parking lots, walkways, residential driveways, and even low traffic roads (TRCA and CVC 2010). Rainwater harvesting can be a large-scale or small-scale measure for retaining rainwater. Rainwater falling onto a roof will be collected and typically stored for future use, or can be released to pervious surfaces during dry periods (Stephens et al. 2002). Rain barrels are commonly used for residential homes and range in size from 190 to 400 L. Large cisterns made of concrete or plastic can be built above and below ground, or even inside buildings. These are typically built in commercial and industrial lots since they are capable of handling anywhere from 750 to 40,000 L of water (TRCA and CVC 2010). Green roofs consist of vegetation and a growing medium and are commonly installed on large commercial and industrial buildings as they have greater load bearing capacities. A simple downspout disconnection from homes allows precipitation to be directed to pervious areas for infiltration instead of being received by storm sewer drains. Vegetative swales are vegetated ditches that are more known for retaining and conveying runoff instead of promoting infiltration (TRCA and CVC 2010). They are typically seen adjacent to roadways and in parking lots (Uda et al. 2013).

Site conditions are a critical component to LID implementation to ensure safety and proper function. In order to effectively implement LID measures, conditions such as soil permeability, soil type, surface slope, water table depth, and surrounding land use must be considered (U.S. Environmental Protection Agency 2000; TRCA and CVC 2010). For regions with a soil permeability below 15 mm/hr, it is recommended that LID measures such as bioretention, permeable pavement, and infiltration trenches contain an underdrain (TRCA and CVC 2010). Placement of infiltration LID measures should be avoided in areas that receive high levels of contaminants such as industrial sites. This reduces the likelihood of clogging and extra maintenance costs. LID measures that promote infiltration should also be avoided in areas with a high water table to ensure the quality of groundwater is not negatively affected (TRCA and CVC 2010).

A considerable amount of research has been conducted to evaluate the capabilities of LID measures (Dietz 2007; Ahiablame et al. 2012; Zhang and Guo 2015) and simulate the performance of LID measures with modeling software (Elliot and Trowsdale 2007; Ahiablame et al. 2012). Dietz (2007) provided an extensive review on bioretention, porous pavements and green roofs, summarizing the effectiveness of these measures and the associated concerns. Elliot and Trowsdale (2007) evaluated ten existing stormwater models for modeling LID measures based on several criteria such as potential uses, spatial and temporal resolution, runoff generating and routing methods, types of contaminants that can be modeled, and types of LID measures that can be modeled. Their review demonstrates a wide variety of stormwater models exist for modeling LID measures with specific purposes and limitations. Ahiablame et al. (2012) provided a literature review on the effectiveness of LID practices and their representation in hydrologic and water quality models.

Joksimovic and Alam (2014) investigated the cost efficiency of implementing a wide variety of LID measures in a proposed land development in London, Ontario. Green roofs and rainwater harvesting were found to be the most and least expensive features in reducing volume of runoff, respectively. Results indicated that infiltration trenches alone and in combination with green roofs were the most cost efficient scenarios for runoff reduction. Zhang and Guo (2015) evaluated the performance of permeable pavement systems in PCSWMM and have provided an alternative method of representing these features in the model. Ahiablame and Shakya (2016) assessed the flood reduction capabilities of porous pavement, rain barrels, and rain gardens in a large-scaled urban watershed. Results using the model PCSWMM indicated a 3-40% reduction in average annual runoff for individual measures, but 16-47% when applying a combination of these measures. Simulations with permeable pavement were demonstrated to have larger effects in comparison to simulations with rain gardens or rain barrels. Chui et al. (2016) investigated optimal designs of green roofs, porous pavement, and bioretention on a large-scale by assessing their hydrological performance and cost-effectiveness. A sensitivity analysis on design parameters such as initial saturation, hydraulic conductivity, and berm height was performed. Results for green roofs indicated lower reductions in the peak runoff with higher initial saturation and hydraulic conductivity rates. Berm height for green roofs was shown to have no effect. Initial saturation status had a minimal effect with bioretention. However, higher hydraulic conductivity resulted in greater peak runoff reduction. Berm height for bioretention had a positive effect as well. Hydraulic conductivity was the main parameter of interest for porous pavement with results showing higher peak runoff reductions for higher values of hydraulic conductivity. Hydrological simulations indicated the most cost-effective measure for peak flow reduction was porous pavement, followed by bioretention and green roofs.

The field of LID technology has been well studied and is becoming an increasingly popular alternative for reducing stormwater runoff associated with heavy urban development. Literature has demonstrated a strong potential for LID measures to reduce stormwater runoff, the ability of LID measures to be incorporated into different types of environments, the wide range of models in existence for simulating these practices, and their associated limitations.

2.2 Tools to assess flooding in urban environments

Numerous tools exist to assist with flood hazard management in urban environments. These include spatial analysis programs and computational models, and when combined together, can provide practitioners with valuable information. Such analyses include land use planning, rainfall-runoff analyses, channel design, floodplain management, and flood estimation. These tools combine together to form the flood hazard management process (see Figure 1) that can be applied in urban environments to reduce flood hazard.

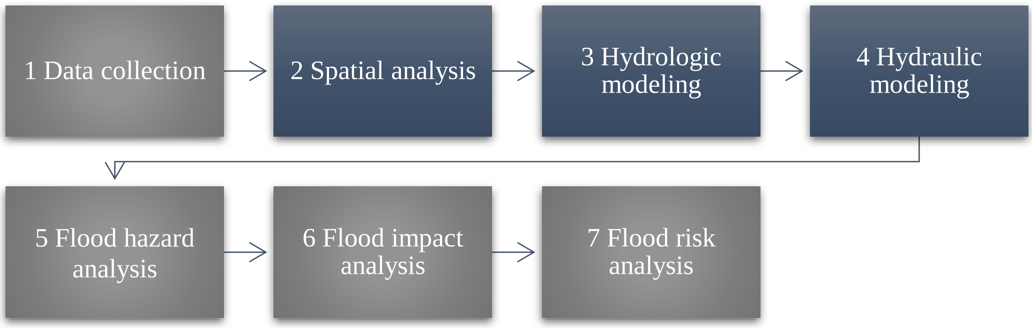


Figure 1: Flood hazard management process in urban environments

2.2.1 Spatial analysis and remote sensing

Spatial analysis systems such as GIS allow users to capture, store, analyze, and display geospatial data for purposes such as land-use planning, natural disaster management, and emergency planning (Chang 2014). In combination with remote sensing imagery, it can be a very efficient and reliable tool for assessing the spatial distribution of land-use changes over time. This assists with prediction of future growth which can aid in land-use planning and flood hazard management. As an example of an application in urban environments, Nirupama and Simonovic (2007) developed a relationship between higher peak flows in the Upper Thames River watershed and impervious areas in the City of London. Analysis of historical remotely sensed data in combination with hydrological and meteorological data allowed for insight on the impact of urbanization on increased risks of flooding.

Remote sensing imagery such as aerial photographs and satellite imagery provide accurate snapshots of the Earth's land cover through the use of aircrafts and satellites, respectively. Aerial photographs can be analyzed with GIS software which allow for changes in land-use to be observed through manual digitizing (Al-Bakri et al. 2001). This method is sufficient for small projects seeking to obtain a general understanding of the temporal changes in development patterns. On the other hand, satellite images can be converted into pixelated raster images using automated classification techniques and programs such as IDRISI, where it is then much easier to distinguish between the different types of land cover (Nirupama and Simonovic 2007). This method is well-suited for larger projects where more accurate and detailed analysis is required. Depending on data availability, financial limitations, and the purpose of the work, aerial photography may or may not be the better option over satellite imagery. Satellite imagery is a newer technology and thus may be limited in terms of long-term historical analysis. However, satellite imagery contains multispectral attributes which allow for more advanced analyses. Once remote sensing imagery has been analyzed this information can be inputted into computational models to investigate hydrological processes.

Nirupama and Simonovic (2007) demonstrated the benefits of using GIS and remote sensing imagery to assist with flood risk management and land-use planning. This study developed a relationship between higher peak flows and impervious areas by analyzing remotely sensed data with hydrological and meteorological data. The goal of this study was to use the City of London as a study site to show that the risks of flooding significantly increase due to continuous urbanization. This study collected historical Landsat images, analyzed the land-use change using computational methods, and compared the results to historical river flows and meteorological events over time. It was observed that for the earlier years, a larger precipitation event would create lower peak flows whereas the later years produced high peak flows to smaller precipitation events. Based off this observation, it was concluded that increasing urban development over the years has significantly increased the risk of flooding. This analysis has demonstrated the important relationship between increasing urban development and the risks of flooding.

2.2.2 Computational modeling

Due to its efficiency and reliability computational modeling is a widely used tool for assisting with flood hazard management. These models provide users with a convenient and interactive tool for predicting hydrologic and hydraulic processes within the environment. Computational models are approximations of real-world systems. These models exist in many forms, each based on specific principles. Calibration, validation, and verification are critical components of modeling applications. Calibration involves altering model parameters until the output results consistently match an observed set of data. This process relies on an extensive amount of data which is not always available for the area of interest. Model accuracy depends on the level of calibration accomplished. Validation is a comparison of output results with an independent data set, without any alterations to the model parameters. Verification involves checking that the model is functioning correctly and that the logical structure makes sense. It is also crucial to understand model operations and their capabilities since all models have unique advantages, disadvantages, abilities, and purposes. The characteristics of the study area or availability of data are large factors in selecting the appropriate modeling program. The below sections discuss hydrologic and hydraulic computational models as they relate to urban flood hazard management.

2.2.2.1 Hydrologic modeling

Hydrological modeling enables users to study the movement of water in a watershed and quantify the amount of water that is drained in a period of time. This modeling aims to mimic the hydrologic cycle by quantifying runoff, infiltration, snowmelt, groundwater, and evapotranspiration based on a meteorological event (Hingray et al. 2015). Hydrologic models are commonly used for rainfall-runoff simulations and reservoir/channel routing. Applications of hydrologic models include flood protection, flood forecasting, stream restoration, and design of reservoirs and storage ponds (Chin 2013). Hydrological models have been applied to quantify the impacts of land-use change on various hydrological processes in order to assist with the flood hazard management process (Im et al. 2009; Wijesekara et al. 2012; Olechnowicz and Weinerowska-Bords 2014).

Hydrologic models can be classified based on criteria such as parameter relationships, treatment of space, and treatment of time. Hydrological models can be categorized as stochastic or deterministic depending on the relationship between parameters within the model. Stochastic hydrologic models are based on probability distributions so that random outputs for the same input parameters are produced. This type of modeling is useful for predicting uncertainty and is not typically used for channel routing applications. Deterministic hydrologic models are very commonly used for rainfall-runoff response and routing as they produce the same output for the same input parameters.

Deterministic hydrologic models can be classified based on spatial characteristics as lumped, semi-distributed, or distributed models. Lumped hydrologic models do not allow the parameters to vary spatially within the watershed. In other words, the watershed is evaluated as one unit instead of as a series of individual basins. Some lumped models do not take into account all of the hydrological processes such as infiltration and snowmelt as they are a simplified representation of the real-world. However, a lumped model may be the preferred option if the application of the model is primarily to predict discharge in urban environments with a minimal amount of input data and a short computational time. Distributed hydrologic models are the most common model type used in urban environments as they allow the parameters to fully vary spatially, best representing real-world conditions. This is the most appropriate type of model for detailed and accurate analyses where flood forecasting or design of stormwater management features is the primary concern. However, this type of modeling can be data intensive and time-consuming. Semi-distributed models provide an excellent alternative since they are a combination of both types of models, providing more accuracy than lumped models yet requiring less data than distributed models (Cunderlik 2003).

Deterministic hydrologic models can be also classified based on temporal characteristics as event-based or continuous simulations. Event-based simulations model short-term hydrologic events and are typically used in flood forecasting scenarios or in the design of stormwater control facilities. Continuous simulations model the periods in between hydrologic events and simulate all conditions in the selected time period which can include anything from low flows to flood discharges (Hingray et al. 2015). These are particularly useful in long-term analyses where, for example, the determination of the water balance in a watershed is important.

Common input required for hydrologic modeling consist of precipitation, flow rates (for calibration), temperature, wind speed, evapotranspiration (if known), topographic information (slope, elevation), and thematic data (land-use, soil characteristics) (Cunderlik 2003; Hingray et al. 2015). However, the specific input will vary depending on the selected model, the goal of the modeling, and the complexity of the study area. Precipitation is the most important meteorological variable and is input in the form of a hyetograph produced from rain gages or design storms. Some models offer the capability of spatializing rainfall across a region based on various methods such as Thiessen Polygon, Inverse Distance Weighting, and Kriging. If applicable, snowmelt can be calculated from wind speed, temperature, and solar radiation parameters. The model then distributes the water to various processes based on the water balance equation which is generally expressed as:

$$P + G_{in} - (Q + ET + G_{out}) = \Delta S \, , \tag{2}$$

where P represents precipitation, G_{in} represents groundwater inflow, Q is the stream outflow, ET represents evapotranspiration, G_{out} represents groundwater outflow, and ΔS is the change in storage over the period of time (Dingman 2008). Hydrological processes that are physically calculated within these models include infiltration, evapotranspiration (if not known), groundwater flow, interception, and runoff. Infiltration can be calculated from various methods such as Horton's method expressed as:

$$f_p = f_\infty + (f_o - f_\infty)e^{-\alpha t} \, , \tag{3}$$

where f_p is the infiltration capacity into the soil [LT^{-1}], f_∞ is the minimum or ultimate value of f_p [LT^{-1}], f_o is the maximum or initial value of f_p [LT^{-1}], α is a decay coefficient [T^{-1}], and t is the time from the beginning of the storm [T] (Horton 1939; Horton 1940).

A runoff hydrograph is typically the desired output for these types of models. Hydrologic flow routing, which is based on the continuous solution of the continuity equation and a second equation that relates storage volume to inflow and outflow can be used to determine this output. The continuity equation can be expressed as:

$$\frac{dS}{dt} = I(t) - O(t) \, , \tag{4}$$

where S represents the storage between the upstream and downstream sections [L^3], t is time [T], $I(t)$ is the inflow rate at the upstream section [L^3T^{-1}], and $O(t)$ is the outflow rate at the downstream section [L^3T^{-1}] The simplicity and reasonable accuracy of routing within hydrologic models make them an appealing alternative to hydraulic routing (Chin 2013), which is discussed in the next section.

2.2.2.2 Hydraulic modeling

Typically, the runoff hydrograph resulting from hydrologic models provides the input into hydraulic models for investigation of kinetic flow properties within a stream network. This type of modeling is capable of predicting such quantities and processes as stream power, water levels, flow velocities, water quality, and sediment transport. This information is important in determining bank stability and areas prone to higher risks of erosion or flooding. Floodplain mapping, determination of flow around hydraulic structures, and flow routing are common applications of hydraulic models. Flow routing in hydraulic models is generally preferred over hydrologic models where backwater effects are significant and where the channel is either very flat or very steep (Chin 2013). Previous research has applied hydraulic models to predict flood inundation zones, investigate bank stability, determine the benefits of reservoir storage in minimizing the risks for flooding, and assessing the effects of urbanization on channel morphology (Horritt and Bates 2002; Nelson et al. 2006; Yang et al. 2006; Chang et al. 2008; Owusu et al. 2013; Akbari et al. 2014).

Hydraulic models are also classified according to spatial and temporal characteristics. These models can be broken down into one-dimensional, two-dimensional, and three dimensional models depending on the assumed direction of flow. One-dimensional models assume only longitudinal direction. Based on this, only basic parameters can be determined such as average velocities, water surface elevation, and sediment transport loads (Papanicolaou et al. 2008). These types of models are commonly used for engineering design and flood risk analysis for open-channels (Wang and Yang 2014). Two-dimensional models assume either longitudinal and lateral directions or longitudinal and vertical directions. They are capable of calculating spatially varied water depth and bed elevations, streamwise and transverse velocity components, as well as sediment transport rates. Three-dimensional models assume longitudinal, lateral, and vertical directions, adding computational effort while allowing for more complicated analyses (Papanicolaou et al. 2008; Tonina and Jorde 2013). These types of models are capable of determining flows around hydraulic structures, flows through spillways, along with flows and sediment transport rates over complex bed morphologies (Wang and Yang 2014).

Steady and unsteady flow simulations are available in most hydraulic models. Steady simulations represent flow conditions that are constant with time whereas unsteady simulations represent flow conditions that vary with time (Sturm 2010). Steady simulations can be used for water surface profile computations in single channels, dendritic systems, or a network of channels. Unsteady simulations are most commonly used as they best-represent real-world conditions, and are capable simulating flow through a network of open channels (U.S. Army Corps of Engineers 2010).

Common input required for hydraulic modeling include flow rates (calibration), inflow hydrographs, grain size distributions, geometric data such as cross-section data, reach lengths, energy loss coefficients, junction information, boundary conditions, initial conditions, and hydraulic structure data (U.S. Army Corps of Engineers 2010). However, similar to hydrologic models, this varies depending on the selected model, the goal of the modeling, and the complexity of the study area. In hydraulic models, unsteady open-channel routing is achieved through simultaneous numerical solution of the continuity and momentum equations. These equations are commonly known as Saint-Venant equations, depth-averaged shallow water equations, and 3D Navier-Stokes equations in one-dimensional, two-dimensional, and three-dimensional models, respectively. As an example, in one-dimensional hydraulic models, the Saint-Venant equations are expressed as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0\,, \tag{5}$$

and

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + g\frac{\partial y}{\partial x} - g\left(S_0 - S_f\right) = 0\,, \tag{6}$$

where Q is the flow rate [L^3T^{-1}], x is the distance along the streamwise direction [L], A is the cross-sectional area [L^2], t is time [T], g is the universal gravity constant [LT^{-2}], y is the flow depth [L], S_0 is the slope of the channel, and S_f is the slope of the energy grade line. When the full momentum equation is used it can also be referred to as the *dynamic model*. However, in many situations some terms in the momentum equation can be neglected due to their small or negligible values. This simplifies the numerical solution and reduces computational efforts. The *diffusion model* neglects the inertial terms whereas the *kinematic model* neglects the inertial and pressure forces. The diffusion and kinematic models are expressed as:

$$g\frac{\partial y}{\partial x} - g\left(S_0 - S_f\right) = 0\,, \tag{7}$$

and

$$\left(S_0 - S_f\right) = 0\,, \tag{8}$$

respectively. Applicability of these models depends on the conditions present in the channel. Dynamic models are ideal for complicated analyses and where the bed and water surface slopes are relatively small. Diffusion models should be used in situations where backwater effects occur and when inertial terms can be neglected (Chin 2013). Kinematic models are suitable in situations where there are no backwater effects and when the slope is relatively steep (Mujumdar and Kumar 2012).

2.3 Gaps in literature

As discussed in this Chapter, a considerable amount of research has contributed to the topic of flood hazard management. Previous studies have applied spatial analysis systems, such as manual digitization and automated classification techniques, to evaluate land use change over time and predict future changes in land use (Qaiser et al. 2012; Wijesekara et al. 2012). Other previous research has demonstrated that historical changes in land use has had a considerable impact on watershed hydrology and increased flood hazard in urban environments (Nirupama and Simonovic 2007; Im et al. 2009; Qaiser et al. 2012; Suriya and Mudgal 2012; Wijesekara et al. 2012). LID measures have been simulated individually and in combinations to evaluate their capabilities at reducing stormwater runoff and therefore flood hazard (Joksimovic and Alam 2014; Ahiablame and Shakya 2016; Bloorchian et al. 2016; Chui et al. 2016).

Including the effects of modifications to urban rivers and streams through processes such as channelization, straightening and lining of stream banks with concrete, in land use analyses is missing in literature. Most urban creeks or rivers have undergone modifications in the past to promote more efficient drainage of water. As a result of more frequent and intense storm events occurring, these channels are no longer able to handle the extra volumes of water without overtopping their banks. While the effects of land use change have been previously studied, the combination of land use change and modifications of natural rivers and streams has yet to be evaluated.

Several previous studies have created hypothetical scenarios of LID measures and evaluated their abilities to reduce stormwater runoff and flood hazard. These scenarios are hypothetical and do not take into consideration realistic municipal operating budgets for stormwater management, which is a critical component of decision-making. Previous research has shown that LID measures are capable of reducing significant amounts of stormwater runoff, however, have lacked more realistic and applied approaches. For example, looking at a neighborhood or subcatchment and identifying how many driveways could be converted to permeable pavement or determining exact locations where other LID measures could be retrofit. The effects of LID measures have been studied at various scales, however, conditions vary considerably from watershed to watershed and the results cannot be easily compared. At present, the ability to evaluate the capabilities of LID measures on different scales for the same location is missing. Doing so would produce a much better understanding of their effects at different scales and would promote more efficient and appropriate planning and investments in LID measures.

Chapter 3

3 Description of study area

The Black Creek watershed, located in southern Ontario, was chosen as the study area for this research (see Figure 2). It is managed by multiple organizations such as the Toronto and Region Conservation Authority, the provincial government, and the local municipalities of Vaughan and Toronto. The Black Creek, Main Humber, East Humber, West Humber, and Lower Humber are subwatersheds of the Humber River watershed, with Black Creek acting as one of the tributaries to the Humber River (see Figure 3).

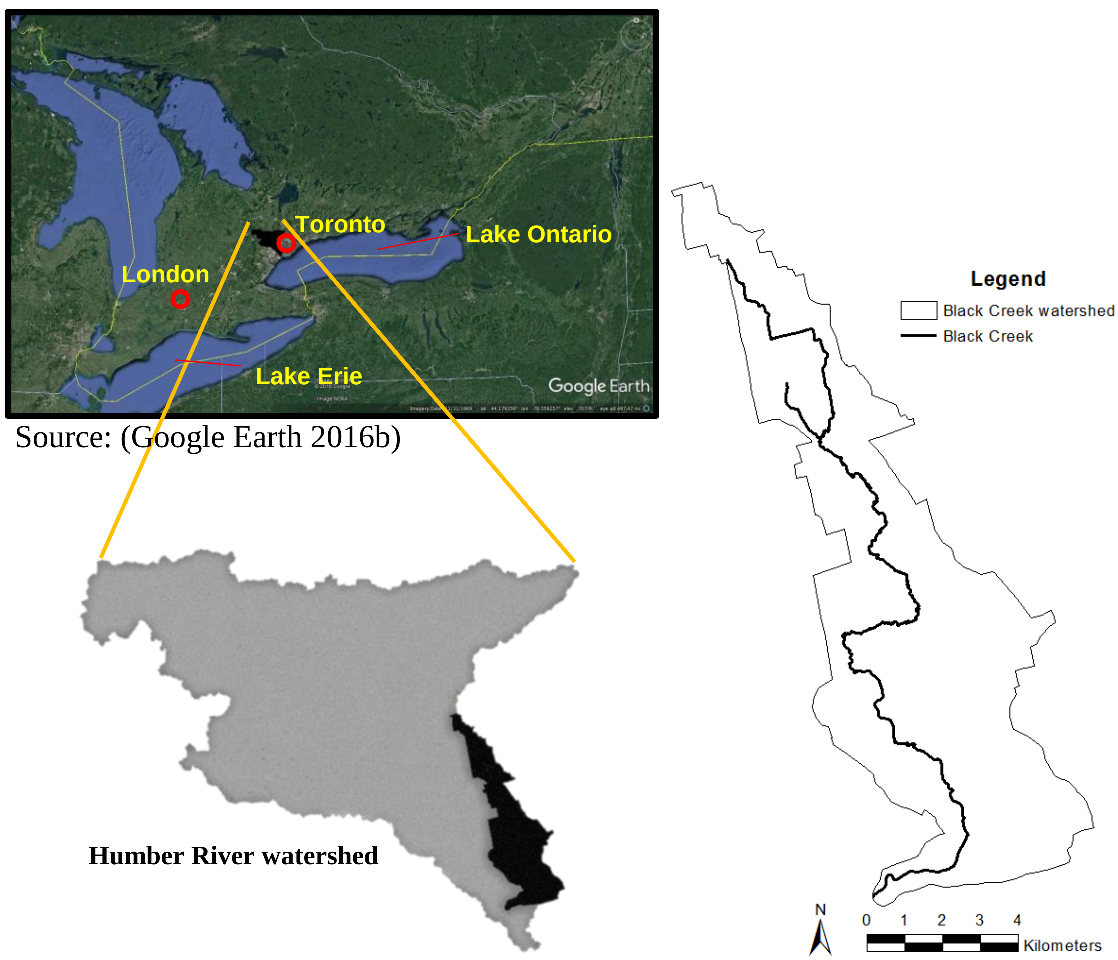


Figure 2: Black Creek watershed located in southern Ontario, Canada

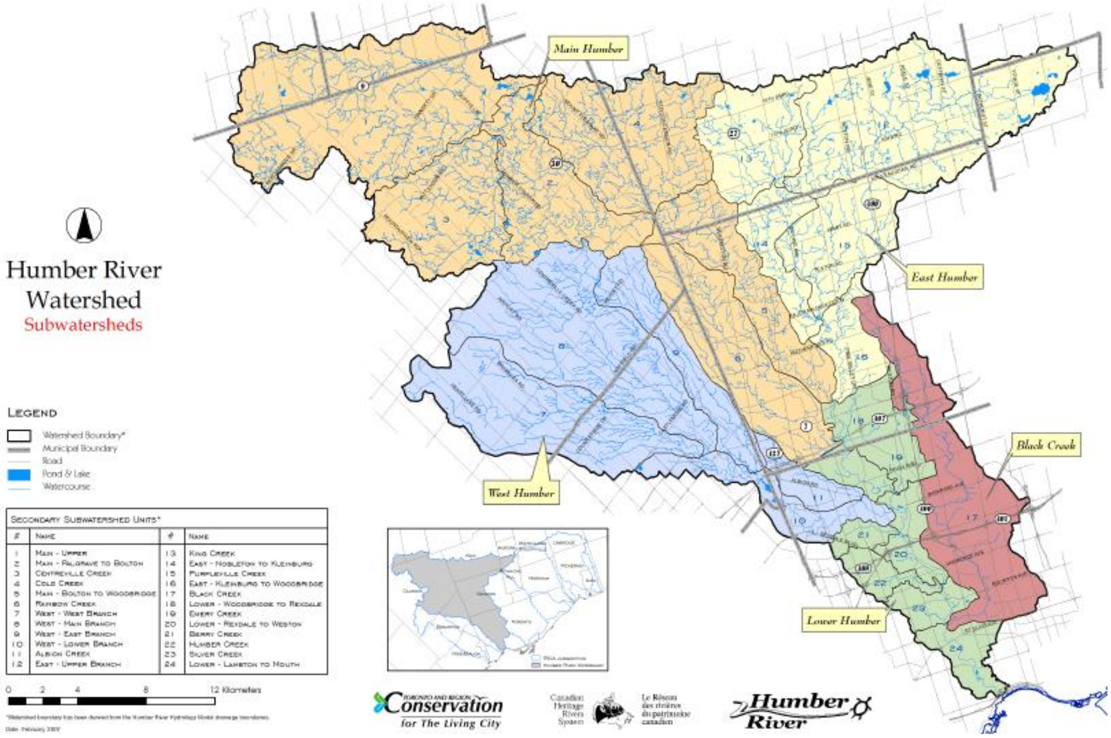


Figure 3: Subwatersheds of the Humber River watershed (TRCA 2008)

On an annual basis, the Humber River watershed typically receives 798 to 933 millimeters of precipitation. Runoff from the Black Creek watershed can be classified as flashy with relatively high peak flows. It lies within the South Slope physiographic region, with some sections in the Peel Plain and Lake Iroquois Sand Plain. As a result, the watershed is gently sloped and consists mainly of glacial till with regions of sand, silt, and clay as well (TRCA 2008).

Draining an area of approximately 65 km^2, the Black Creek watershed is fully developed with a mixture of residential, commercial, institutional, and industrial land use. Presently, the subwatershed consists of 76% urban cover, 15% open land, 8% forest cover, and 1% water. Approximately 48% of this land use is considered to be impervious (TRCA 2008). Prior to the shifts towards stormwater management, flood prone segments of Black Creek were converted into concrete channels to allow for more rapid movement of water downstream. These segments are located in the southern part of Black Creek. The most southern segment of Black Creek consists of a very large constructed rectangular concrete channel, surrounded by a residential neighborhood and school zone. Moving upstream, Black Creek becomes a concrete trapezoidal channel, which is surrounded by major roadways and commercial parking lots (see Figure 6).

Today, Black Creek is sensitive to flooding in many areas due to the high levels of imperviousness, large amounts of runoff, concrete lined channels, lack of natural vegetation, and effects of climate change. This is shown in Figure 4, with pink areas representing flood vulnerable areas and orange markers representing flood vulnerable roads.

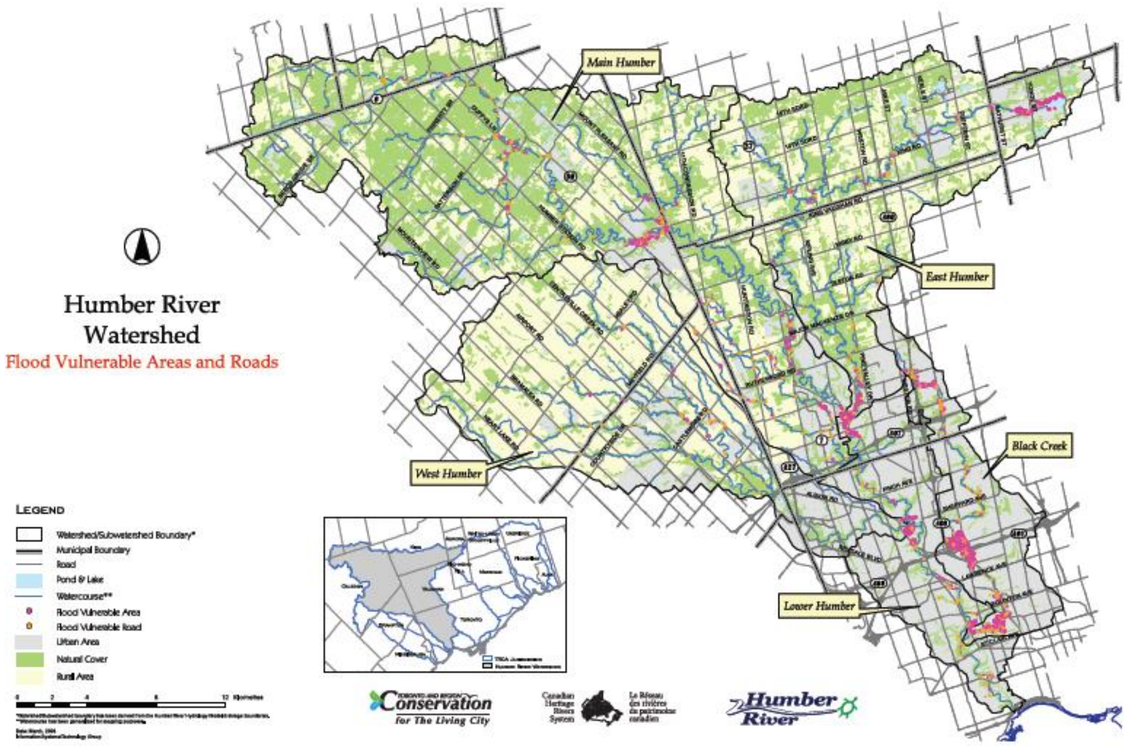


Figure 4: Flood vulnerable areas and roads in the Humber River watershed (TRCA 2008)

Stormwater management facilities for controlling erosion, quantity, and quality have been installed in newer developments north of Highway 7 (see Figure 5). Most of the watershed was developed prior to stormwater quality and quantity control measures, and as a result, the region south of Highway 7 contains minimal runoff control (TRCA 2008). However, due to the lack of available land, opportunities for development of stormwater ponds is very low.

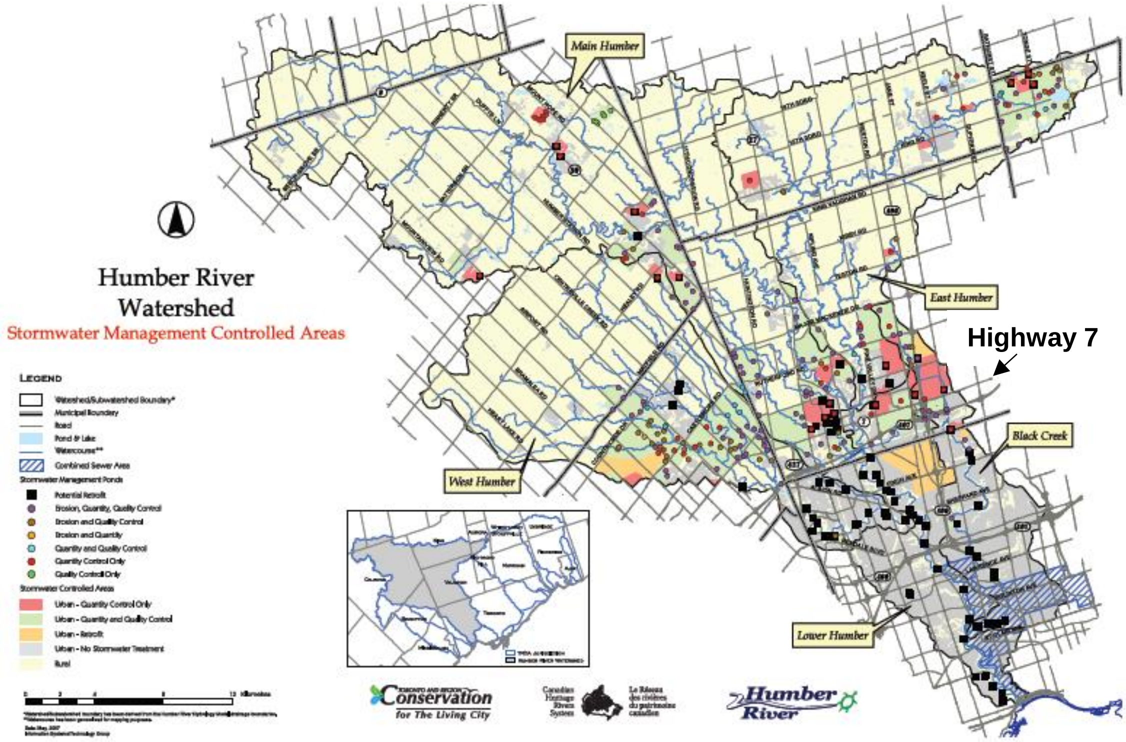
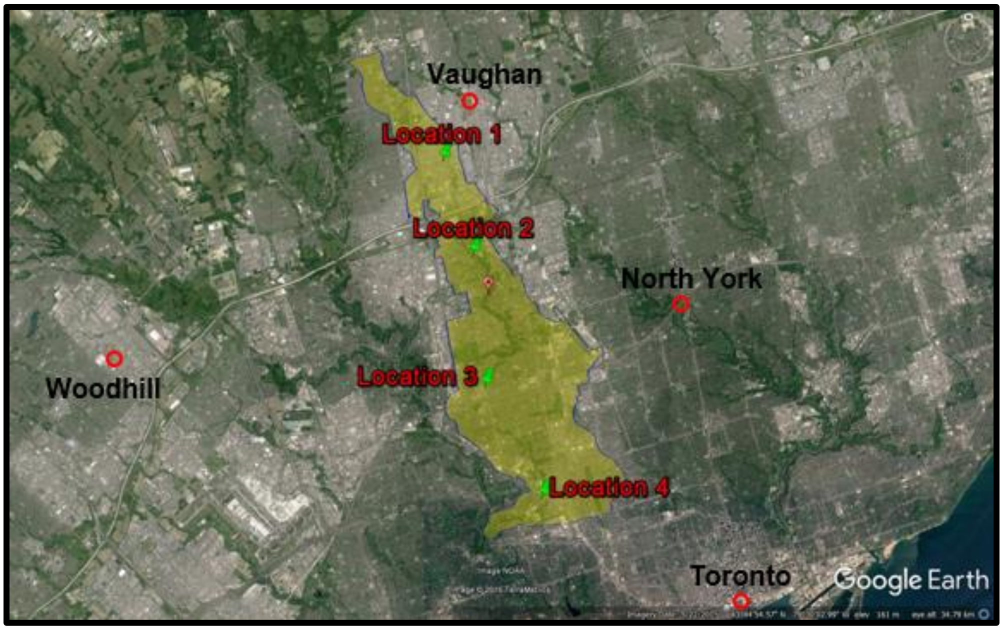


Figure 5: SWM controlled areas in the Humber River watershed (TRCA 2008)

The channel characteristics of Black Creek varies significantly throughout its watercourse. Some sections have been left natural, while others have been lined with concrete or transformed into a linear concrete waterway. Figure 6 depicts the conditions of the northern, middle, and southern sections. The northern section of Black Creek demonstrates a very natural environment with a narrow waterway (location 1). As you move downstream, a natural setting is still present but with a wider and much shallower channel (location 2). Images of the lower half of Black Creek show the modified concrete channel constructed in the 1960s for efficient drainage purposes (locations 3 and 4). These sections are highly susceptible to flooding now due to the lack of natural vegetation and high amounts of runoff being received from surrounding development. This is especially concerning considering these channel segments are surrounded by major roadways, commercial parking lots, residential neighborhoods, and school zones. Basement flooding is also a major concern in the southern portion of the Black Creek watershed, marked with a blue hashed area in Figure 5, due to the presence of combined sewer systems.

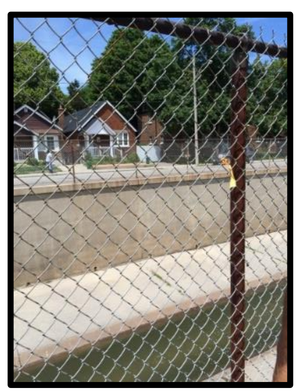


Source: (Google Earth 2016a)

Location 1

Location 2

Location 3



Location 4

Figure 6: Channel characteristics of Black Creek

Black Creek originates at an elevation of 209 meters in the City of Vaughan from a retention pond, called Fossil Hill Pond, near the intersection of Rutherford Road and Weston Road. The creek heads southeast travelling under the intersection of Highway 400 and Langstaff Road, briefly flows south, and turns east towards Jane Street. It flows south along Jane Street, turns east under Jane Street, and continues south into a stormwater management pond near the intersection of Jane Street and Highway 7. The channel heads south under Highway 7 and Highway 407, then moves southwest under Jane Street. A separate channel originating from the southeast side of the intersection of Highway 400 and Portage Parkway merges at this location. From this junction, Black Creek continues southeast under the intersection of Steeles Avenue West and Jane Street, then under Shoreham Drive. It flows south through the Black Creek Parkland and then under Finch Avenue West. Black Creek heads southwest through Derrydowns Park, southeast through Northwood Park, south under Sheppard Avenue West, and heads west through Downsview Dells. It flows under Jane Street into the Oakridge Golf & Country Club and heads south through Chalkfarm Park. The channel crosses under Jane Street and Wilson Avenue, heads south under Highway 401, and moves under Jane Street twice before making its way south under Black Creek Drive and through Upwood Park. It flows southeast under Lawrence Avenue West, Black Creek Drive, then south through multiple parks, and under Eglinton Ave. West. Black Creek heads southwest under Weston Road, and then west through the Rockcliffe-Smythe neighborhood. It flows through the Lambton Golf & Country Club before connecting to the Humber River at an elevation of 104 meters and discharging into Lake Ontario.

Multiple flooding incidents have occurred in Black Creek in recent years (AMEC Environment & Infrastructure 2013), however one particular storm in August of 2005 has been considered to be one of the most devastating and costliest floods in this region. An extreme rainfall event, generating 103 mm of rain in one hour, caused flash flooding and impacted thousands of residents in the Toronto area. Numerous insurance claims were submitted by homeowners as a result of basement flooding caused by sewer backups. This storm event is remembered as washing out a large culvert under Finch Avenue West, creating significant damages to the roadway and further downstream (Environment and Climate Change Canada 2013). As a result of this storm, the City of Toronto approved a Basement Flooding Work Plan that would require comprehensive engineering reviews to be completed in order to address basement flooding in 31 study areas across the City, many of which are within the Black Creek watershed (City of Toronto 2016a). Several of the completed studies have recommended upgrading sewer systems, promoting source controls, constructing storage tanks, and diverting overland flow. This thesis will focus on evaluating LID measures as source controls.

Chapter 4

4 Description of data, model, and methodology

The following chapter discusses the data requirements, hydrologic/hydraulic model used, and the methodology implemented in the present research. Specifications regarding the model will also be reviewed, which consists of an overview of the modeling program, model components selected, setup of the model, design storm events used, model calibration, as well as limitations of the model.

4.1 Description of data

Data collected for this research consisted mainly of historical aerial photographs. Over 400 digital copies, ranging in years from 1946 to 1999, were provided by the TRCA. These photographs were all black and white, with the exception of the 1999 photographs in color, and contained no spatially referenced properties. Photographs from the 1960s were missing and were collected through the City of Toronto Archives (City of Toronto 2016b). No other data was required for this research. A fully operational and calibrated PCSWMM model of the Humber River watershed was provided by the TRCA.

4.2 Description of model

4.2.1 Overview of modeling software

The Storm Water Management Model (SWMM) is the most widely used and accepted model for evaluating stormwater runoff quantity and quality in urban areas. A number of platforms of SWMM exist such as PCSWMM, EPA-SWM, XPSWMM, etc., with all of them using the same SWMM engine. Applications of the model are mainly focused on urban areas but the model can also be used for rural and riverine flooding studies. PCSWMM is a fully dynamic rainfall-runoff model that is capable of simulating hydrologic, hydraulic, and water quality components (James et al. 2010). Event-based and continuous simulation options are available in this model. Inputs include precipitation, flow rates, temperature, wind speed, substratum geology, as well as land-use and soil characteristics. The structure of the model is based on multiple subcatchment areas where the runoff is generated from precipitation and snowmelt. Various hydrologic processes such as infiltration, evapotranspiration, and storage are also simulated from a wide availability of methods. This runoff can then be routed through infrastructure such as pipes, channels, and pumps while tracking the flow rate, flow depth, and runoff water quality. It is capable of evaluating detention storage, SWM practices, LID measures, and water treatment facilities (James et al. 2010; Mujumdar and Kumar 2012). Additional information is included in Appendix A. Denault et al. (2006) applied the SWMM model to the Mission/Wagg Creek Watershed in British Columbia in hopes of reducing the risks for future flooding due to climate change. The study predicted future climate change and then evaluated the effects on future design peak flows and drainage infrastructure. This research provided tremendous insight on future conditions which demonstrated the importance of implementing measures to reduce the risk of future flooding.

LID practices that can be simulated within the model include permeable pavement, rain barrels, infiltration trenches, bioretention cells, rain gardens, green roofs, rooftop disconnection, and vegetative swales. They are mainly represented by surface, pavement, soil, storage, and underdrain layers, with drainage mat and roof drain parameters available for green roofs and rooftop disconnection, respectively. The layers for each LID measure is shown in Table 1. A conceptual representation of LID measures in PCSWMM is shown in Figure 7. The model has been widely used in practice and in research for evaluating LID measures, as discussed in Chapter 2 (Joksimovic and Alam 2014; Ahiablame and Shakya 2016; Chui et al. 2016).

Table 1: Available layers in PCSWMM for modeling LID measures (after James et al. 2010)

LID measure	Surface	Pavement	Soil	Storage	Underdrain
Permeable pavement	x	x		x	o
Rain barrel				x	x
Infiltration trench	x			x	o
Bioretention cell	x		x	x	o
Rain garden	x		x		
Green roof	x		x		
Rooftop disconnection	x				
Vegetative swale	x				

x = required, o = optional

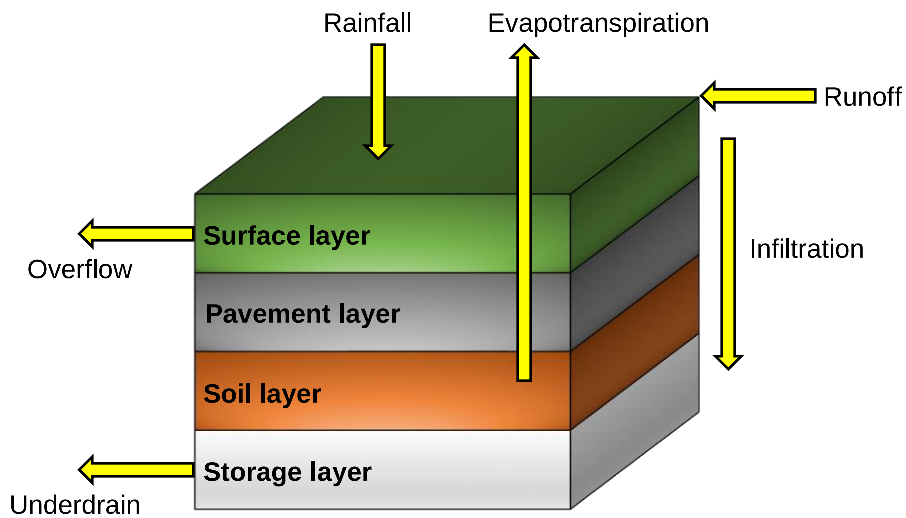


Figure 7: Conceptual representation of LID measures in PCSWMM

4.2.2 Selected model components

PCSWMM is capable of simulating six different process models. These consist of: rainfall/runoff, rainfall dependent infiltration/inflow, snowmelt, groundwater, flow routing, and water quality. For the purpose of this research, only the rainfall/runoff and flow routing process models were activated. The infiltration model chosen was the Green-Ampt method as it was decided to be the most appropriate infiltration approach due to its well-defined physical values. The dynamic wave routing method was selected for flow routing. The temperature, evaporation, wind speed, and snowmelt modules in the climatology editor were not active.

4.2.3 Model setup

The model provided by the TRCA was a fully operational and calibrated model of the Humber River watershed. The focus of this thesis, however, was only on the subcatchments within the Black Creek watershed. This model included hydraulic structures, such as dams and culverts, and current stormwater ponds in place. The Black Creek watershed was broken down into 31 subcatchments, ranging in size from 12.46 hectares to 835.6 hectares. Black Creek was created from 134 channel segments, with 134 observation nodes between each channel segment. Seven stormwater ponds ranging in size from 3,080 m^2 to 65,000 m^2 exist in the upper half of the watershed. The saturated soil hydraulic conductivity rates and average surface slope of each subcatchment ranged from approximately 0.31 mm/hr to 12.3 mm/hr and 1.29% to 3.96%, respectively. Due to the model being fully setup for simulations, minor modifications were required for this thesis. A schematic describing the inputs, model processes, and outputs is shown in Figure 8.

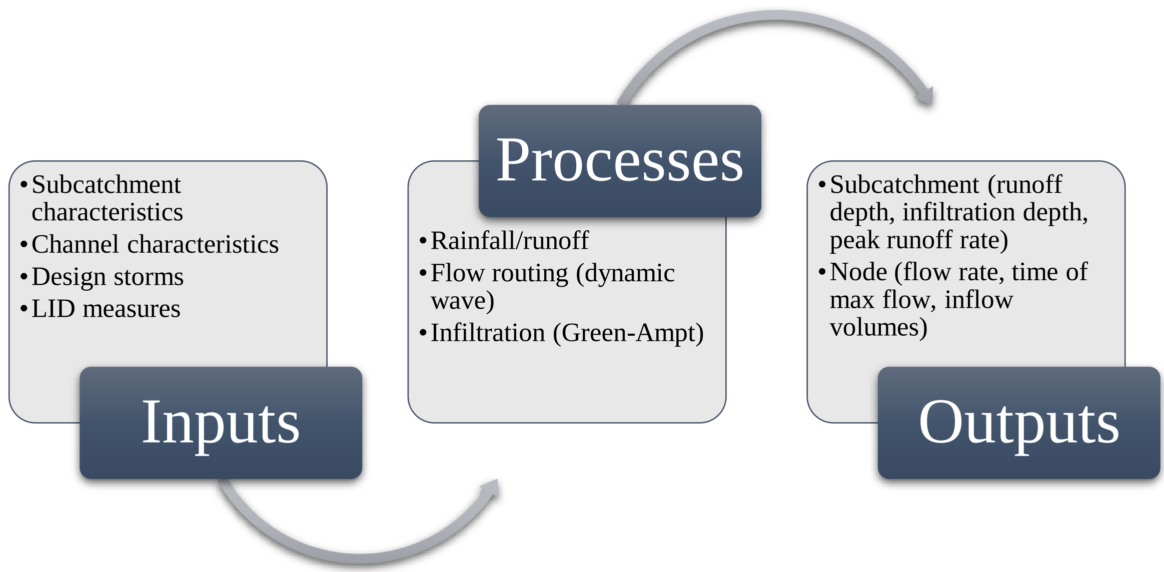


Figure 8: Execution of a modeling simulation

4.2.4 Design storm events

According to the TRCA's stormwater management criteria, the 2, 5, 10, 25, 50, and 100 year storms are to be used for stormwater quantity control in the Humber River watershed (TRCA 2008). These are single storm events and are typically used for stormwater management design. Further, hyetographs with a 4 hour duration and Chicago storm distribution are commonly used in Southern Ontario with a recommended time step of 10 minutes (Lake Simcoe Region Conservation Authority 2013). Eq. 9 represents the IDF curve equation which displays rainfall intensities over storm durations for a storm of a certain return period. This can be written as,

$$I = \frac{a}{(t+b)^c}\ , \tag{9}$$

where *I* is the rainfall intensity in mm/hr, *t* is the time of concentration in hours, and a, b, and *c* are coefficients for each IDF curve. From the City of Toronto's Wet Weather Flow Management Guidelines, a time of concentration of 10 minutes is used and values for coefficients a, b, and *c* are shown in Table 2 (City of Toronto 2006). However, these coefficients are based off the time of concentration being in units of hours. To input these into PCSWMM correctly, they need to be based off the time of concentration being in units of minutes. Since coefficient *b* has a value of 0 and coefficient *c* is a unitless exponent, no modifications were required to coefficients *b* and c. By calculating the rainfall intensities for each return period with the initial values, a backwards calculation is done to solve for the proper value of a. These new values are also shown in Table 2 and are used as inputs in PCSWMM.

Table 2: IDF curve coefficients

Return period	Coefficient *a*	Coefficient *b*	Coefficient *c*	Rainfall intensity (mm/hr)	New coefficient *a*
2	21.8	0	0.78	88.2	531.39
5	32.0	0	0.79	131.8	812.62
10	38.7	0	0.80	162.3	1023.84
25	45.2	0	0.80	189.5	1195.80
50	53.5	0	0.80	224.3	1415.39
100	59.7	0	0.80	250.3	1579.41

The new coefficient a values, original values for coefficients b and c, a storm duration of 4 hours, and a rainfall interval of 10 minutes is input for each return period. The 4 hour Chicago hyetograph distribution for each return period is generated by PCSWMM and presented in Table 3.

Table 3: 4 hour Chicago hyetographs generated by PCSWMM

Time (hr:min)	Rainfall intensity (mm/hr)					
	2 year	5 year	10 year	25 year	50 year	100 year
0:00	1.708	2.362	2.684	3.135	3.711	4.141
0:10	1.899	2.629	2.992	3.495	4.136	4.616
0:20	2.146	2.976	3.393	3.963	4.691	5.234
0:30	2.483	3.449	3.94	4.601	5.446	6.077
0:40	2.971	4.137	4.736	5.531	6.547	7.306
0:50	3.755	5.245	6.022	7.033	8.325	9.29
1:00	5.272	7.396	8.53	9.963	11.792	13.159
1:10	10.087	14.276	16.609	19.398	22.961	25.621
1:20	88.108	131.675	162.13	189.361	224.134	250.107
1:30	13.564	19.267	22.496	26.275	31.1	34.703
1:40	7.878	11.11	12.879	15.042	17.804	19.867
1:50	5.772	8.106	9.359	10.931	12.939	14.438
2:00	4.629	6.482	7.463	8.717	10.317	11.513
2:10	3.899	5.449	6.26	7.311	8.653	9.656
2:20	3.389	4.727	5.42	6.331	7.493	8.362
2:30	3.009	4.191	4.798	5.604	6.634	7.402
2:40	2.714	3.776	4.317	5.042	5.968	6.66
2:50	2.478	3.443	3.933	4.593	5.436	6.066
3:00	2.284	3.171	3.617	4.225	5.001	5.58
3:10	2.122	2.942	3.354	3.917	4.636	5.174
3:20	1.984	2.748	3.13	3.656	4.327	4.828
3:30	1.865	2.581	2.937	3.43	4.06	4.531
3:40	1.761	2.435	2.769	3.234	3.828	4.272
3:50	1.669	2.307	2.621	3.062	3.624	4.044
4:00	0	0	0	0	0	0
Rainfall depth (mm)	29.57	42.81	51.06	59.64	70.59	78.77

4.2.5 Model calibration

As the model provided by the TRCA was already fully calibrated (AMEC Environment & Infrastructure 2012) and ready for operational use, no further calibration was required. However, the storm events used for model calibration and verification will be discussed in the following section. Six rainfall events were used for model calibration based on a few criteria including watershed coverage, produced relatively high flows to evaluate the reliability of streamflow data, and a minimum average rainfall of 20 mm. These events are:

- June 13, 2005
- July 14, 2005
- July 10 to 12, 2006
- October 11, 2006
- May 15, 2007
- September 14, 2008

Observed rainfall data from nine rainfall gauges and observed streamflow data from nine streamflow gauges throughout the Humber River watershed were used for the calibration of the model. A sensitivity analysis was conducted on model parameters with hydraulic conductivity, suction head, and initial moisture deficit ultimately being chosen as the calibration parameters. In addition, radar-based rainfall data from the following storm events were used for model verification:

- August 19, 2005
- November 15, 2005
- July 10, 2006
- October 11, 2006
- October 17, 2006
- November 30, 2006

The simulated responses of the model have been classified as having close matches to the observed response at the Black Creek stream gauge for peak flows, with relatively close correlations for runoff volumes (AMEC Environment & Infrastructure 2012).

4.2.6 Model limitations

The PCSWMM model has important limitations that should be understood. One limitation included assigning flow length values for each subcatchment. These values represent the distance from the inlet to the furthest drainage point in the subcatchment and it is recommended that they not exceed 500 feet or approximately 150 m (James et al. 2010). Values ranged between 30 and 145 m, independent of subcatchment size. As a result, sharply-peaked runoff hydrographs are produced from larger subcatchments that would typically have much larger flow length values in the real world. The response is unrealistic and provides limitations on being able to only observe the peak flow values and not the overall shape of the hydrograph. The effectiveness of LID measures at increasing the times to peak will therefore go unnoticed due to the model setup.

Another limitation in the model was the lumped approach used for distribution of LID measures in the subcatchments. Two methods exist for inputting LID features in PCSWMM. The first approach involves creating individual polygons that can represent each LID. The second approach involves assigning them to a subcatchment that has been already created. The latter approach was used in order to prevent any modifications to the existing calibrated model. The limitation with this approach is that LID measures will be lumped in space and the exact location in a subcatchment cannot be specified. A hypothetical volume of runoff is assigned to each measure and the overflow volume is sent to pervious surfaces or to the subcatchment outlet. With this method, LID measures are simulated together and not individually which creates a group response of reducing peak flows and volumes of runoff.

4.3 Methodology

The methodology of this thesis is provided in Figure 9. The flow diagram demonstrates how the land use analysis and hydrologic modeling were used to achieve objectives (1) and (2).

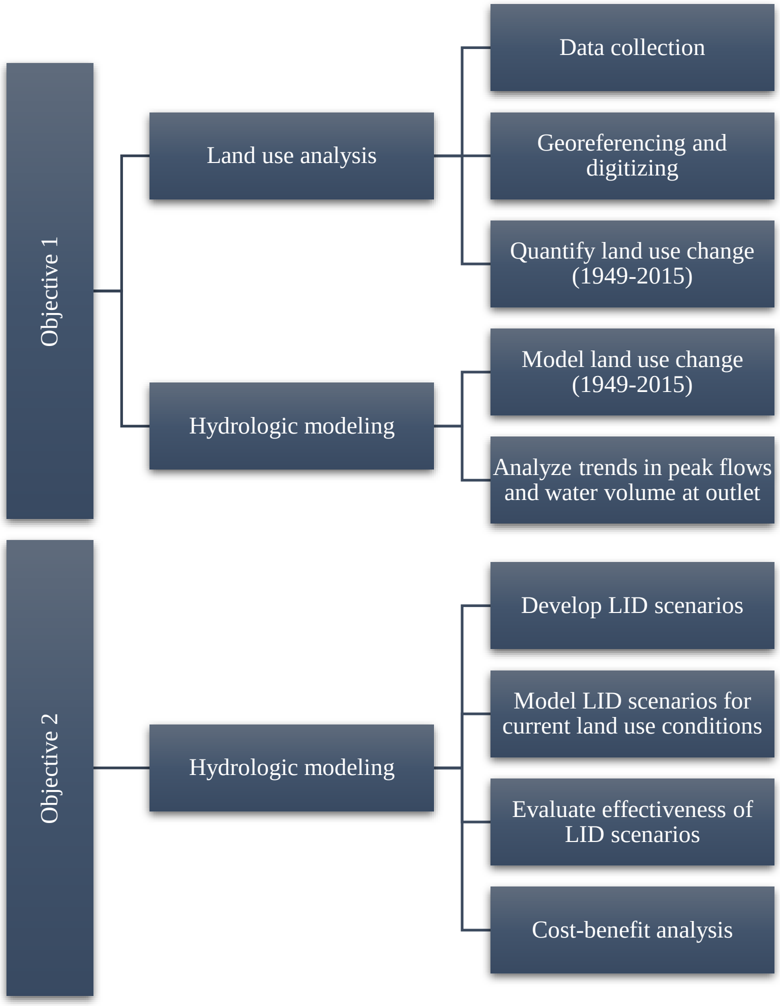


Figure 9: Methodology for land use analysis and hydrologic modeling

4.3.1 Land use analysis

In order to satisfy objectives (1) and (2) of the thesis, land use change was initially evaluated over an extensive period of time to determine correlations between urbanization and changes in hydrographs. Available historical aerial photographs dated back to 1949 with the most recent time period being 1999. A more recent view of the region was simply viewed in ArcMap by adding a world imagery basemap available in the program, providing an evaluation period between 1949 and 2015. This provided a sufficiently long time period for an accurate land use analysis, and was conducted in approximate 10 year increments. The photographs were georeferenced and digitized in the computer program ArcMap in order to quantify the changing land use over time. Results from this section are incorporated into the analysis.

4.3.2 Hydrologic modeling

In order to satisfy objective (1), simulations of design storms were run from 1949 to 2015 in approximate 10 year increments. Using the historical land use analysis results from Section 4.3.1, each subcatchment was updated with their total imperviousness percentage for each time period. These simulations provided insight on the impact of historical urban development on peak flows and water volume in Black Creek, and therefore its effect on flood hazard over time.

In order to satisfy objective (2), hypothetical scenarios were developed that incorporated different levels of investment of LID measures in a wide variety subcatchments and locations across the watershed. These scenarios were created by analyzing aerial views of each region and allocating suitable LID measures, dependent on the type of land use, as well as a reasonable quantity. Once these scenarios were created, hydrological simulations in PCSWMM were completed to evaluate the effectiveness of LID measures on reducing peak flows. Available LID practices that can be simulated in PCSWMM consist of bioretention cells, infiltration trenches, permeable pavement, rain barrels, rain gardens, and vegetative swales. All of these measures are suitable for placement in the Black Creek watershed due to the wide variety of land uses currently in place such as residential subdivisions, commercial parking lots, industrial sites, and institutional zones. These LID measures were lumped together in a variety of scenarios to provide an overall effect in each subcatchment. Comparison of various scenarios lead to final conclusions and recommendations on the suitability of LID measures at reducing flood hazard in highly urbanized cities.

Chapter 5

5 Effect of urbanization on flood hazard in Black Creek

This chapter addresses objective (1) of the thesis which investigates the impact of urban development and channel modifications on flood hazard in Black Creek. This chapter provides details on the methodology implemented from the land use analysis and hydrologic modeling. Results from each are presented, discussed, and combined together to determine the effects of historical urban development and channel modifications on flood hazard.

5.1 Land use analysis

5.1.1 Delineation of Black Creek subcatchments

Delineation of the Black Creek watershed and inner subcatchments was completed in ArcMap. A world imagery basemap was added, in the NAD 1983 UTM Zone 17N coordinate system, to act as a reference when working with the aerial photographs. To ensure uniform boundaries throughout the research, the outlines of the Black Creek watershed and inner subcatchments in the PCSWMM model were used as the reference. The PCSWMM model was opened in Google Earth in order to view the boundaries over satellite imagery of the region. Using this as a reference and the world imagery basemap, the Black Creek watershed, along with the 31 subcatchments, were manually delineated in ArcMap. A visual image of this is shown in Figure 10. Although these boundaries are based on today’s topographical data and would differ from 1949, they were used throughout the land use analysis for consistency. This allows for comparison of the land use change from time period to time period.

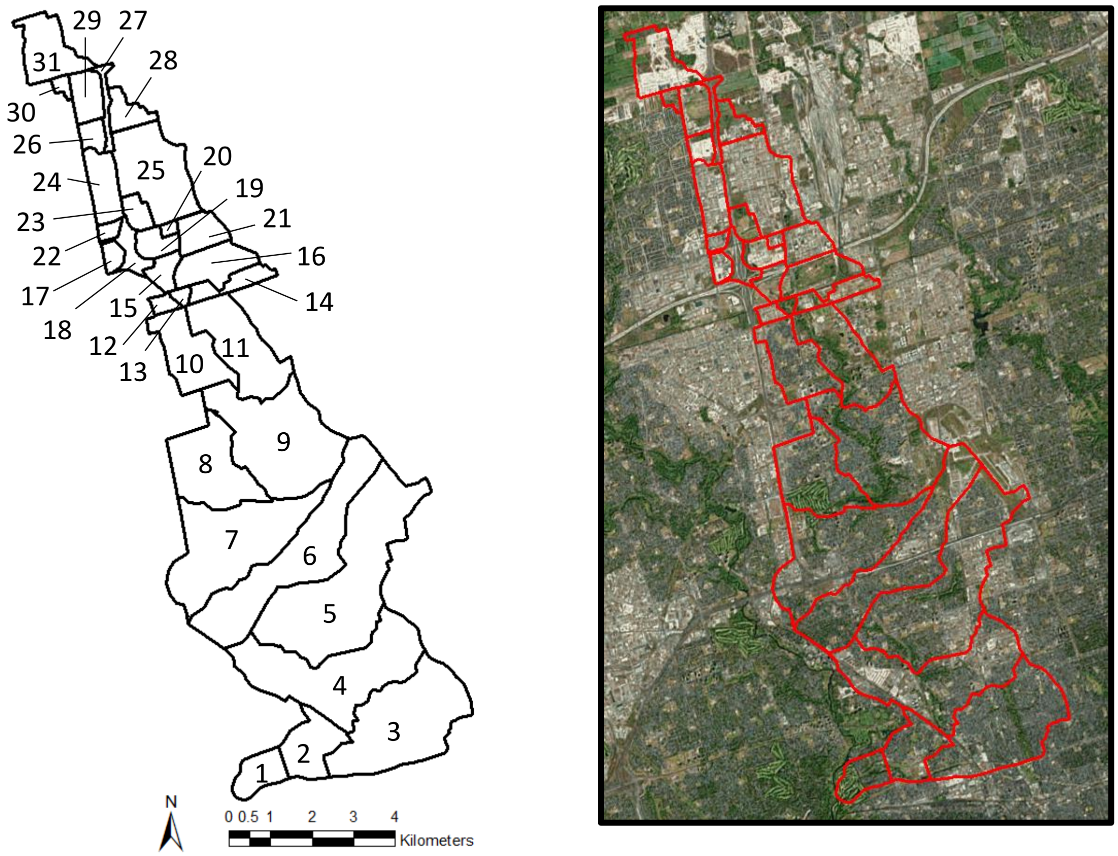


Figure 10: Delineation of the 31 subcatchments

5.1.2 Georeferencing and digitizing aerial photographs

The aerial photographs collected contained no spatial referenced properties and thus required georeferencing in ArcMap. Initially, each photograph was observed with 4 or 8 locations being selected as georeferencing points. These locations were chosen to be distinct features such as road intersections or buildings to ensure the highest accuracy. Most commonly, four locations were selected at or near the corners of the photographs that were large-scale and contained distinct features in each corner. Some photographs that were small-scale or lacked distinct features in the corners required eight georeferencing locations to ensure an accurate fit. Using Google Earth, the latitude and longitude coordinates of each location were determined in decimal degrees. These values were assigned to the locations on each photograph and updated with their new coordinates in ArcMap. If photographs demonstrated poor overlay in some areas, new georeferencing locations were selected until the total RMS error was at its minimum (typically around 5m or less) and the photograph fit well with the world imagery basemap.

All aerial photographs contained black borders which were clipped out in ArcMap. For the years 1949, 1959, 1971, 1981, 1989, and 1999, the photographs were merged together and saved as a layer to provide a complete aerial view of the watershed for that corresponding time period. An aerial image of subcatchment 31 for the year 1999 was missing so this was simply collected from Google Earth's historical land view. Land use categories of commercial/industrial, residential, forest, open land, and water were created in order to begin the digitization process. For each subcatchment of every time period, polygons were manually created to represent the 5 land use categories. To increase efficiency, subcatchments that demonstrated high levels of open land, commercial/industrial, or residential land use, were only digitized with the 4 other categories and the remaining area was considered to represent the main land use. An example of this is shown in Figure 11 where the residential land use was the dominant type of land use and was therefore not digitized.

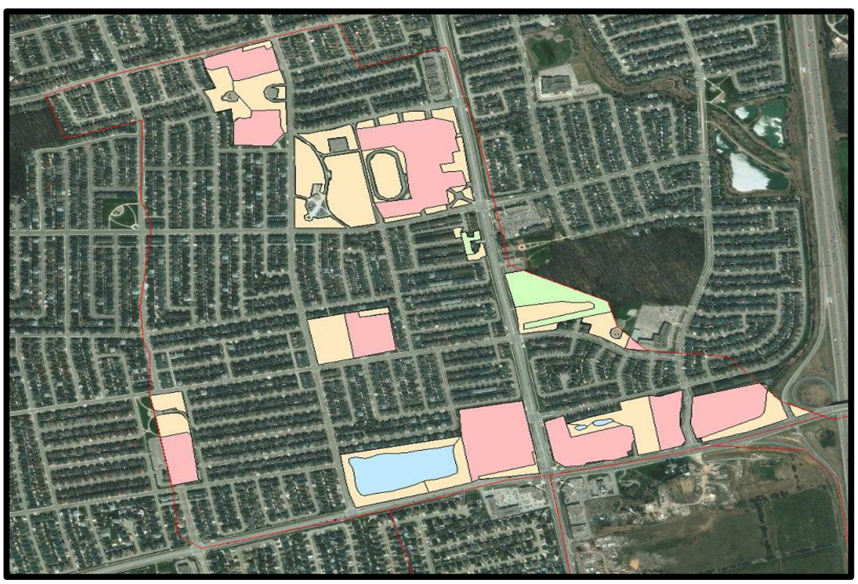


Figure 11: Digitizing subcatchment 31

Polygons representing commercial/industrial, forest, open land, and water are coded pink, green, beige, and blue, respectively. Once digitization was complete, the area of each polygon was calculated in ArcMap in order to determine the total area of each land use for all 31 subcatchments. The same function was used to determine the total area of each subcatchment, in order to calculate the percentage of the land use categories occupied in every subcatchment. Results are shown in Tables 25-30 located in Appendix B.

The world imagery basemap in ArcMap was used to obtain a reliable aerial view of the watershed for 2015. The same process was followed as discussed above for digitizing all 31 subcatchments of the Black Creek watershed into the 5 land use categories, and quantifying the percentage of each. Results are shown in Table 24 located in Appendix B.

5.1.3 Results and discussion

In order to determine the total imperviousness of each subcatchment, imperviousness values were assigned to each land use category (see Table 4). These values are dependent on the user, and in this case, were selected based on numbers commonly seen in literature. Using these values and the results from the land use analysis, the total imperviousness of each subcatchment was calculated. This was determined simply by multiplying the percentage of each land use category by the corresponding imperviousness value and summing up the total. A summary of these results, displayed by period of study, is shown in Table 5.

Table 4: Imperviousness of land use categories

Land use	Imperviousness (%)
Commercial & industrial	95
Residential	55
Forest	1
Open land	2
Water	0

Once the total imperviousness of each subcatchment was determined, the change could be quantified between each period of study. A summary of these changes can also be seen in Table 5. Positive and negative numbers indicate an increase and decrease in subcatchment total imperviousness, respectively. Changes in 5% or greater are marked with red ink in the table. Changes below 5% are considered to be negligible and may result from human error with inconsistencies in the digitization from time period to time period. Table 5 is shown in graphical format in Figures 23-25, located in Appendix C.

Table 5: Temporal variation in subcatchment total imperviousness

Sub. #	2015		1999		1989		1981		1971		1959		1949
	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)
1	28.4	-1.9	30.3	0.3	30.0	1.0	29.0	0.1	28.9	9.0	19.9	6.8	13.1
2	50.2	-3.6	53.8	1.2	52.6	2.0	50.6	-0.7	51.3	11.5	39.8	15.9	23.9
3	56.2	1.6	54.6	-1.8	56.4	-0.4	56.8	0.7	56.1	1.5	54.6	3.7	50.9
4	58.9	-1.1	60.0	-5.0	65.0	-0.7	65.7	3.2	62.5	10.0	52.5	31.4	21.1
5	57.2	0.8	56.4	0.3	56.1	0.0	56.1	2.4	53.7	25.5	28.2	22.7	5.5
6	53.4	-1.5	54.9	2.1	52.8	1.2	51.6	1.0	50.6	18.2	32.4	18.7	13.7
7	53.3	-1.2	54.5	-0.1	54.6	0.2	54.4	3.3	51.1	18.5	32.6	26.5	6.1
8	42.2	-2.1	44.3	2.8	41.5	1.0	40.5	3.5	37.0	28.3	8.7	5.3	3.4
9	39.2	-2.7	41.9	0.4	41.5	2.2	39.3	2.6	36.7	26.4	10.3	6.8	3.5
10	48.5	-5.0	53.5	6.3	47.2	0.8	46.4	8.3	38.1	34.5	3.6	0.9	2.7
11	36.2	1.0	35.2	1.8	33.4	2.0	31.4	3.3	28.1	24.7	3.4	0.5	2.9
12	82.6	-3.2	85.8	8.8	77.0	67.2	9.8	1.0	8.8	0.6	8.2	6.2	2.0
13	15.7	0.0	15.7	0.1	15.6	2.6	13.0	0.5	12.5	7.9	4.6	1.2	3.4
14	38.0	10.3	27.7	4.9	22.8	18.3	4.5	-1.0	5.5	1.8	3.7	1.3	2.4
15	27.2	9.2	18.0	13.9	4.1	1.8	2.3	0.1	2.2	-0.2	2.4	0.1	2.3
16	46.1	24.1	22.0	4.3	17.7	3.9	13.8	-1.7	15.5	12.0	3.5	0.8	2.7
17	87.8	31.7	56.1	53.4	2.7	0.1	2.6	0.1	2.5	0.1	2.4	0.2	2.2
18	49.6	-4.1	53.7	33.1	20.6	2.2	18.4	0.4	18.0	0.2	17.8	11.7	6.1
19	67.1	35.9	31.2	22.2	9.0	1.0	8.0	3.8	4.2	1.1	3.1	0.8	2.3
20	82.1	-0.9	83.0	14.6	68.4	4.3	64.1	59.8	4.3	-3.4	7.7	2.9	4.8
21	81.3	-1.5	82.8	5.7	77.1	5.1	72.0	27.0	45.0	32.8	12.2	8.5	3.7
22	79.4	-2.2	81.6	61.6	20.0	12.0	8.0	4.2	3.8	0.5	3.3	0.9	2.4
23	41.4	24.1	17.3	5.1	12.2	3.2	9.0	2.5	6.5	1.6	4.9	2.6	2.3
24	83.6	8.6	75.0	41.3	33.7	25.9	7.8	-0.3	8.1	-0.2	8.3	3.1	5.2
25	83.5	10.2	73.3	37.4	35.9	29.9	6.0	0.9	5.1	0.5	4.6	1.6	3.0

Table 5 (cont.): Temporal variation in subcatchment total imperviousness

Sub. #	2015		1999		1989		1981		1971		1959		1949
	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)	+/- (%)	Imp. (%)
26	72.4	63.5	8.9	2.9	6.0	2.4	3.6	0.6	3.0	0.4	2.6	0.1	2.5
27	49.0	14.0	35.0	9.6	25.4	2.5	22.9	1.6	21.3	-0.4	21.7	9.7	12.0
28	72.9	43.1	29.8	26.2	3.6	0.0	3.6	-0.1	3.7	0.4	3.3	0.8	2.5
29	21.9	5.8	16.1	2.9	13.2	2.2	11.0	4.3	6.7	0.1	6.6	3.9	2.7
30	57.5	43.4	14.1	3.1	11.0	5.3	5.7	0.2	5.5	0.9	4.6	0.6	4.0
31	51.8	47.0	4.8	0.4	4.4	0.1	4.3	0.0	4.3	0.9	3.4	0.5	2.9

It is evident from the results that the Black Creek watershed has experienced extensive urbanization over the past several decades. Urban development occurred in the lower subcatchments between the years 1949 and 1971. Minor development in some subcatchments continued between 1971 and 1989, but it was not until after 1989 when the upper subcatchments started to experience extensive urbanization. The results show initial growth starting in the southern region, close to Lake Ontario, and expanding northward over time. For the present conditions, it is shown that 18 of the 31 subcatchments are classified as being more than 50% impervious. The results indicate that the Black Creek watershed is extremely urbanized and will remain relatively the same as it is considered to be fully developed in today's time period.

As discussed in Section 2.2.1, other methods exist for evaluating land use change. Manual digitization was selected for this research. As seen in Table 5, this method can result in minor errors due to human error. The accuracy of manual digitization relies heavily on the user and the quality of photographs used in the analysis. However, for the purpose of this research manual digitization was deemed sufficient as historical aerial photographs were readily available from the TRCA and only a general understanding of land use change over time was required.

5.2 Hydrologic modeling

5.2.1 Historical and current time simulations (without LID measures)

All hydraulic structures and stormwater ponds in the Black Creek watershed were initially investigated in the analysis of historical aerial photographs, and were labelled based on the time period they were implemented. This information is shown in Tables 31 and 32 located in Appendix D. The PCSWMM model was initially run with a 24 hour 25 mm "hotstart" file, for 168 hours (as recommended by the TRCA), to ensure the soil did not start with a dry matrix. A hotstart file is a generic storm event that is simulated and the output is used as the starting point for simulations afterwards. These files provide an initial depth and flow conditions that help to avoid some of the numerical instabilities that can occur when using the dynamic wave routing method. Utilizing the land use results from Section 5.1.3, each time period was simulated using the 4 hour Chicago design storm, for each return period. The only parameters changing between each return period was the total imperviousness of each subcatchment and roughness coefficient for each concrete lined segment of Black Creek, along with the removal of hydraulic structures and stormwater ponds in the corresponding time periods. This allows for hypothetical historical simulations to determine the impact on the outlet hydrograph of Black Creek due to the changing land use and concrete lining of certain channel segments.

As the original model was fully set-up for simulations for the current time period, only the imperviousness of each subcatchment was modified to the values presented in Table 5 due to the original numbers in the model being slightly outdated. The model was initially run with the 24 hour 25 mm hotstart file, for 168 hours. The 4 hour Chicago design storm was then simulated one at a time for each return period, for a duration of 12 hours. These simulation results were used as a base when comparing the various LID scenario simulations presented in Chapter 6.

5.2.2 Results and discussion

Table 6 displays outlet peak flows for various return periods as well as the percent change between each time period. Results indicate large changes in peak flows from 1949 to 1971 and relatively no changes from 1971 to 2015. Figure 12 presents the outlet hydrographs for Black Creek from 1949 to 2015 based on a 2-year event. The initial peak can be contributed to the runoff from the lower subcatchments and the second peak is due to remaining flow from the upstream channel segments. The initial peak flow value of approximately 39 m^3/s in 1949 is shown to significantly increase to approximately 55 m^3/s in 1971 and remain relatively constant afterwards. This is due to the fact that the lower part of the watershed, which is substantially larger in land mass than the upper part of the watershed, was quite developed prior to 1971. The northern subcatchments started to rapidly develop after 1971 but several stormwater ponds were put in place to control the quantity of runoff. Therefore, minimal changes in the outlet hydrographs are seen between 1971 and the current time period. The year 1981 had a slightly higher initial peak than the hydrographs from 1989 to current time and this could be due to the implementation of more stormwater ponds after 1981 that has reduced the initial peak

Table 6: Temporal variation in outlet peak flows

Return period	2015		1999		1989		1981		1971		1959		1949
	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)	+/- (%)	Peak flow (m^3/s)
2	54.3	0	54.3	-3.5	56.2	-0.3	56.4	3.1	54.6	21.5	42.9	8.7	39.2
5	100	0	100	-1.3	102	0.1	101	0	101	20.4	80.1	14.6	68.4
10	124	0	124	-0.6	125	0	125	0	125	10.4	112	11.5	98.8
25	147	0	147	0	147	0	147	2.8	143	3.3	138	7.3	128
50	171	0	171	0	171	0	171	1.8	168	-0.6	169	5.8	159
100	189	0	189	0	189	0	189	2.2	185	-1.6	188	5.0	178

flow. A large difference in the area under each hydrograph is shown between 1949 and the more recent time periods. This area represents the volume of water leaving Black Creek at the outlet. The increasing volume with time is due the extreme amounts of runoff generated by urban development. The year 1981 had the most volume of water leaving the outlet of Black Creek and this can be attributed to the lack of stormwater ponds implemented during this time period.

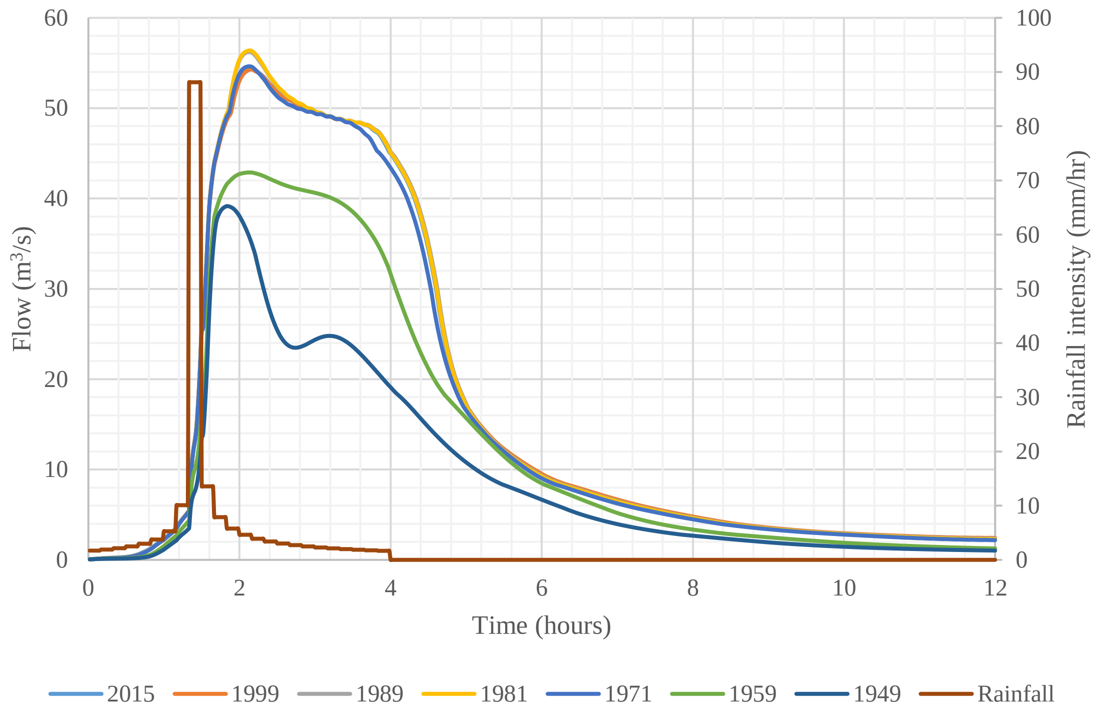


Figure 12: Outlet hydrographs in response to a 2-year event for all time periods

Figure 13 presents the outlet hydrographs based on a 100-year event. The high peak flows and runoff volumes are seen regardless of time period and land use change. This is logical as the 100-year event is an extremely intense rainfall event and the native soil infiltration rates in the watershed is considered to be very low. The double peak flow values are relatively the same for each time period and this is due to the vast amounts of runoff being directed to the upstream sections of Black Creek. However, the positive effect of hydraulic structures is shown in recent time periods approximately 7 hours into the simulation when most of the rainfall volume has left Black Creek. Figures 26, 27, 28, and 29 located in Appendix E present outlet hydrographs based on 5, 10, 25, and 50-year events, respectively.

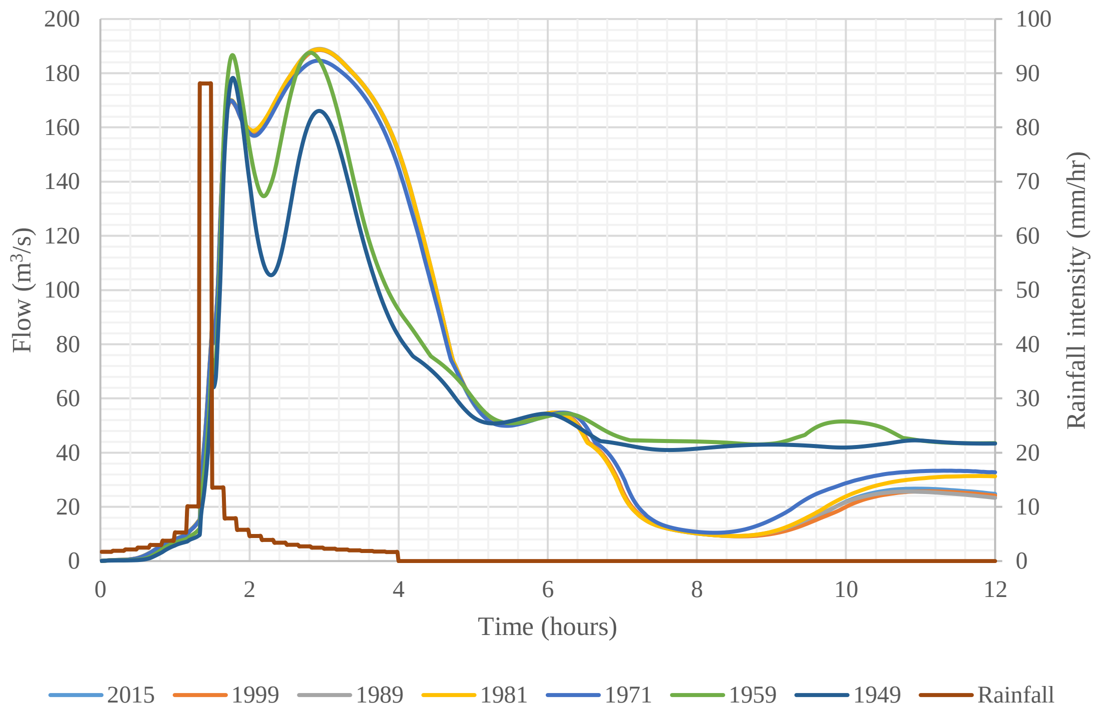


Figure 13: Outlet hydrographs in response to a 100-year event for all time periods

Large amounts of runoff from impervious urban lots have created significantly higher peak flows throughout the watercourse. Channelization has modified natural segments of Black Creek, losing the ability to adapt to these changing conditions. Stormwater ponds have successfully controlled runoff in the upper part of the watershed, but early development in the lower region, prior to SWM practices, has left minimal available land for implementation of large-scale stormwater ponds.

Figure 14 below represents an outlet hydrograph for Black Creek in response to a real storm event that occurred in October, 2015 (Environment Canada 2017). By comparing Figures 12 and 14, the hydrographs vary in shape but the concept of the double peak is still noticed. This suggests the type of design storm selected for this research may not have been the most appropriate option, and that the PCSWMM model requires minor adjustments for more accurate simulations.

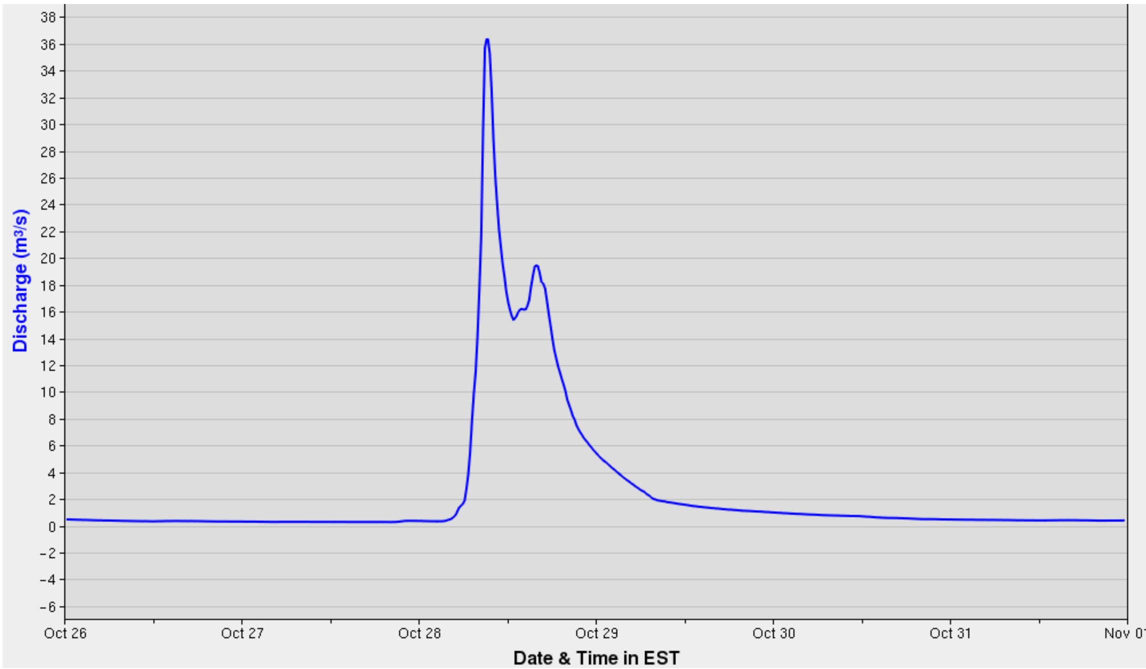


Figure 14: Black Creek outlet hydrograph for 2015 storm event (Environment Canada 2017)

Chapter 6

6 Effectiveness of LID measures to mitigate flood hazard

This chapter addresses objective (2) of the thesis which seeks to evaluate the effectiveness of various LID measures for managing flood hazard in the Black Creek watershed. This chapter provides details on the development of LID scenarios as well as their capabilities at reducing outlet peak flows and subcatchment runoff peak flows. The impacts of modifying LID infiltration parameters to lower values in the PCSWMM model will be investigated. A final cost-benefit analysis will evaluate the suitability of each scenario.

6.1 Development of LID scenarios

In order to develop LID scenarios, a single unit of each LID measure was hypothetically created and used consistently throughout the simulations. These were applied in different numerical combinations to create LID scenarios of various investments. The area and depth of these measures, along with their estimated costs, were developed according to realistic examples from literature (Uda et al. 2013). The values for the layer properties were assigned based on typical values outlined in the SWMM user manual (James et al. 2010), as well as the literature (TRCA and CVC 2010; Zhang and Guo 2015; Chui et al. 2016). The maximum amount of impervious area draining to each LID unit was determined through the TRCA's LID design guidelines (TRCA and CVC 2010). In PCSWMM, these values provide a limit to the volume of stormwater entering each LID unit to ensure proper function. Underdrains were incorporated into select LID measures due to the native soil in the watershed having permeability values lower than 15 mm/hr (Uda et al. 2013).

A summary of the dimensions, layer properties, maximum runoff assigned to each LID measure, and the associated cost of each LID unit are presented in Tables 7 through 13 below. Tables 7 and 8 present details for a single residential driveway and commercial lot permeable pavement unit, respectively. These permeable pavement measures exhibit identical properties and only differ in the area, surface width, and area of runoff treated per unit. These units contain pavement and storage layers with additions of underdrains.

Table 9 presents details for a single rain barrel unit. These units contain no underdrain and are assumed to simply retain a volume of water and overflow when the maximum capacity is reached. Table 10 presents details for a single vegetative swale unit. Table 11 presents details for a single infiltration unit. These units are simply storage layers with additions of an underdrain and a berm. Table 12 presents details for a single bioretention unit. These units contain soil and storage layers, with additions of an underdrain and a berm. Table 13 presents details for a single rain garden unit. These units are simply soil layers and contain a berm.

Table 7: Properties of permeable pavement (residential driveway) unit

Surface		**Pavement**		**Storage**		**Underdrain**	
Berm height (mm)	0	Thickness (mm)	100	Thickness (mm)	300	Drain coefficient (mm/hr)	0.5
Vegetation volume (fraction)	0	Void ratio (voids/solids)	0.15	Void ratio (voids/solids)	0.75	Drain exponent	0.5
Surface roughness (Manning's n)	0.012	Impervious surface (fraction)	0	Seepage rate (mm/hr)	750	Drain offset height (mm)	50
Surface slope (percent)	1	Permeability (mm/hr)	10,000	Clogging factor	0		
		Clogging factor	0				

Area of each unit (m^2) = 50
Surface width per unit (m) = 5
Area of runoff treated per unit (m^2) = 60

COST: $5,000

Table 8: Properties of permeable pavement (commercial lot) unit

Surface		Pavement		Storage		Underdrain	
Berm height (mm)	0	Thickness (mm)	100	Thickness (mm)	300	Drain coefficient (mm/hr)	0.5
Vegetation volume (fraction)	0	Void ratio (voids/solids)	0.15	Void ratio (voids/solids)	0.75	Drain exponent	0.5
Surface roughness (Manning's n)	0.012	Impervious surface (fraction)	0	Seepage rate (mm/hr)	750	Drain offset height (mm)	50
Surface slope (percent)	1	Permeability (mm/hr)	10,000	Clogging factor	0		
		Clogging factor	0				

Area of each unit (m^2) = 1,000
Surface width per unit (m) = 20
Area of runoff treated per unit (m^2) = 1,200

COST: $100,000

Table 9: Properties of rain barrel unit

Storage		Underdrain	
Barrel height (mm)	1,000	Drain coefficient (mm/hr)	0
		Drain exponent	0
		Drain offset height (mm)	0

Area of each unit (m^2) = 0.29
Area of runoff treated per unit (m^2) = 50

COST: $150

Table 10: Properties of vegetative swale unit

Surface	
Berm height (mm)	500
Vegetation volume (fraction)	0.1
Surface roughness (Manning's n)	0.2
Surface slope (percent)	1
Swale side slope (run/rise)	3

Area of each unit (m^2) = 100
Surface width per unit (m) = 3
Percent initially saturated (%) = 20
Area of runoff treated per unit (m^2) = 1,000

COST: $6,500

Table 11: Properties of infiltration trench unit

Surface		Storage		Underdrain	
Berm height (mm)	100	Thickness (mm)	1,600	Drain coefficient (mm/hr)	0.5
Vegetation volume (fraction)	0	Void ratio (voids/solids)	0.75	Drain exponent	0.5
Surface roughness (Manning's n)	0	Seepage rate (mm/hr)	750	Drain offset height (mm)	100
Surface slope (percent)	0	Clogging factor	0		

Area of each unit (m^2) = 102
Area of runoff treated per unit (m^2) = 2,000

COST: $27,575

Table 12: Properties of bioretention cell unit

Surface		Soil		Storage		Underdrain	
Berm height (mm)	100	Thickness (mm)	1,000	Thickness (mm)	680	Drain coefficient (mm/hr)	0.5
Vegetation volume (fraction)	0.1	Porosity (volume fraction)	0.5	Void ratio (voids/solids)	0.75	Drain exponent	0.5
Surface roughness (Manning's n)	0	Field capacity (volume fraction)	0.2	Seepage rate (mm/hr)	750	Drain offset height (mm)	500
Surface slope (percent)	0	Wilting point (volume fraction)	0.1	Clogging factor	0		
		Conductivity (mm/hr)	250				
		Conductivity slope	10				
		Suction head (mm)	100				

Area of each unit (m^2) = 130
Percent initially saturated (%) = 20
Area of runoff treated per unit (m^2) = 2,000

COST: $41,476

Table 13: Properties of rain garden unit

Surface		Soil	
Berm height (mm)	50	Thickness (mm)	100
Vegetation volume (fraction)	0.1	Porosity (volume fraction)	0.5
Surface roughness (Manning's n)	0	Field capacity (volume fraction)	0.2
Surface slope (percent)	0	Wilting point (volume fraction)	0.1
		Conductivity (mm/hr)	250
		Conductivity slope	10
		Suction head (mm)	100

Area of each unit (m^2) = 10
Percent initially saturated (%) = 20
Area of runoff treated per unit (m^2) = 50

COST: $500

All scenarios were developed with consideration of the 2016 operating budget for the City of Toronto. Approximately $23 million was allocated to stormwater management in 2016, with a rise of approximately $1 million projected for the 2017 stormwater management budget (City of Toronto 2016c). To determine the number of units for each LID feature, aerial images of each subcatchment were studied and a decision was made based on the land use. The priority was allocating units of permeable pavement (residential driveways and commercial lots) as these units would be the most expensive, but ideally the most efficient in reducing runoff. This is assumed based on the large area of each unit and the ability of permeable pavement to infiltrate large amounts of runoff. Infiltration trenches and bioretention cells followed, with rain barrels, rain gardens, and vegetative swales used as minor, inexpensive additions. These scenarios invest different amounts of money into various sized subcatchments and locations in the watershed. Large, medium, and small sized subcatchments are classified based on having an area of approximately 350-836 hectares, 100-350 hectares, and 12-100 hectares, respectively. Scenarios 1 and 2 invest LID measures in multiple subcatchments, with the goal of investigating the effects on outlet peak flows of Black Creek. Scenarios 3-8 invest LID measures in individual subcatchments of different sizes, with the goal of investigating the effects on subcatchment runoff peak flows.

6.1.1 Scenario 1

Scenario 1 was developed by distributing a realistic value of approximately $20 million to multiple subcatchments. Priority in allocating the money was given to subcatchments that exhibited very high imperviousness percentage rates and large amounts of runoff. A higher priority was given to the lower subcatchments (3-7) as these were the largest in Black Creek, heavily urbanized, and contained minimal stormwater management features. The upper subcatchments (17, 19-25) were allocated some money due to heavy urbanization, however, this region already contains seven stormwater ponds for controlling quantity, quality, and erosion. Based on this methodology, Scenario 1 comes to a total cost of $20,003,831 and provides a very realistic option for implementing a wide variety of LID measures throughout the watershed. The distribution of LID measures and the investment is shown in Table 14.

Table 14: Summary of Scenario 1

Sub. #	LID (# of units)							Total cost ($)
	RB	BC	RG	IT	RD	CL	VS	
3	1,000	1	50	10	500	3	8	3,344,226
4	1,000	1	50	10	500	5	10	3,557,226
5	1,000	1	50	15	500	10	10	4,195,101
6	1,000	1	50	10	500	3	10	3,357,226
7	1,000	1	50	10	500	3	10	3,357,226
17	50	0	0	2	0	1	1	169,150
19	50	0	0	2	0	2	1	269,150
20	50	0	0	1	0	1	0	135,075
21	50	0	0	2	0	2	1	269,150
22	50	0	0	2	0	1	1	169,150
23	50	0	0	1	0	1	0	135,075
24	50	0	0	3	0	2	1	296,725
25	250	1	0	5	0	5	5	749,351
Total cost: $20,003,831								

6.1.2 Scenario 2

Scenario 2 is by far the most expensive scenario at $50,003,315 and was developed to evaluate the effectiveness of LID measures with a considerably larger budget. Similar to Scenario 1, the largest and most urbanized subcatchments received a higher investment of LID measures than the smaller and less urbanized subcatchments. However, in this scenario all subcatchments in the watershed received some LID investment. This is a hypothetical scenario that is not realistically feasible with the current operating budget, but it provides insight into the effectiveness of various LID measures distributed throughout the watershed. The distribution of LID measures and the investment is shown in Table 15.

Table 15: Summary of Scenario 2

Sub. #	**LID (# of units)**							**Total cost ($)**
	RB	**BC**	**RG**	**IT**	**RD**	**CL**	**VS**	
1	10	0	0	1	10	0	3	98,575
2	500	2	50	5	100	2	5	1,053,327
3	5,000	5	500	10	1,000	3	10	6,848,130
4	5,000	5	500	10	1,000	5	10	7,048,130
5	5,000	5	500	15	1,000	10	15	7,718,505
6	5,000	5	500	10	1,000	3	10	6,848,130
7	5,000	5	500	10	1,000	3	10	6,848,130
8	500	3	50	2	50	1	5	662,078
9	5,000	3	500	5	500	2	10	4,027,303
10	500	3	50	2	50	1	5	662,078
11	500	3	50	2	50	2	5	762,078
12	10	1	0	2	0	3	5	430,626
13	10	0	0	0	0	0	5	34,000
14	100	1	0	2	0	3	5	444,126
15	0	0	0	0	0	0	5	32,500
16	100	1	0	2	0	2	5	344,126
17	100	1	0	2	0	4	3	531,126
18	0	0	0	0	0	2	5	232,500
19	100	2	0	2	0	4	5	585,602
20	10	1	0	2	0	2	5	330,626
21	100	3	0	2	0	2	5	427,078
22	10	2	0	2	0	4	5	572,102
23	100	2	0	2	0	2	3	372,602
24	100	2	0	2	0	3	6	492,102
25	500	3	0	5	0	5	10	902,303
26	100	2	0	3	0	2	5	413,177
27	10	0	0	0	0	0	5	34,000
28	100	2	0	2	0	2	5	385,602
29	100	2	0	2	0	1	5	285,602
30	10	1	10	1	10	0	2	138,551
31	250	0	50	0	50	1	4	438,500
Total cost: $50,003,315								

6.1.3 Scenario 3

Scenarios 3-8 were developed in order to better understand the relationship between the effectiveness of LID measures in different sized subcatchments. Scenario 3 evaluates the effectiveness of LID measures in a small subcatchment with a size of 61.4 hectares. This scenario is based on a reasonable investment of approximately $10 million, a value well within the current operating budget.

6.1.4 Scenario 4

Scenario 4 was developed on the same principle as Scenario 3, except the number of LID measures and the total cost have been doubled in the same subcatchment. This was challenging to develop a realistic scenario due to the high level of investment and low opportunity for LID implementation. All parking lots were hypothetically transformed into permeable lots, which in the real world may not actually occur. This may not be the most realistic scenario, however, it provides a means of comparing an investment of $20 million in a small, medium, and large sized subcatchments.

6.1.5 Scenario 5

Scenario 5 investigates the effectiveness of LID measures in a medium sized subcatchment with a size of 231.1 hectares. This scenario looks at an investment of approximately $10 million, similar to Scenario 3.

6.1.6 Scenario 6

Scenario 6 was developed on the same principle as Scenario 5, except the number of LID measures and the total cost have been doubled in the same subcatchment (similar to Scenario 4).

6.1.7 Scenario 7

Scenario 7 investigates the effectiveness of LID measures in the largest subcatchment in the Black Creek watershed with a size of 835.3 hectares. This scenario looks at an investment of approximately $10 million, similar to Scenarios 3 and 5.

6.1.8 Scenario 8

Scenario 8 was developed on the same principle as Scenario 7, except the number of LID measures and the total cost have been doubled in the same subcatchment (similar to Scenarios 4 and 6).

6.1.9 Results and discussion

Scenarios 1-8 were simulated individually using the design storms described in Section 4.2.4, where Scenarios 1 and 2 invest LID measures throughout the watershed and Scenarios 3-8 invest LID measures in individual subcatchments. Table 16 provides the simulation results of Scenarios 1 and 2. Results show very minor changes to outlet peak flows of Black Creek, demonstrating that investments of $20 and $50 million throughout the watershed are not enough to significantly reduce outlet peak flows. Specifically, a reduction of 2.5% and 6.3% for a 2-year event are not enough to validate the large investment of LID measures for Scenarios 1 and 2, respectively. Reductions for 5-year to 100-year events are even lower as a result of extreme amounts of rainfall and runoff. It is shown that distributions of $20 and $50 million throughout the watershed provide minimal reductions in the outlet peak flows of Black Creek. This is due to the vast size of the lower subcatchments, the level of imperviousness, and the low investment of LID measures in the larger subcatchments. These results suggest that individual subcatchments must be examined, with higher levels of investment, in order to fully evaluate the effectiveness of LID measures.

Table 16: Change in outlet peak flows for Scenarios 1 and 2

Return period	No LID Peak flow (m^3/s)	Scenario 1 Peak flow (m^3/s)	Change (%)	No LID Peak flow (m^3/s)	Scenario 2 Peak flow (m^3/s)	Change (%)
2	54.3	52.9	2.5	54.3	51.1	6.3
5	100.4	98.9	1.5	100.4	96.8	3.7
10	124.4	123.3	0.9	124.4	121.9	2.0
25	147.0	145.1	1.3	147.0	143.4	2.5
50	170.9	169.4	0.9	170.9	167.9	1.8
100	188.6	186.5	1.1	188.6	184.4	2.2

Table 17 provides an overview of Scenarios 3-8. The location, size of the subcatchment receiving LID measures, the distribution of LID measures, and the total cost invested is included.

Table 17: Summary of Scenarios 3 to 8

Scenario #	Sub. #	Size of sub.	Area (ha)	LID (# of units)							Total cost ($)
				RB	BC	RG	IT	RD	CL	VS	
3	19	Small	61.4	3,250	30	0	57	0	62	77	10,004,055
4	19	Small	61.4	6,500	60	0	114	0	124	154	20,008,110
5	10	Medium	231.1	3,000	15	0	40	0	75	50	10,000,140
6	10	Medium	231.1	6,000	30	0	80	0	150	100	20,000,280
7	5	Large	835.3	3,000	15	0	40	0	75	50	10,000,140
8	5	Large	835.3	6,000	30	0	80	0	150	100	20,000,280

Figures 15, 16, and 17 present the runoff hydrographs for a 2-year event with and without an investment of LID measures for a small, medium, and large sized subcatchment, respectively (i.e., Scenarios 3 to 8). Runoff hydrographs for the 5, 10, 25, 50, and 100-year events are shown in Figures 30-44 in Appendix F. Figures 15 and 30-34 depict simulations from Scenarios 3 and 4, Figures 16 and 35-39 depict simulations from Scenarios 5 and 6, and Figures 17 and 40-44 depict simulations from Scenarios 7 and 8. The effectiveness of LID measures at peak runoff reduction in a small sized subcatchment is evident in Figures 15 and 30-34. From Figure 15, the peak has been reduced significantly from its original value of approximately 12 m^3/s to approximately 3 m^3/s. From Figures 16, 17, and 35-44 the peak reduction is less noticeable with the same investments of $10 million and $20 million in medium and large sized subcatchments, respectively. A moderate impact in subcatchment 10 is noticed with increasing levels of investment, however, peak flow values remain relatively unchanged in subcatchment 5. This demonstrates the limited abilities of LID measures to reduce flood hazard on a large-scale.

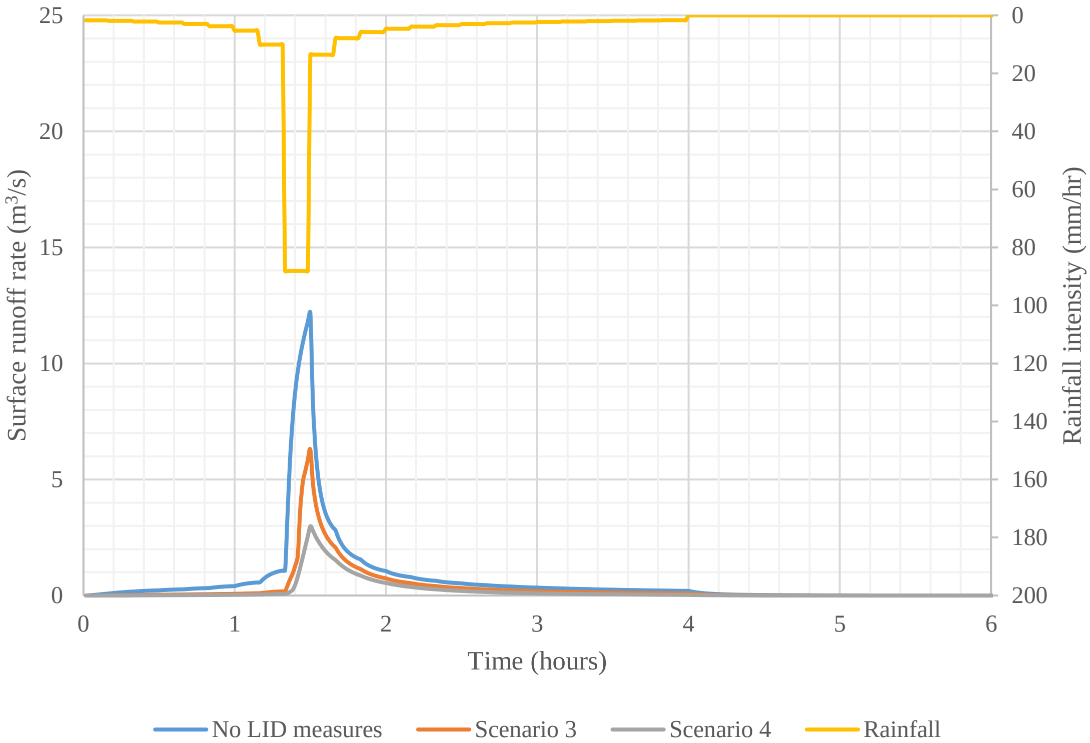


Figure 15: Runoff hydrographs for Scenarios 3 and 4 (2-year event)

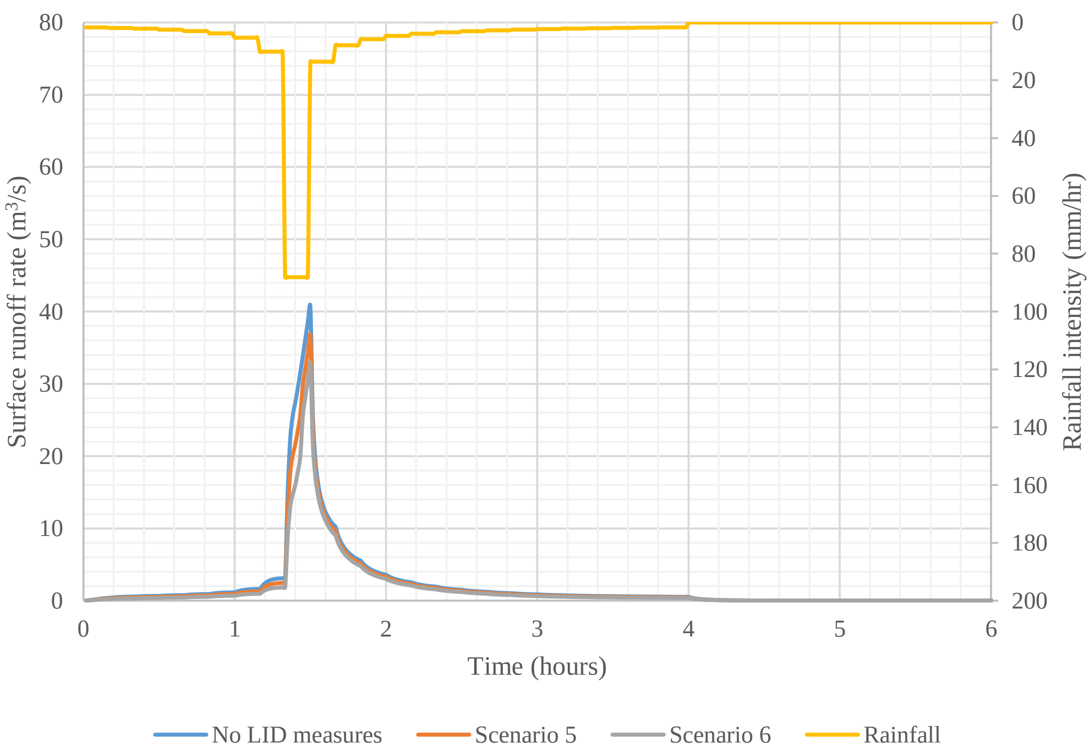


Figure 16: Runoff hydrographs for Scenarios 5 and 6 (2-year event)

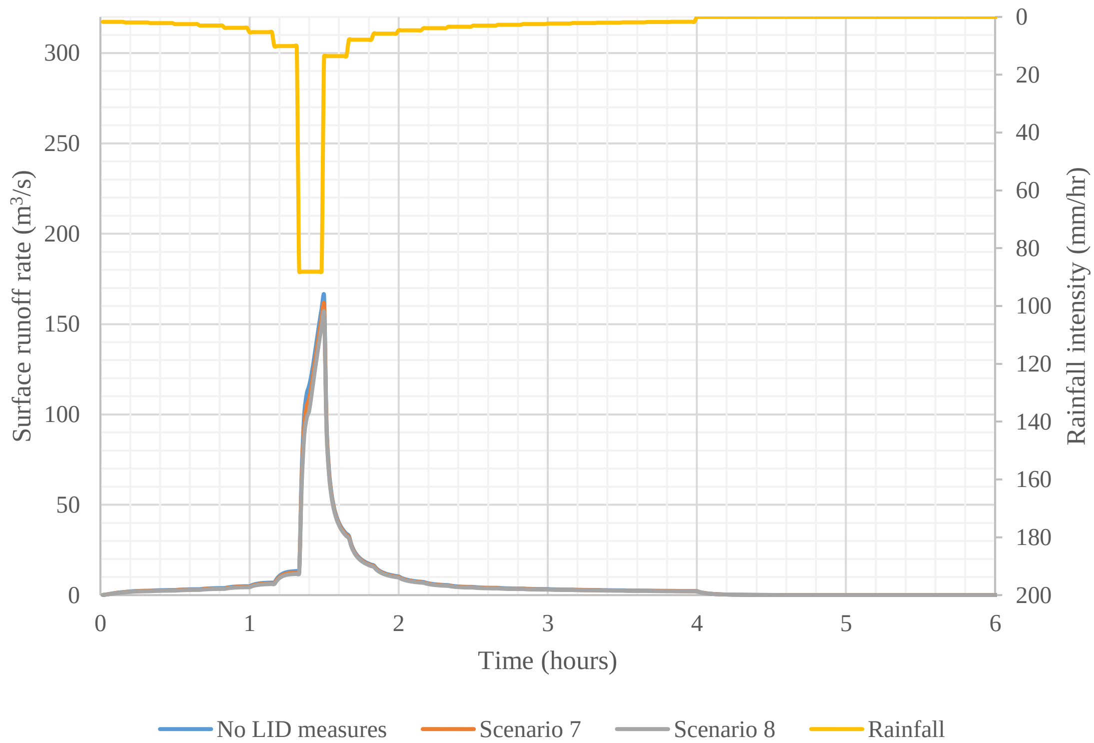


Figure 17: Runoff hydrographs for Scenarios 7 and 8 (2-year event)

A comparison of peak runoff reduction from Scenarios 3, 5, and 7 is presented in Figure 18. Similarly, a comparison of peak runoff reduction from Scenarios 4, 6, and 8 is presented in Figure 19. These plots compare the effectiveness of LID measures in being able to reduce the peak runoff in different sized subcatchments, for each return period.

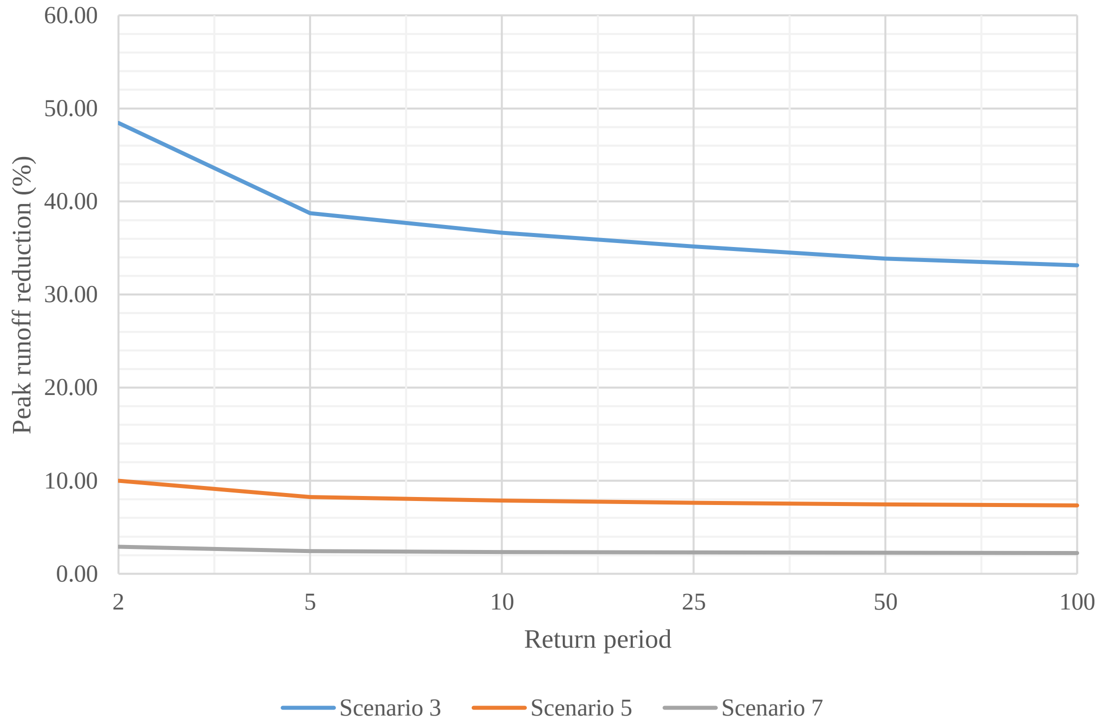


Figure 18: Comparison of $10 million investment in small (Scenario 3), medium (Scenario 5) and large (Scenario 7) subcatchments

Figures 18 and 19 demonstrate a consistent investment of $10 and $20 million, respectively, in a small, medium, and large sized subcatchment. From Figure 18, the effectiveness of LID measures in a small subcatchment is immediately noticed. For a 2-year event, the peak runoff reduction is approximately 48%, 10%, and 3% for a small, medium, and large sized subcatchment, respectively. From the plot it is apparent that LID measures are most effective for lower intensity storms due to the decrease in slope from a 2-year to 100-year event. This is expected as higher return periods contain much larger intensities and volumes of water that may not have enough time to fully infiltrate into the LID measure. The effect is largely noticed in a small sized subcatchment, less for the medium sized subcatchment, and negligible in a large sized subcatchment. The results are understandable due to the vast size of the large subcatchments and the limited capabilities of the small-scale LID measures.

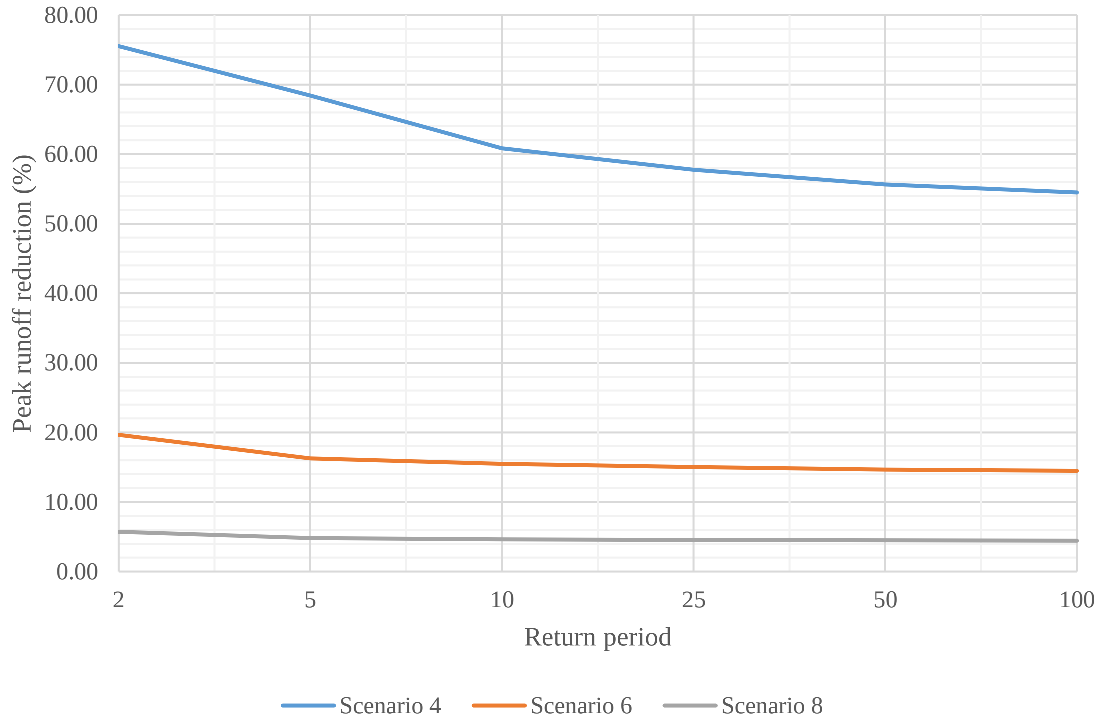


Figure 19: Comparison of $20 million investment in small (Scenario 4), medium (Scenario 6), and large (Scenario 8) subcatchments

Similarly, from Figure 19, the peak runoff reduction is approximately 76%, 20%, and 6% for a small, medium, and large sized subcatchment, respectively. Similar trends are noticed as from Figure 18, however, it is interesting to note the elongated steep slope from a 2-year to 10-year event for a small sized subcatchment. With the considerable investment of $20 million, the peak runoff reduction during a 10-year event is more significant than with half the investment. This is due to the large number of LID measures being implemented in the subcatchment which are able to handle more precipitation from even a 10-year event. Effects in a medium and large sized subcatchments are very similar to before, being the most effective in a 2-year event, and slightly decreasing with increasing return period.

The maximum amount of runoff directed to each LID was a limiting factor. For example, the 76% reduction of the peak runoff in a small sized subcatchment, for a 2-year event, could potentially be 100% as detailed results show that the infiltration trenches, commercial parking lots, and bioretention cells are capable of handling more runoff.

Tables 18-23 present the distribution of receiving runoff (total inflow) in each LID measure for Scenarios 3-8, respectively. These tables outline the amount of runoff infiltrating, overflowing, and being sent through underdrains as well as the initial storage, final storage, and continuity error of each LID measure. The underdrains implemented in bioretention cells, infiltration trenches, and commercial parking lots are barely being filled with water due to the infiltration of the receiving runoff. However, to be conservative, it can be argued that the amount of runoff being sent to each LID measure is an acceptable amount. The continuity errors seen in Tables 18-23 present the difference between the inflow and outflow of water from each LID measure. All errors are 0% or close to 0% which confirms that large amounts of water have not been lost in the system.

Table 18: LID inflow distribution for Scenario 3

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
19	VS	193.34	3.66	189.74	0	0	0	-0.03
19	RB	2896.49	0	1896.49	0	0	1000	0
19	IT	354.9	354.85	0	0.05	0	0	0
19	CL	49.91	49.91	0	0	0	0	0
19	BC	283.62	322.2	0	0	238.29	200	-0.06

Table 19: LID inflow distribution for Scenario 4

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
19	VS	70.4	2.56	67.89	0	0	0	-0.06
19	RB	365.31	0	0	0	0	365.31	0
19	IT	198.79	198.79	0	0	0	0	0
19	CL	44.76	44.76	0	0	0	0	0
19	BC	191.85	230.32	0	0	238.29	200	-0.04

Table 20: LID inflow distribution for Scenario 5

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
10	VS	166.73	3.92	162.87	0	0	0	-0.04
10	RB	2430.09	0	1430.09	0	0	1000	0
10	IT	304.61	304.61	0	0	0	0	0
10	CL	46.2	46.2	0	0	0	0	0
10	BC	237.38	276.04	0	0	238.29	200	-0.08

Table 21: LID inflow distribution for Scenario 6

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
10	VS	166.5	3.92	162.65	0	0	0	-0.04
10	RB	2373.55	0	1373.55	0	0	1000	0
10	IT	297.33	297.33	0	0	0	0	0
10	CL	45.89	45.89	0	0	0	0	0
10	BC	238.73	277.04	0	0	238.29	200	-0.01

Table 22: LID inflow distribution for Scenario 7

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
5	VS	196.61	5.76	190.92	0	0	0	-0.04
5	RB	2909.57	0	1909.57	0	0	1000	0
5	IT	360.52	360.35	0	0.17	0	0	0
5	CL	49.99	49.99	0	0	0	0	0
5	BC	286.56	324.95	0	0	238.29	200	-0.02

Table 23: LID inflow distribution for Scenario 8

Sub.	LID	Total inflow (mm)	Infiltration loss (mm)	Surface outflow (mm)	Drain outflow (mm)	Initial storage (mm)	Final storage (mm)	Continuity error (%)
5	VS	193.99	5.73	188.33	0	0	0	-0.04
5	RB	2943.04	0	1943.04	0	0	1000	0
5	IT	365.39	365.19	0	0.19	0	0	0
5	CL	49.67	49.67	0	0	0	0	0
5	BC	286.03	324.39	0	0	238.29	200	-0.01

6.2 Modified LID parameters

The parameters chosen for layer properties of the LID units were selected on the higher range of typical values in order to maximize infiltration rates in the simulations. However, these values vary considerably in the real world due to inconsistencies in the design and construction of LID measures. It is therefore important to investigate their effectiveness with lower infiltration values, as seen in literature. For applicable LID measures, pavement permeability rates were changed from 10,000 mm/hr to 500 mm/hr, soil conductivity rates were changed from 250 mm/hr to 25 mm/hr, and storage seepage rates were changed from 750 mm/hr to 250 mm/hr. These changes represent a much lower capability of LID measures to infiltrate stormwater runoff, making them less effective. All other values remained constant. Figures 20 and 21 demonstrate the peak runoff reduction percentages with the modified LID parameters. Scenarios labelled with a (2) represent scenarios with the modified LID parameters.

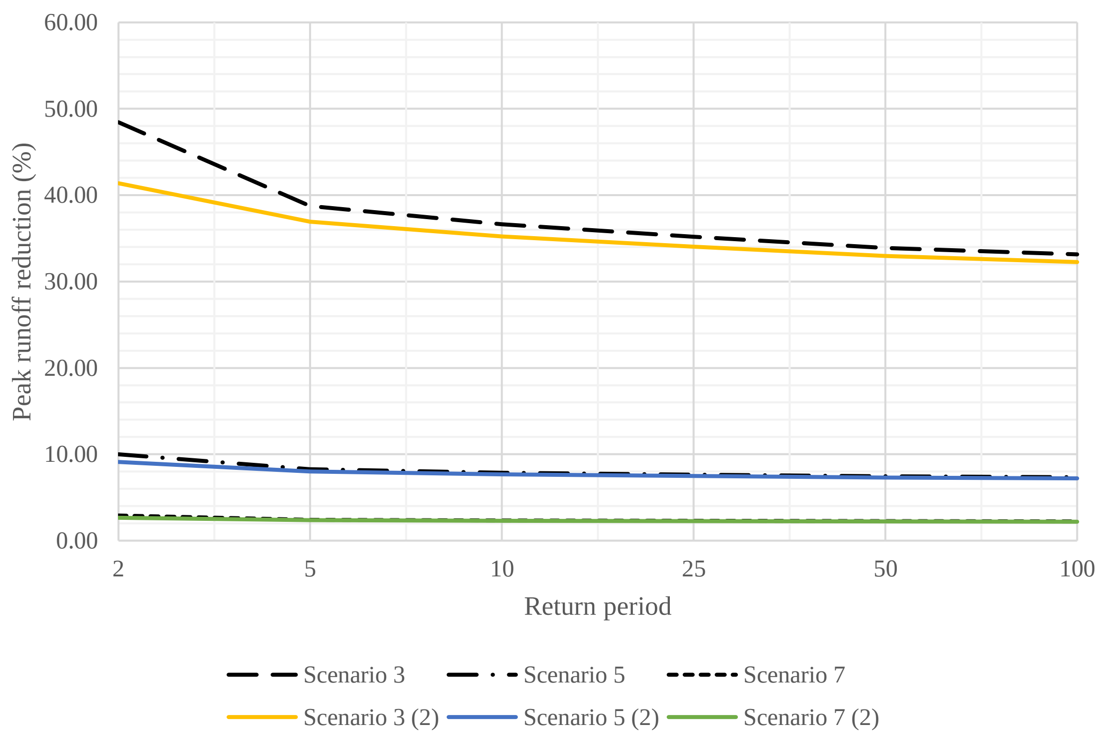


Figure 20: Comparison of $10 million investment in small (Scenario 3), medium (Scenario 5) and large (Scenario 7) subcatchments with modified parameters

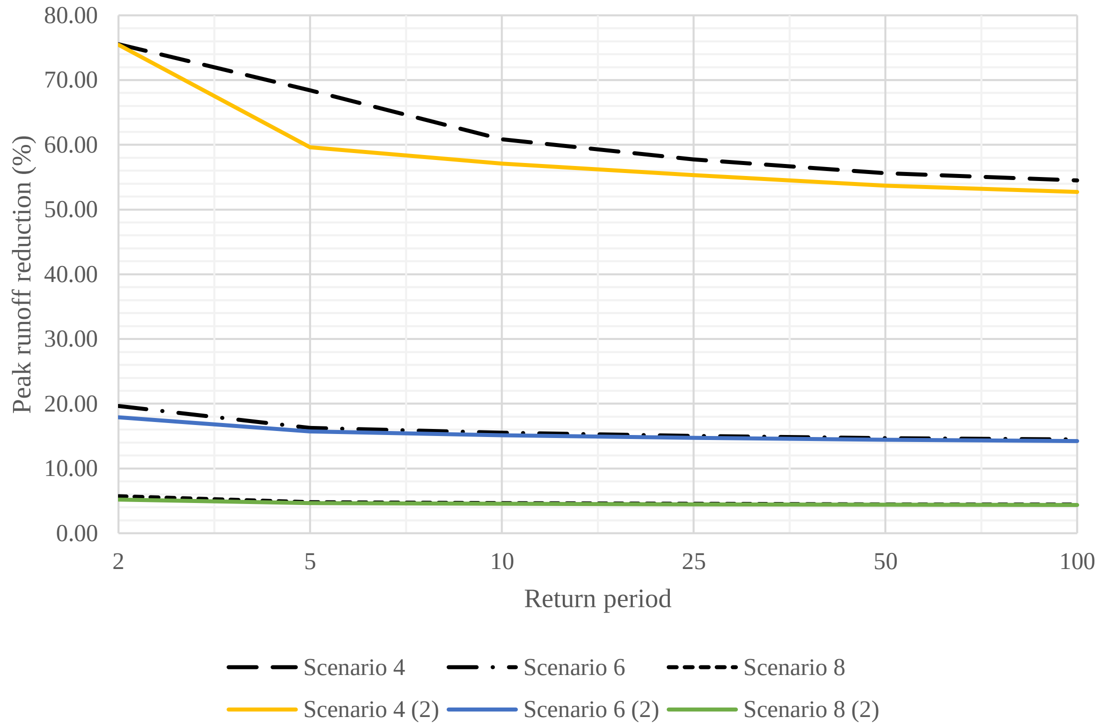


Figure 21: Comparison of $20 million investment in small (Scenario 4), medium (Scenario 6) and large (Scenario 8) subcatchments with modified parameters

It is shown from these figures that modifications to the infiltration rates of the layers had an effect, but very minimal. This is likely due to the fact that only a certain percentage of runoff is being allocated to each LID measure, therefore not maximizing its abilities. From Figure 20, the modified LID parameters decreased the peak runoff reduction of Scenario 3 from approximately 48% to 42% for a 2-year event. All other return periods and scenarios demonstrate a difference of approximately 1% or less and is considered to have a minimal effect. From Figure 21, the modified LID parameters had no effect on the peak runoff reduction of Scenario 4 for a 2-year event and this is likely due to the LID measures being able to infiltrate all runoff being received from the original LID parameters. A more noticeable effect is seen for a 5-year event as the peak runoff reduction has decreased from approximately 69% to 60%. All other return periods and scenarios demonstrate a difference of approximately 1% or less and is considered to have a minimal effect.

6.3 Cost-benefit analysis

This section discusses the cost-benefit analysis of implementing LID measures in different sized subcatchments in the Black Creek watershed. The expected return in value for various levels of investment provides great insight and will ensure the best decision-making and proper allocation of money. Figure 22 illustrates the expected peak runoff reduction for different levels of capital costs, where each line represents a small, medium, and large sized subcatchment. The plot demonstrates a significant return in value for investment of LID measures in small sized subcatchments. Medium and large sized subcatchments provide much lower returns in value and does not significantly change with higher levels of investment. From the plot, it is shown that an increasing investment in small sized subcatchments becomes more significant in comparison to medium and large sized subcatchments. The benefit, however, is largely based on the size of the subcatchment and does not consider the area in medium and large sized subcatchments that would not require investment of LID measures.

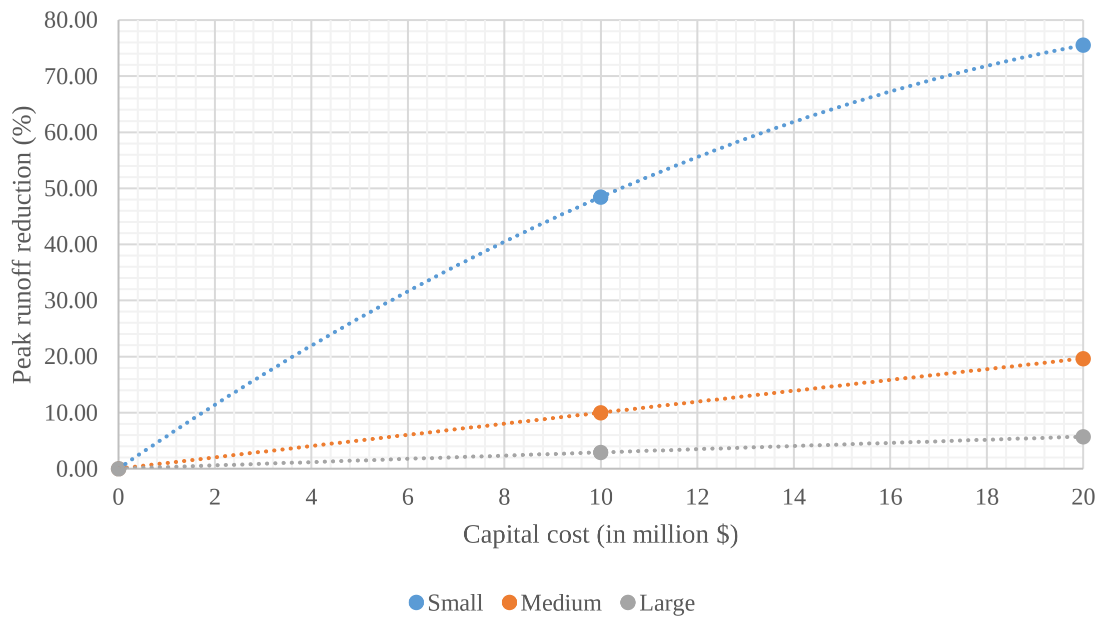


Figure 22: Peak runoff reduction for different levels of investment

Figure 22, which is based on the simulation results, provides an understanding of the effectiveness of LID measures in different sized subcatchments. LID measures are fully capable of reducing peak flows and flood hazard in small sized subcatchments, providing value in the level of investment. Incorporating LID measures on a large scale will not have the same effects and will not provide sufficient value in the investment. However, medium sized subcatchments, as classified in this thesis, have the potential to have moderate peak runoff reduction and does provide some return in the investment. The same principles can be applied to other watersheds in Canada that exhibit similar characteristics. This information greatly assists with flood hazard management as it allows LID measures to be implemented at an appropriate scale. This ensures LID measures are being maximized to their full capability and therefore decreasing urban flood hazard.

6.4 Recommendations for LID investment in Black Creek

Simulations were conducted for the Black Creek watershed that investigated the effects of LID measures on outlet peak flows and subcatchment runoff peak flows. From the results presented in this chapter, the following recommendations are presented:

1. LID measures should be considered for placement in subcatchments that have very high imperviousness percentages and are creating large amounts of runoff to Black Creek.
2. LID measures should be considered largely in the southern region of the watershed due to the minimal amount of SWM in place and heavy residential land use that currently exists.
3. LID measures can be considered in the northern region, however, the effect will be minimal or nearly negligible due to several stormwater management ponds currently in place for controlling runoff.
4. Since many of the subcatchments in the southern region are considered to be relatively large in size, implementation of LID measures should be prioritized in areas of the subcatchment that are the most developed.

Chapter 7

7 Conclusions and recommendations

This thesis provides a contribution to practitioners and decision makers for improving flood hazard management in urban environments. This thesis develops a better understanding of the relationship between urbanization and flood hazard, and evaluates the effectiveness of various LID measures to reduce this hazard in urbanized environments. This chapter summarizes and discusses the main findings of the thesis, provides recommendations for future research, and discusses the applicability of this research to other situations.

7.1 Conclusions

Urbanization has had a significant impact on the environment. Changing land use and channelization over time has created unnatural settings, limiting the ability of the environment and local channels to handle increased rainfall and runoff. Flood hazard is now drastically higher in comparison to several decades ago when the environment was able to naturally handle precipitation events. LID measures have the ability to retrofit into large urban cities that have minimal room for large SWM such as ponds and wetlands. In combination with large SWM or multiple LID measures, they have the ability to greatly reduce flood hazard in urban environments. LID measures have been shown in research to be capable of reducing flood hazard through peak flow reduction, reduction in volume of water, and delays in times to peak.

Many tools have been developed that can be utilized to improve flood hazard management in urban environments. Spatial analysis computer programs are effective at evaluating and quantifying land use change over time from aerial photographs and satellite imagery. This method is valuable for land-use planning and improving flood hazard management by predicting future urban growth. Computational models have also become an efficient and significant tool for reducing flood hazard. These models aim to mimic the natural environment in order to predict the response to storm events of varying duration, magnitude, and intensity. Hydrologic models simulate the distribution of rainfall through different processes such as infiltration, evapotranspiration, runoff, and percolation. Hydraulic models simulate the conveyance of flow through artificial pipes or waterways by taking into consideration fluid mechanics and physics.

This thesis used the Black Creek watershed, located in southern Ontario, to investigate the effects of urbanization on increased flood hazard and evaluated various LID measures in reducing this hazard. A land use analysis from historical aerial photographs demonstrated significant land use change from 1949-1971 and 1989-2015. Historical hydrologic simulations from 1949-2015 have demonstrated the significant change in peak flows and volumes of water exiting Black Creek. Current time hydrological simulations of various LID scenarios have provided great insight in the capabilities of LID measures at reducing peak runoff values and thus flood hazard. A cost benefit analysis has shown the value in investing a certain amount of money into LID measures, dependent on subcatchment size, and the expected return in peak runoff reduction.

Proper allocation of LID measures in the Black Creek watershed is critical in order to receive appropriate value for the investment. LID measures were shown to have a much greater effect in smaller sized subcatchments in comparison to medium and large sized subcatchments. This is due to the vast size of the subcatchments considered ‘medium’ and ‘large’ and the small scale abilities of LID measures.

The following main conclusions can be drawn from this thesis:

1. Urban development from 1949 to 2015 has considerably altered the natural landscape of the greater Toronto area into a densely urbanized and impervious region. Flood hazard in Black Creek has increased due to the rapid urban development, channelization of the watercourse, and climate change.
2. Stormwater ponds implemented in recent decades in the northern region of the watershed have controlled runoff rates to reduce flood hazard downstream. The southern region of the watershed lacks stormwater management and contains minimal available land for potential implementation of large SWM measures.
3. Through hydrologic simulations, LID measures have been shown to greatly reduce peak runoff values in small sized subcatchments, moderately in medium sized subcatchments, and negligibly in large sized subcatchments.

4. An investment of $10 million and $20 million in a small sized subcatchment has the potential to reduce the peak runoff by approximately 48% and 76% for a 2-year event, respectively, as well as 32% and 54% for a 100-year event (see Figures 18 and 19).
5. An increase in infiltration rates of LID measures has a minor impact in small sized subcatchments but a negligible effect in medium and large sized subcatchments (see Figures 20 and 21).
6. A much greater return in value for investments of LID measures can be expected in small subcatchments than in medium and large sized subcatchments (see Figure 22).
7. LID measures should be prioritized in the southern region of the Black Creek watershed where development, runoff, and flood hazard are at their highest. Some areas in the northern region that are still susceptible to urban flooding due to hydraulic issues could utilize placement of LID measures, however, several SWM ponds are in place for controlling large amounts of runoff.

7.2 Recommendations for future research

The following recommendations are suggested for future research in this topic:

1. Refine the model of Black Creek to incorporate more accurate flow length values for each subcatchment. Discretizing larger subcatchments into multiple smaller subcatchments is one solution. This will decrease the likelihood of sharp-peaked hydrographs and will provide better insight on the effects of LID measures at also reducing the times to peak.
2. Modify the method used to implement LID measures in PCSWMM. Create individual catchments for each LID measure and evaluate their effectiveness at reducing flood hazard in a non-lumped approach. Allow runoff from each LID measure to flow directly to another LID measure instead of directly to the subcatchment outlet. This could promote higher reductions in runoff volume and therefore lower flood hazard in Black Creek.

3. Utilize the approach in this thesis and apply it to other urban watersheds in Canada. This will allow for a comparison of the effectiveness of LID measures at reducing flood hazard in other cities.
4. Simulate the effect of implementing various SWM/LID measures historically (1949-1999), and evaluate their effects on changes in flows today.

7.3 Applicability of results to other situations

The research in this thesis was specific to the conditions of the Black Creek watershed. Conditions such as city structure, topography, climate, and degree of urbanization can vary considerably from one city to another across the world. This thesis used a general case study site and demonstrated that historic land use change and channel modifications have greatly impacted flood hazard. While the degree to which this is impacted may vary for different locations, the concept remains the same. The introduction of more impervious surfaces, infrastructure, drainage systems, and modifications to natural waterways have altered the ability of the environment to naturally distribute rainfall into the hydrologic cycle. As discussed in Section 2.1, hydrologic processes such as infiltration and evapotranspiration decrease, while runoff significantly increases. As a result, urbanization has increased flood hazard within many cities across the world.

The development of the LID measures and LID scenarios in this thesis were also specific to the Black Creek watershed and southern Ontario. LID specifications such as cost, parameter values, regulations, and design may vary for different locations. This research demonstrates that LID measures are effective at reducing peak flows and volumes of water in an urban environment and are most efficient on a smaller scale (as demonstrated in this research). While the degree of peak flow and volume of water reduction may vary, it is evident that LID measures can be a suitable alternative for reducing urban flood hazard.

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Appendices

The following appendices listed in this section provides supplementary information, such as figures, tables, and equations, not included in the main body of the thesis.

Appendix A: Background information on PCSWMM

PCSWMM is a fully dynamic rainfall-runoff model that is capable of simulating hydrologic, hydraulic, and water quality components (James et al. 2010). It is considered to be a hydrologic and hydraulic modeling program. Single event and long-term (continuous) simulation options are available in the model. PCSWMM computes runoff quantity and quality from urban or rural areas. The runoff component of the model is based on multiple subcatchment areas where the runoff is generated from precipitation and snowmelt. The routing component of the model transports the runoff overland and underground through a combination of channels, pipes, pumps, and storage devices.

Basic parameters for creating subcatchments in PCSWMM include imperviousness, slope, Manning's roughness coefficient, surface area, outlet point, assigned rain gage, as well as others. Precipitation is input into the model through rain gages which can be a user-defined time series collected from real-world data, or synthetic design storm events created based on user-defined parameters such as rainfall data type, time interval, and type of distribution (Chicago, SCS, AES, Huff). Parameters such as temperature, wind speed, evaporation can be input as a constant value or user-defined time series if the snowmelt module is desired. Snowmelt is modeled using Anderson's NWS (1973) temperature index-method and is based on two different melt situations, with and without rainfall. With rainfall, the snowmelt equation is expressed as:

$$SMELT = (TA - 32)\times(0.00167 + SGAMMA\times UADJ + 0.007\times PREC) + 8.5\times UADJ\times(EA - 0.18)\ , \qquad (10)$$

where *SMELT* is the melt rate (in./hr), *TA* is the air temperature (°F), *SGAMMA* is 7.5**GAMMA* (in. Hg/°F), *UADJ* is a wind speed function (in./in. Hg-hr), *PREC* is the rainfall intensity (in./hr), and *EA* is the saturation vapor pressure at air temperature (in. Hg). Without rainfall, the snowmelt equation is expressed as:

$$SMELT = DHM \times (TA - TBASE) \,, \tag{11}$$

where *SMELT* is the melt rate (in./hr), *DHM* is a melt factor (in./hr-°F), *TA* is the air temperature (°F), and *TBASE* is the base melt temperature (°F).

Infiltration can be modeled using methods such as Horton, Green-Ampt, and SCS Curve Number. Horton's equation in PCSWMM is expressed as:

$$f_p = f_\infty + (f_o - f_\infty)e^{-\alpha t} \,, \tag{12}$$

where f_p is the infiltration capacity into the soil [LT^{-1}], f_∞ is the minimum or ultimate value of f_p [LT^{-1}], f_o is the maximum or initial value of f_p [LT^{-1}], α is a decay coefficient [T^{-1}], and t is the time from the beginning of the storm [T].

Overland flow is generated by approximating subareas as non-linear reservoirs. The non-linear reservoir equation is created by combining the continuity equation with Manning's equation. The depth, *d*, can be solved at each time step from the non-linear differential equation, expressed as:

$$\frac{dd}{dt} = i^* - \frac{1.49 \times W}{A \times n}(d - d_p)^{5/3} S^{1/2} \,, \tag{13}$$

where *d* is the water depth (ft), t is time (sec), i^* is the rainfall excess, *W* is the subcatchment width (ft), *A* is the surface area of the subcatchment (ft^2), *n* is Manning's roughness coefficient, d_p is the depth of the depression storage, and *S* is the subcatchment slope (ft/ft).

Routing through drainage systems is done by steady state, kinematic wave, and dynamic wave approaches. Dynamic wave routing for sewers and open channels is done by combining the momentum and continuity equations into a single equation that is solved along each link at every time step. This equation is expressed as:

$$\frac{\partial Q}{\partial t} + gAS_f - 2V\frac{\partial A}{\partial t} - V^2\frac{\partial A}{\partial x} + gA\frac{\partial H}{\partial x} = 0\,, \tag{14}$$

where Q is the discharge along the conduit, t is time, g is the universal gravity constant, A is the cross-sectional area of flow, S_f is the friction slope, V is the velocity in the conduit, and H is the hydraulic head.

Appendix B: Distribution of land use from 1949-2015

Table 24: Distribution of land use by subcatchment in 2015

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	50.2	20.3	28.7	0.8	28.4
2	17.7	59.9	9.1	13.2	0.0	50.2
3	15.1	75.8	7.4	1.7	0.0	56.2
4	28.1	58.3	6.2	7.4	0.0	58.9
5	27.1	56.7	8.1	8.1	0.0	57.2
6	20.3	61.6	3.0	15.2	0.0	53.4
7	18.1	65.1	5.2	11.5	0.2	53.3
8	11.1	56.7	14.3	17.8	0.2	42.2
9	5.3	61.3	17.8	15.0	0.7	39.2
10	13.4	64.3	3.5	18.8	0.0	48.5
11	8.6	49.7	23.0	17.9	0.7	36.2
12	86.7	-	4.3	9.0	0.0	82.6
13	15.4	-	63.2	20.5	0.9	15.7
14	38.8	-	3.1	58.0	0.1	38.0
15	27.2	-	2.0	70.4	0.4	27.2
16	47.5	-	9.5	42.6	0.4	46.1
17	92.3	-	2.1	3.2	2.4	87.8
18	51.2	-	0.2	48.5	0.0	49.6
19	70.0	-	0.7	28.5	0.8	67.1
20	86.2	-	4.2	8.6	0.9	82.1
21	85.3	-	3.7	10.9	0.1	81.3
22	83.2	-	2.3	14.5	0.0	79.4
23	42.4	-	0.0	55.7	1.9	41.4
24	87.8	-	2.3	9.8	0.0	83.6
25	87.6	-	1.6	10.3	0.5	83.5
26	75.7	-	1.6	20.8	1.9	72.4
27	50.6	-	2.7	46.7	0.0	49.0
28	76.3	-	0.8	21.4	1.5	72.9
29	21.5	-	3.0	73.4	2.1	21.9
30	7.4	91.6	0.0	0.9	0.0	57.5
31	11.8	73.2	1.4	11.8	1.8	51.8

Table 25: Distribution of land use by subcatchment in 1999

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	53.9	20.3	25.1	0.6	30.3
2	21.6	59.9	4.0	14.5	0.0	53.8
3	13.4	75.8	6.8	4.0	0.0	54.6
4	33.9	50.0	5.0	11.1	0.0	60.0
5	26.3	56.7	6.1	11.0	0.0	56.4
6	21.8	61.6	0.6	16.1	0.0	54.9
7	22.3	60.0	3.5	14.2	0.0	54.5
8	13.3	56.7	10.1	20.0	0.0	44.3
9	8.1	61.3	13.2	17.4	0.0	41.9
10	18.8	64.3	1.9	15.0	0.0	53.5
11	10.2	45.0	13.5	31.0	0.3	35.2
12	90.1	-	0.0	9.9	0.0	85.8
13	15.2	-	42.4	41.3	1.1	15.7
14	27.6	-	1.2	71.1	0.1	27.7
15	17.2	-	0.3	82.1	0.4	18.0
16	21.5	-	4.8	73.5	0.2	22.0
17	58.3	-	0.0	39.5	2.2	56.1
18	55.6	-	0.0	44.2	0.2	53.7
19	31.5	-	0.0	66.6	1.9	31.2
20	87.2	-	3.8	7.9	1.1	83.0
21	86.9	-	0.0	13.1	0.0	82.8
22	85.6	-	0.0	14.4	0.0	81.6
23	16.6	-	10.0	73.1	0.3	17.3
24	78.5	-	0.0	21.5	0.0	75.0
25	76.7	-	0.5	22.4	0.3	73.3
26	7.5	-	7.8	83.7	1.0	8.9
27	35.5	-	1.7	62.8	0.0	35.0
28	29.9	-	2.2	68.0	0.0	29.8
29	15.2	-	1.2	82.6	0.9	16.1
30	8.0	8.8	0.0	83.2	0.0	14.1
31	2.5	1.0	3.0	93.2	0.3	4.8

Table 26: Distribution of land use by subcatchment in 1989

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	53.1	17.0	29.2	0.7	30.0
2	20.4	59.9	3.5	16.2	0.0	52.6
3	15.4	75.8	6.9	1.9	0.0	56.4
4	39.3	50.0	4.1	6.6	0.0	65.0
5	25.9	56.7	5.6	11.9	0.0	56.1
6	19.6	61.6	1.7	17.2	0.0	52.8
7	22.4	60.0	3.9	13.6	0.0	54.6
8	10.3	56.7	11.9	21.2	0.0	41.5
9	7.7	61.3	12.7	18.3	0.0	41.5
10	12.0	64.3	1.6	22.1	0.0	47.2
11	8.3	45.0	11.5	34.8	0.4	33.4
12	80.6	-	0.0	19.4	0.0	77.0
13	15.0	-	30.7	52.8	1.5	15.6
14	22.4	-	1.6	76.0	0.0	22.8
15	2.3	-	1.5	96.2	0.0	4.1
16	16.9	-	6.2	76.7	0.2	17.7
17	0.9	-	14.1	85.0	0.0	2.7
18	20.1	-	2.0	77.9	0.0	20.6
19	7.5	-	1.2	91.0	0.4	9.0
20	71.4	-	2.3	25.4	0.9	68.4
21	80.8	-	0.8	18.4	0.0	77.1
22	19.4	-	0.0	80.6	0.0	20.0
23	11.2	-	12.3	76.3	0.2	12.2
24	34.1	-	0.7	65.2	0.0	33.7
25	36.5	-	0.6	62.8	0.1	35.9
26	4.4	-	9.4	86.1	0.0	6.0
27	25.2	-	4.6	70.2	0.0	25.4
28	1.8	-	2.8	95.4	0.0	3.6
29	12.1	-	1.7	86.1	0.2	13.2
30	4.6	9.0	1.5	84.9	0.0	11.0
31	2.0	1.0	4.5	92.5	0.0	4.4

Table 27: Distribution of land use by subcatchment in 1981

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	51.3	15.1	32.8	0.8	29.0
2	18.7	59.0	3.6	18.7	0.0	50.6
3	15.8	75.8	6.5	1.9	0.0	56.8
4	40.0	50.0	3.2	6.8	0.0	65.7
5	26.3	56.0	5.9	11.8	0.0	56.1
6	18.6	61.0	0.8	19.6	0.0	51.6
7	22.8	59.0	3.5	14.7	0.0	54.4
8	9.6	56.0	8.6	25.8	0.0	40.5
9	6.0	60.0	11.9	22.1	0.0	39.3
10	11.1	64.3	1.2	23.4	0.0	46.4
11	7.2	43.0	11.1	38.4	0.3	31.4
12	8.4	-	0.5	91.1	0.0	9.8
13	12.0	-	10.8	77.2	0.0	13.0
14	2.7	-	1.7	95.6	0.0	4.5
15	0.4	-	1.5	98.1	0.0	2.3
16	12.8	-	5.2	81.9	0.1	13.8
17	0.8	-	13.5	85.7	0.0	2.6
18	17.7	-	1.5	80.7	0.1	18.4
19	6.5	-	0.5	92.6	0.4	8.0
20	66.8	-	1.3	30.6	1.2	64.1
21	75.3	-	0.5	24.2	0.0	72.0
22	6.5	-	0.0	93.5	0.0	8.0
23	7.7	-	14.0	78.0	0.3	9.0
24	6.3	-	7.2	86.5	0.0	7.8
25	4.3	-	2.3	93.3	0.0	6.0
26	1.8	-	5.5	92.6	0.0	3.6
27	22.5	-	2.9	74.6	0.0	22.9
28	1.8	-	1.8	96.4	0.0	3.6
29	9.7	-	1.7	88.6	0.1	11.0
30	1.1	5.0	2.3	91.5	0.0	5.7
31	2.0	1.0	4.2	92.8	0.0	4.3

Table 28: Distribution of land use by subcatchment in 1971

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	50.9	13.2	35.9	0.0	28.9
2	19.5	59.0	4.7	16.9	0.0	51.3
3	15.5	75.0	6.3	3.2	0.0	56.1
4	37.2	49.0	3.4	10.4	0.0	62.5
5	24.3	55.0	5.5	15.2	0.0	53.7
6	18.1	60.0	1.4	20.6	0.0	50.6
7	19.8	58.0	4.2	18.0	0.0	51.1
8	7.5	53.0	5.8	33.7	0.1	37.0
9	4.9	57.0	10.7	27.4	0.0	36.7
10	2.4	64.0	3.1	30.5	0.0	38.1
11	4.9	41.0	10.8	43.2	0.2	28.1
12	7.3	-	0.2	92.6	0.0	8.8
13	11.4	-	10.0	78.6	0.0	12.5
14	3.8	-	3.5	92.7	0.0	5.5
15	0.2	-	4.1	95.7	0.0	2.2
16	14.6	-	6.2	79.2	0.0	15.5
17	0.7	-	13.6	85.7	0.0	2.5
18	17.2	-	4.0	78.8	0.0	18.0
19	2.4	-	1.3	96.3	0.0	4.2
20	2.5	-	3.1	94.4	0.0	4.3
21	46.3	-	2.2	51.5	0.0	45.0
22	1.9	-	0.9	97.1	0.0	3.8
23	5.0	-	18.0	76.6	0.3	6.5
24	6.6	-	7.2	86.3	0.0	8.1
25	3.4	-	5.9	90.7	0.0	5.1
26	1.1	-	7.5	91.4	0.0	3.0
27	20.8	-	3.5	75.7	0.0	21.3
28	1.8	-	2.0	96.1	0.0	3.7
29	5.0	-	2.2	92.7	0.1	6.7
30	1.0	4.9	3.9	90.2	0.0	5.5
31	1.9	1.0	4.6	92.5	0.0	4.3

Table 29: Distribution of land use by subcatchment in 1959

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	34.2	21.9	43.9	0.0	19.9
2	7.7	58.0	5.5	28.8	0.0	39.8
3	14.5	74.0	6.1	5.4	0.0	54.6
4	27.0	48.0	5.1	19.8	0.0	52.5
5	2.6	45.0	7.8	44.6	0.0	28.2
6	7.1	45.0	1.4	46.5	0.0	32.4
7	4.5	50.0	4.4	41.1	0.0	32.6
8	1.0	11.0	9.2	78.8	0.0	8.7
9	2.0	12.5	18.4	67.1	0.0	10.3
10	0.0	3.1	7.5	89.4	0.0	3.6
11	0.0	2.8	13.2	84.0	0.0	3.4
12	6.6	-	0.7	92.7	0.0	8.2
13	2.9	-	8.9	88.2	0.0	4.6
14	1.8	-	4.3	93.8	0.0	3.7
15	0.5	-	4.5	95.0	0.0	2.4
16	1.7	-	6.9	91.4	0.0	3.5
17	0.6	-	13.5	85.9	0.0	2.4
18	17.0	-	2.6	80.4	0.0	17.8
19	1.2	-	1.3	97.5	0.0	3.1
20	6.2	-	1.9	91.0	0.9	7.7
21	11.0	-	3.0	86.0	0.0	12.2
22	1.4	-	1.2	97.4	0.0	3.3
23	3.4	-	21.6	75.0	0.0	4.9
24	6.8	-	4.0	89.2	0.0	8.3
25	2.9	-	4.6	92.6	0.0	4.6
26	0.7	-	5.8	93.4	0.0	2.6
27	21.2	-	3.6	75.3	0.0	21.7
28	1.4	-	1.9	96.7	0.0	3.3
29	5.0	-	1.8	93.0	0.2	6.6
30	0.0	4.9	1.4	93.7	0.0	4.6
31	0.0	2.7	3.9	93.4	0.0	3.4

Table 30: Distribution of land use by subcatchment in 1949

Sub. #	Land use area (%)					Total imperviousness (%)
	Commercial & industrial	Residential	Forest	Open land	Water	
1	-	21.2	18.7	60.1	0.0	13.1
2	6.6	30.0	11.1	52.3	0.0	23.9
3	11.6	72.0	5.7	10.7	0.0	50.9
4	2.0	32.7	11.9	53.4	0.0	21.1
5	0.0	6.9	13.3	79.8	0.0	5.5
6	2.0	18.6	3.8	75.6	0.0	13.7
7	2.0	4.3	9.4	84.3	0.0	6.1
8	0.0	2.7	10.7	86.6	0.0	3.4
9	1.0	1.6	23.8	73.6	0.0	3.5
10	0.0	1.4	8.5	90.1	0.0	2.7
11	0.0	1.9	13.8	84.3	0.0	2.9
12	0.0	-	1.2	98.8	0.0	2.0
13	2.9	-	9.6	87.5	0.0	3.4
14	0.9	-	5.1	94.0	0.0	2.4
15	0.6	-	5.3	94.0	0.0	2.3
16	1.5	-	7.3	91.2	0.0	2.7
17	0.8	-	17.9	81.3	0.0	2.2
18	7.9	-	3.1	89.1	0.0	6.1
19	0.5	-	2.5	96.9	0.0	2.3
20	5.4	-	4.3	90.2	0.0	4.8
21	3.3	-	6.7	90.0	0.0	3.7
22	0.8	-	2.2	97.0	0.0	2.4
23	0.9	-	22.1	76.9	0.0	2.3
24	6.1	-	6.6	87.2	0.0	5.2
25	1.9	-	7.0	91.0	0.0	3.0
26	1.1	-	7.8	91.1	0.0	2.5
27	19.0	-	3.6	77.4	0.0	12.0
28	1.0	-	2.7	96.3	0.0	2.5
29	1.3	-	3.1	95.6	0.0	2.7
30	0.0	3.86	7.8	88.3	0.0	4.0
31	0.0	1.74	4.9	93.3	0.0	2.9

Appendix C: Temporal variation of subcatchment total imperviousness from 1949-2015

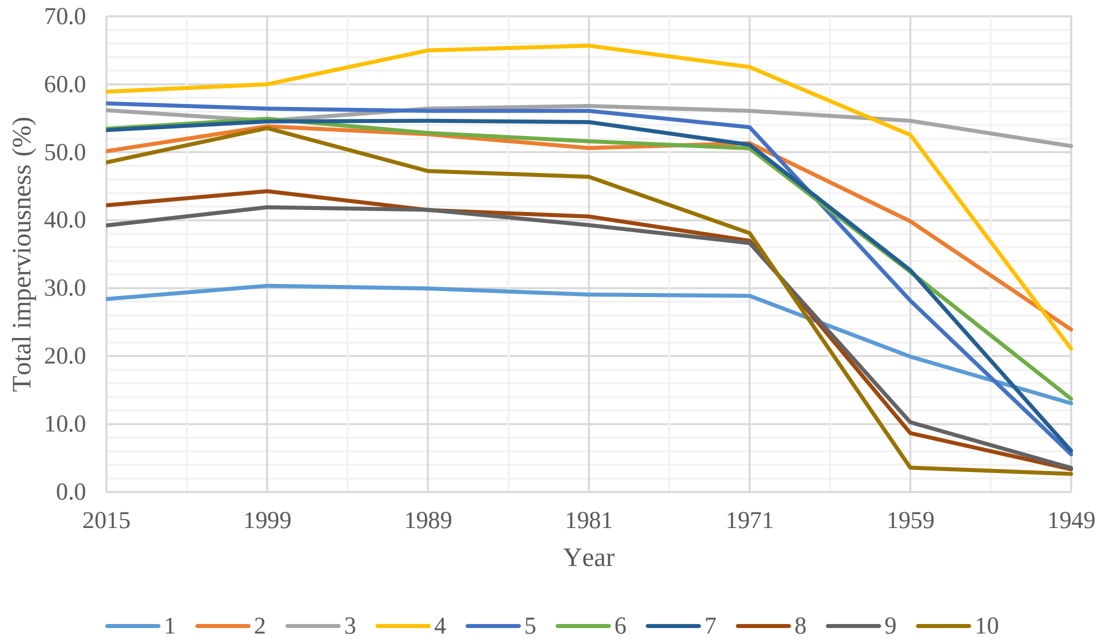

Figure 23: Temporal variation of subcatchment total imperviousness (subcatchments 1-10)

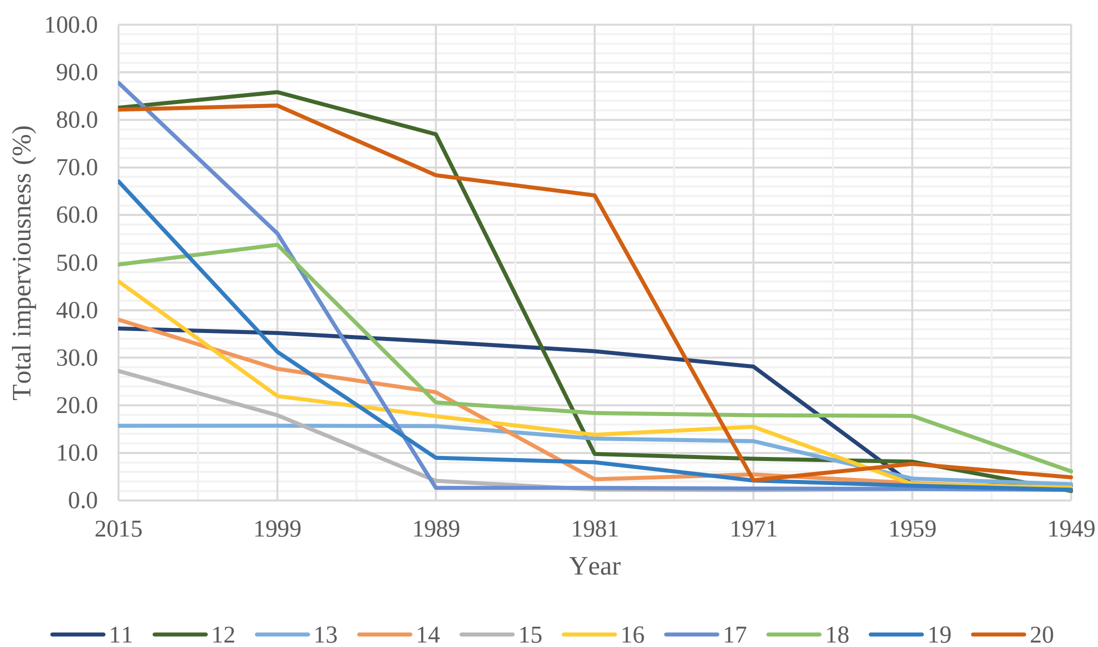

Figure 24: Temporal variation of subcatchment total imperviousness (subcatchments 11-20)

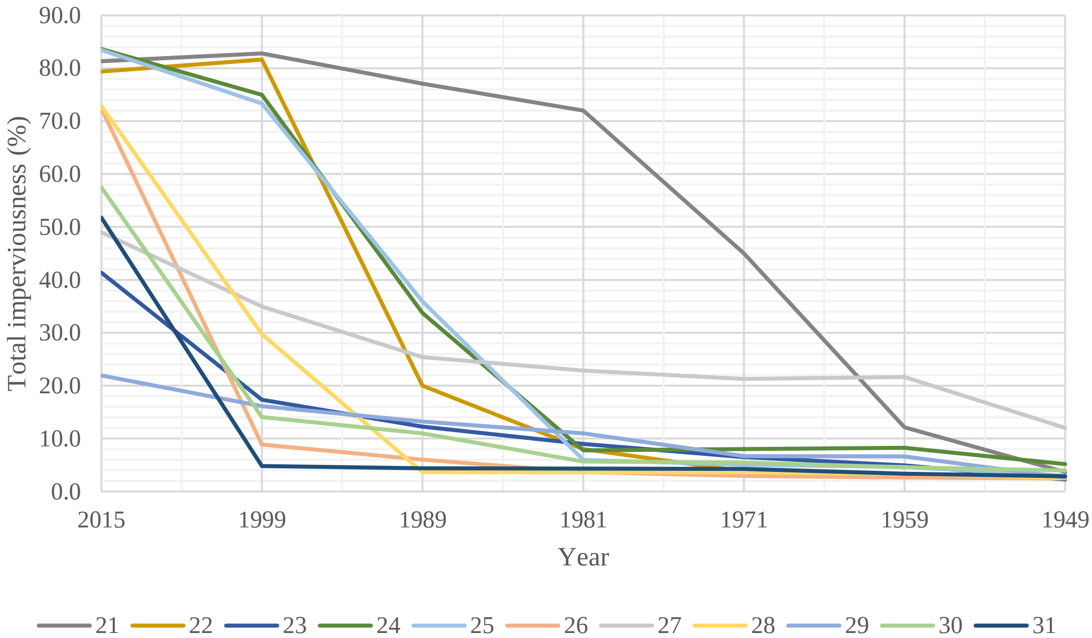

Figure 25: Temporal variation of subcatchment total imperviousness (subcatchments 21-31)

Appendix D: Ponds and hydraulic structures in the Black Creek watershed

Table 31: Details of stormwater ponds

Pond #	Pond name	Area (m^2)	Depth (m)	Earliest year present
1	SWM_VA33	9145	2	2013
2	SWM_VA34	5863	2	1989
3	SWM_VA36	39340	2	1999
4	SWM_VA35	3080	2	1981
5	SWM_ONLINE_VA01	65000	6.2	1989
6	SWM_VA31	13777	2	2013
7	SWM_VA32	54465	2	2013

Table 32: Details of hydraulic structures

Channel #	Channel name	Type	Description	Earliest year present
1	Cha2	Bridge	Scarlett Rd.	1971
2	Cha5	Culvert	Jane St.	1971
3	Cha7	Bridge	Rockcliffe Blvd.	1971
4	Cha10	Bridge	Alliance Ave.	1971
5	Cha11	Bridge	Humber Blvd.	1971
6	Cha14	Bridge	Weston Rd.	1971
7	Cha16	Bridge	Eglinton Ave.	1971
8	Cha19	Bridge	Trethwey Dr.	1981
9	Cha23	Bridge	Black Creek Dr.	1981
10	Cha26	Bridge	Lawrence Ave.	1971
11	Cha29	Bridge	Queen's Dr.	1971
12	Cha31	Bridge	Maple Leaf Dr.	1971
13	Cha33	Bridge	Highway 400	1981
14	Cha35	Bridge	Highway 400 W. on ramp (#1)	1971
15	Cha37	Bridge	Jane St.	1971
16	Cha39	Bridge	Highway 400 W. on ramp (#2)	1971
17	Cha41	Bridge	McKay St.	1971
18	Cha43	Bridge	Jane St.	1971
19	Cha46	Culvert	Highway 401	1971
20	Cha48	Bridge	Downsview Ave.	1971
21	Cha50	Bridge	Wilson Ave.	1971
22	Cha52	Bridge	Jane St.	1971
23	Cha54	Bridge	Private Concrete Driveway	1971
24	Cha56	Bridge	Concrete footbridge	1981

25	Cha61	Culvert	Jane St.	1971
26	Cha63	Reservoir with culvert	Stone Flood Control Reservoir	1971
27	Cha66	Bridge	Park Access Rd.	1971
28	Cha69	Culvert	Sheppard Ave.	1971
29	Cha73	Culvert	Grandravine Dr.	1981
30	Cha77	Bridge	Park Access Rd.	1981
31	Cha79	Culvert	Finch Ave.	1971
32	Cha83	Bridge	Shoreham Dr.	1971
33	Cha87	Bridge	Steeles Ave. & Jane St.	1971
34	Cha91	Culvert	CNR	1971
35	Cha94	Culvert	Jane St.	1999
36	Cha96	Culvert	Highway 407	1999
37	Cha98	Culvert	Peelar Rd.	1971
38	Cha100	Culvert	Private Driveway	1989
39	Cha102	Culvert	Private Driveway	1989
40	Cha104	Culvert	Paradise Convention Centre	1989
41	Cha106	Culvert	Doughton Rd.	1971
42	Cha108	Culvert	Private Driveway	1981
43	Cha110	Culvert	Private Driveway	1981
44	Cha112	Culvert	Highway 7	1971
45	Cha116	Culvert	Jane St.	1989
46	Cha118	Bridge	Jane St.	1989
47	Cha120	Culvert	Pennsylvania Ave.	1989
48	Cha122	Culvert	Millway Ave.	1989
49	Cha125	Culvert	Edgeley Blvd.	1989
50	Cha127	Culvert	Applewood Cresc.	1989
51	Cha129	Culvert	Highway 400/Langstaff Rd. Ramp	1999
52	Cha131	Culvert	Highway 400	1999
53	Cha133	Culvert	Creditview Rd.	2013

Appendix E: Outlet hydrographs for other return periods

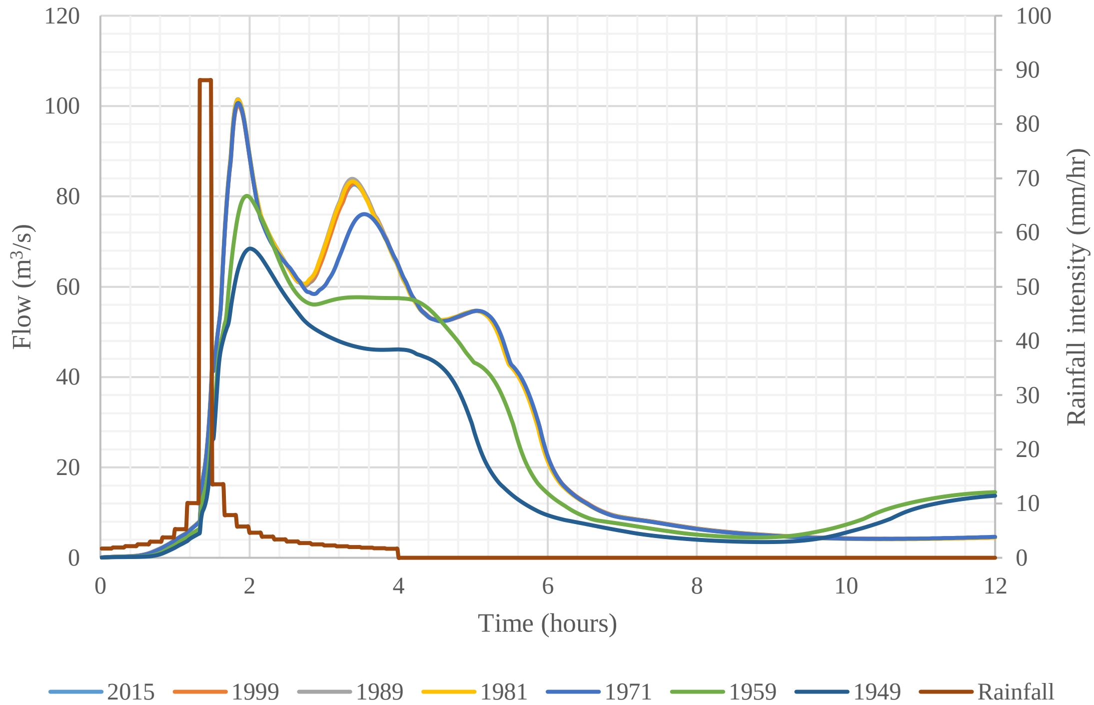

Figure 26: Outlet hydrographs in response to a 5-year event for all time periods

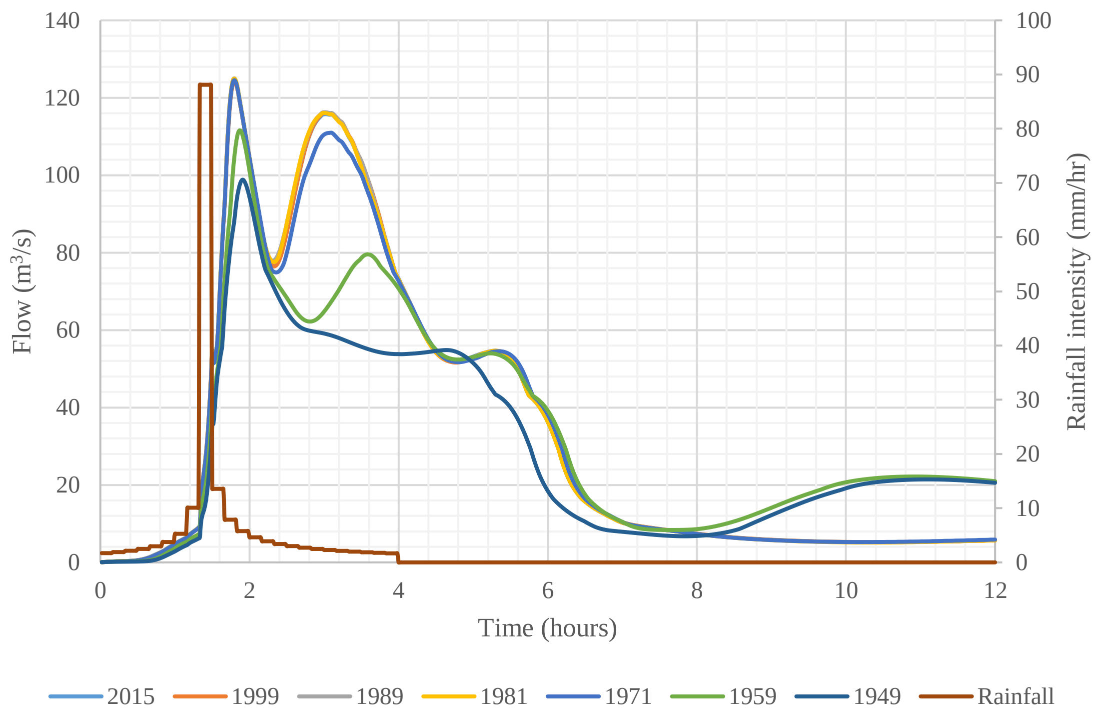

Figure 27: Outlet hydrographs in response to a 10-year event for all time periods

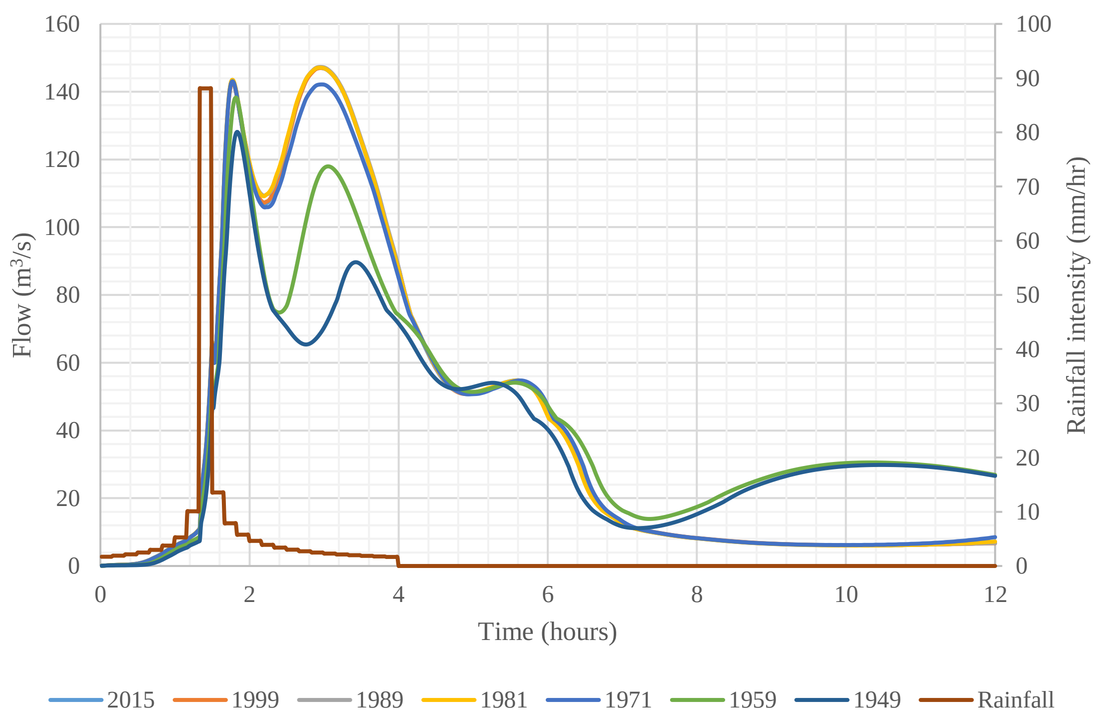

Figure 28: Outlet hydrographs in response to a 25-year event for all time periods

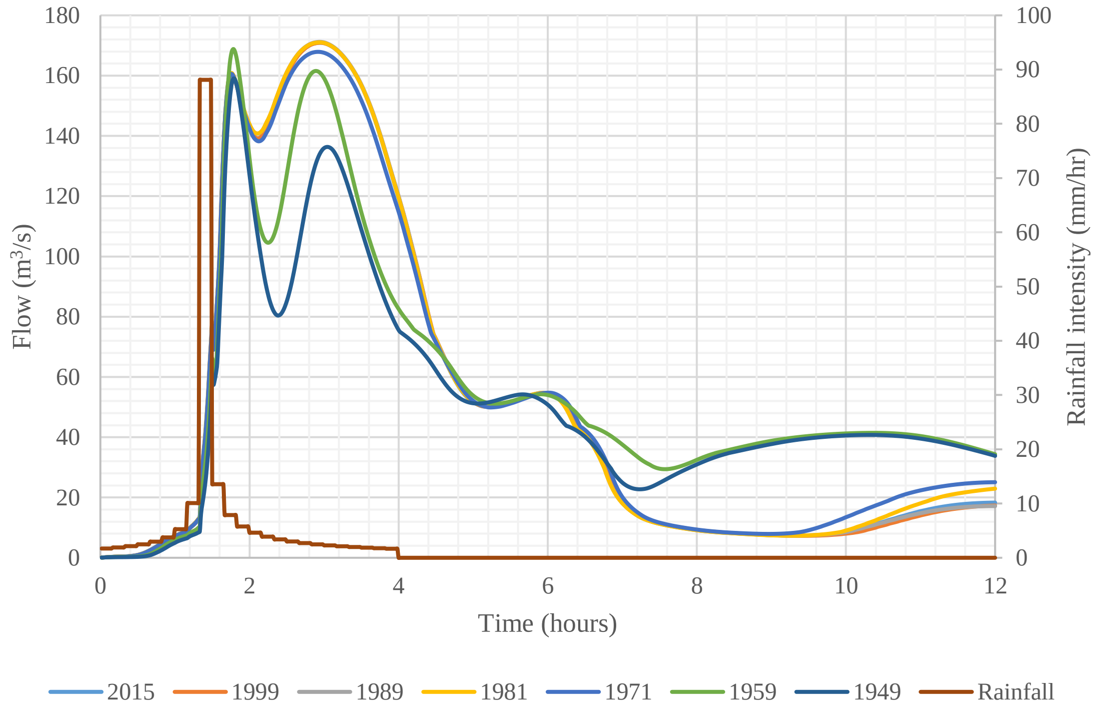

Figure 29: Outlet hydrographs in response to a 50-year event for all time periods

Appendix F: Runoff hydrographs for other return periods

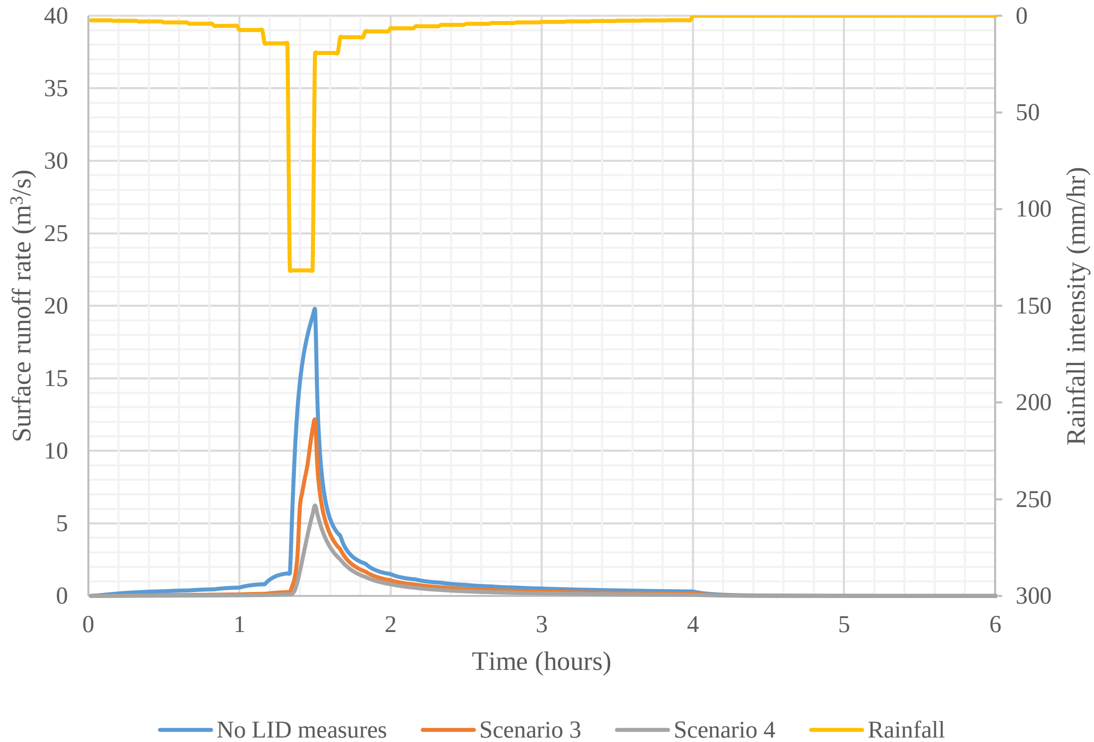

Figure 30: Runoff hydrographs for Scenarios 3 and 4 (5-year event)

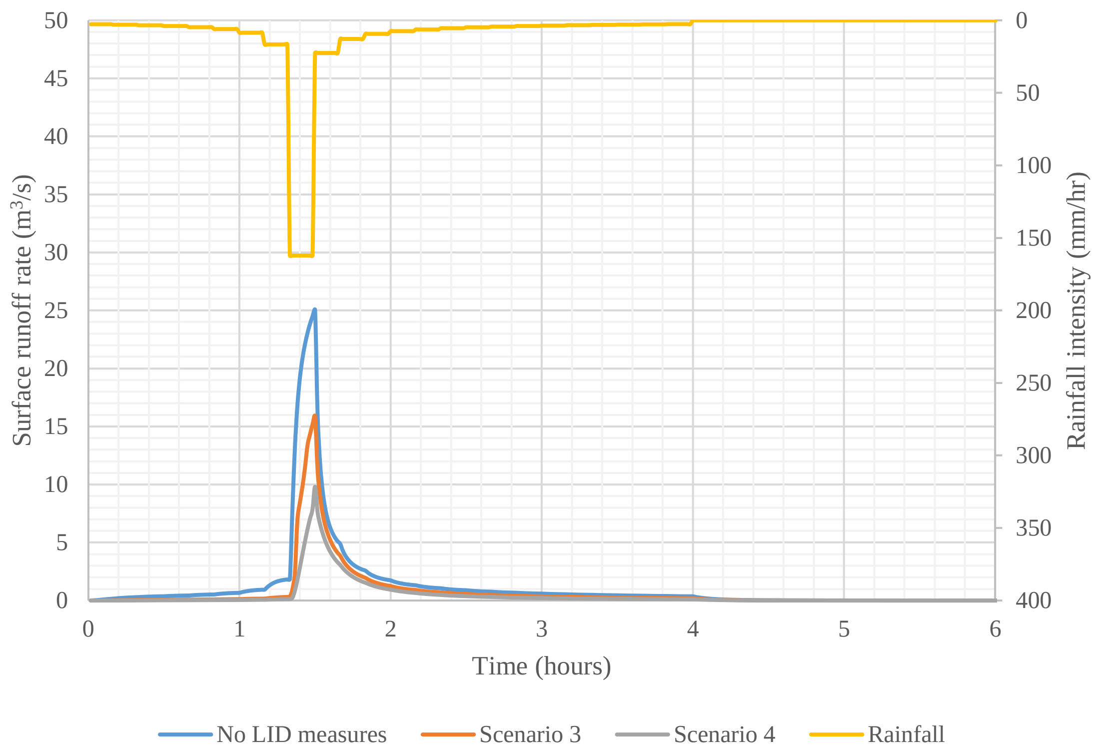

Figure 31: Runoff hydrographs for Scenarios 3 and 4 (10-year event)

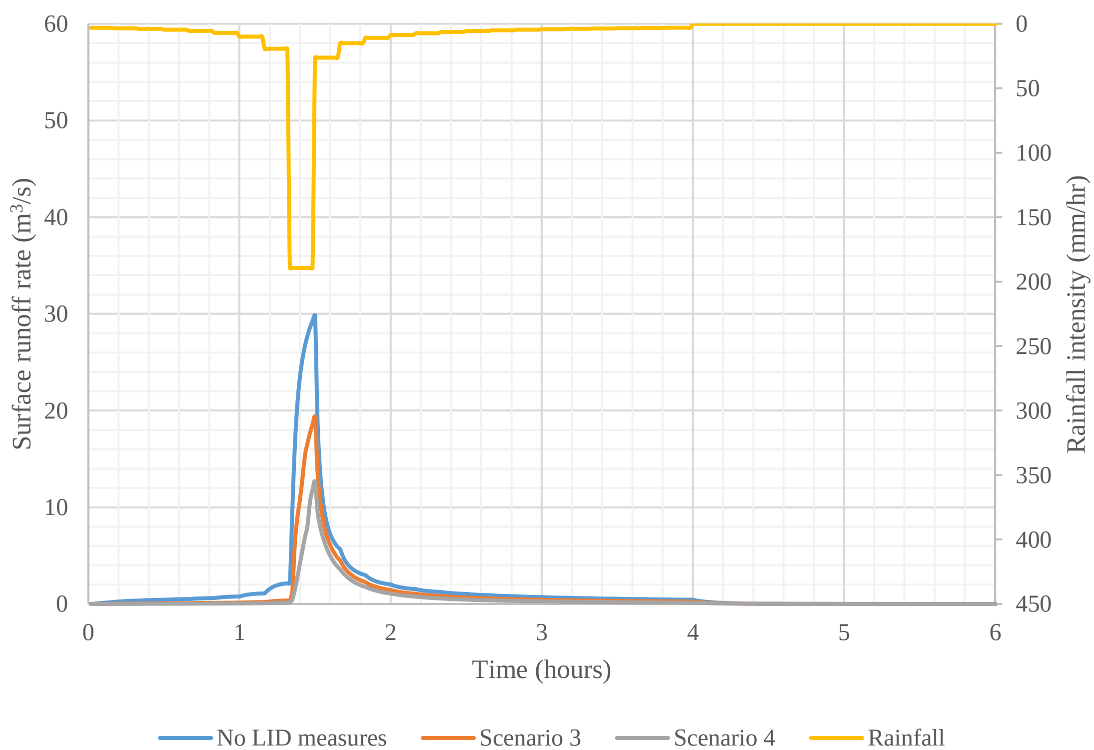

Figure 32: Runoff hydrographs for Scenarios 3 and 4 (25-year event)

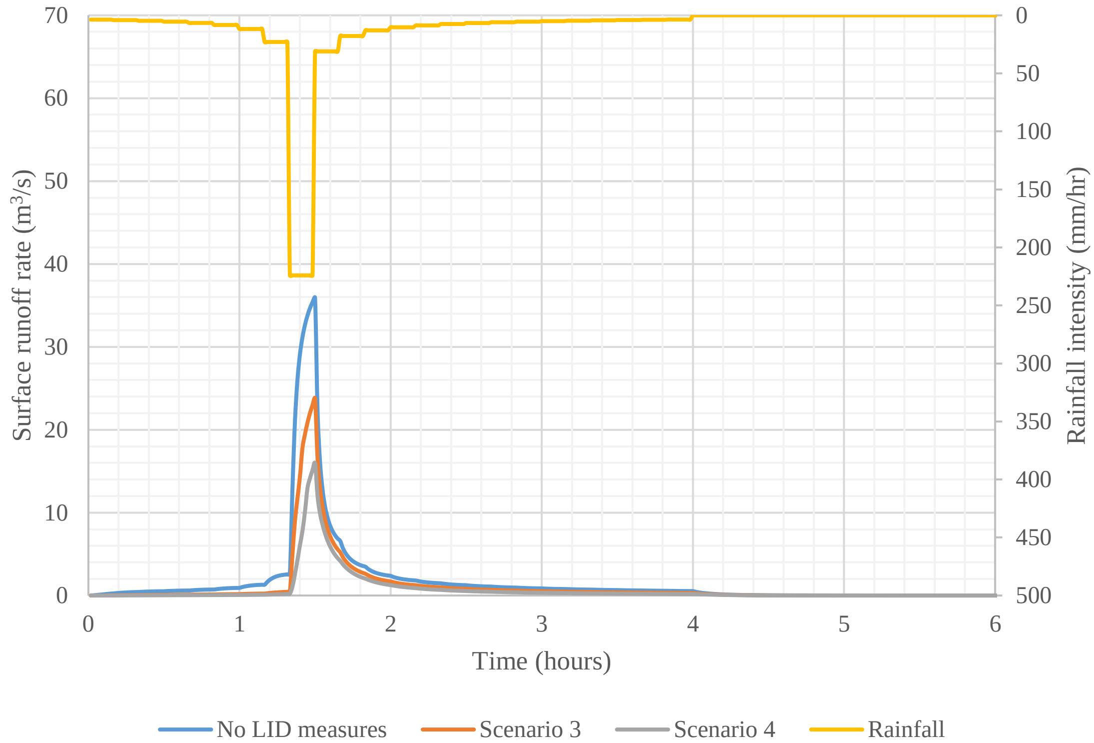

Figure 33: Runoff hydrographs for Scenarios 3 and 4 (50-year event)

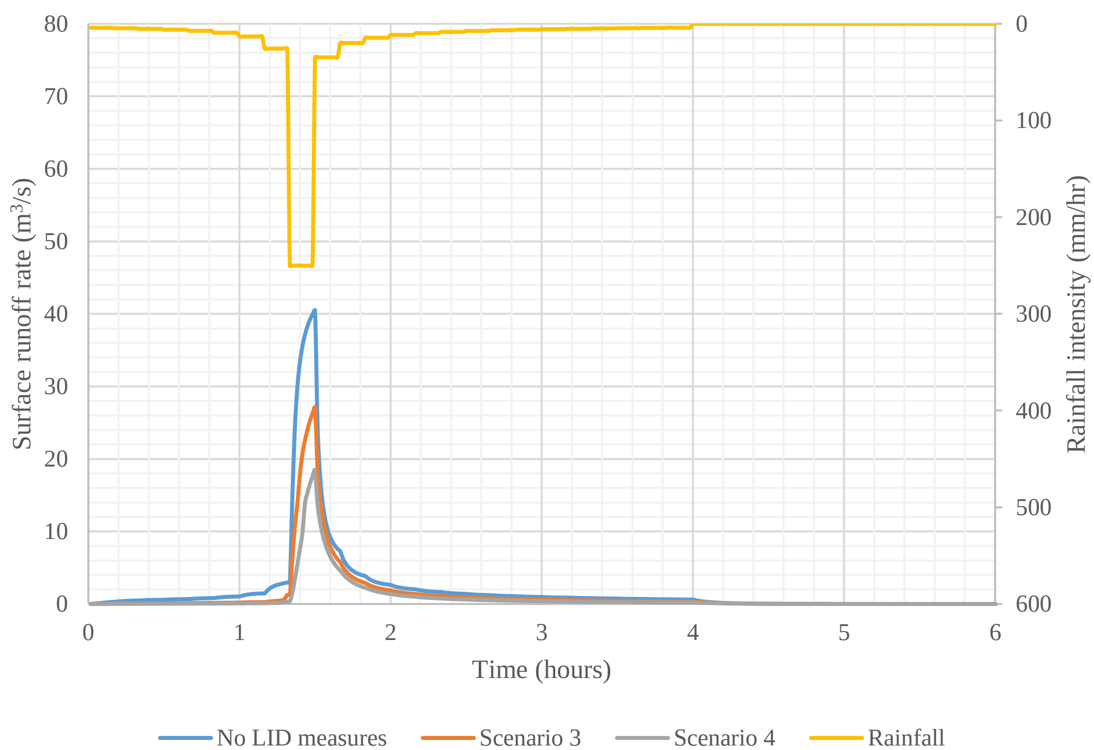

Figure 34: Runoff hydrographs for Scenarios 3 and 4 (100-year event)

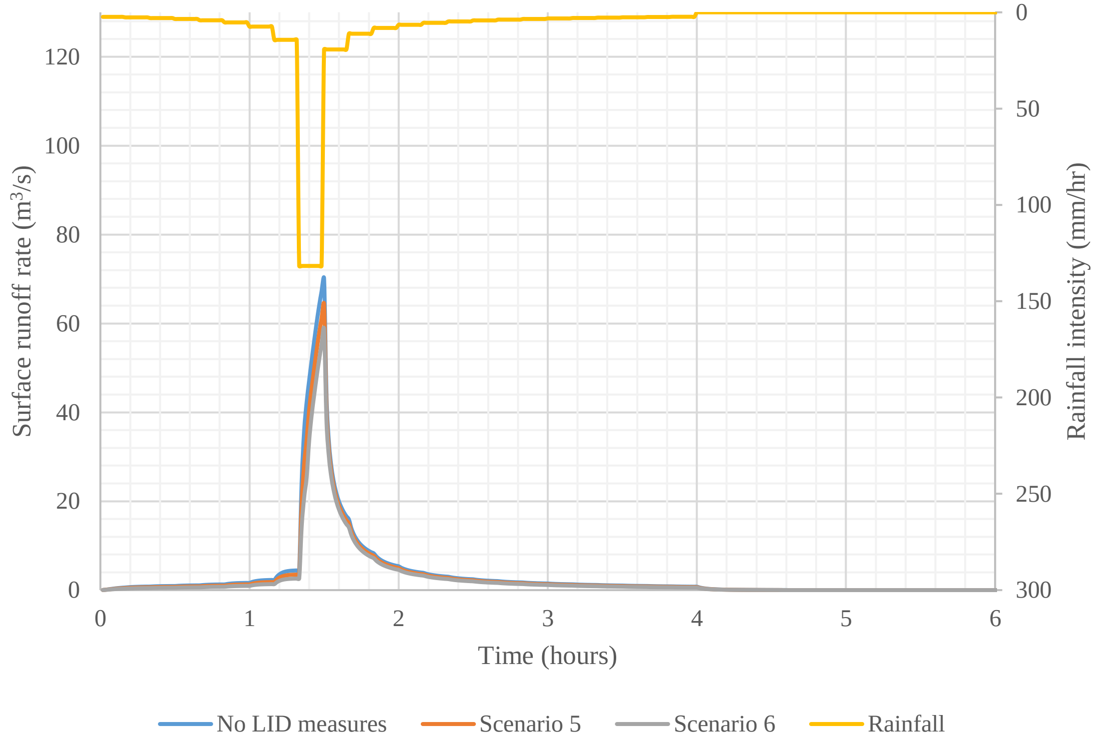

Figure 35: Runoff hydrographs for Scenarios 5 and 6 (5-year event)

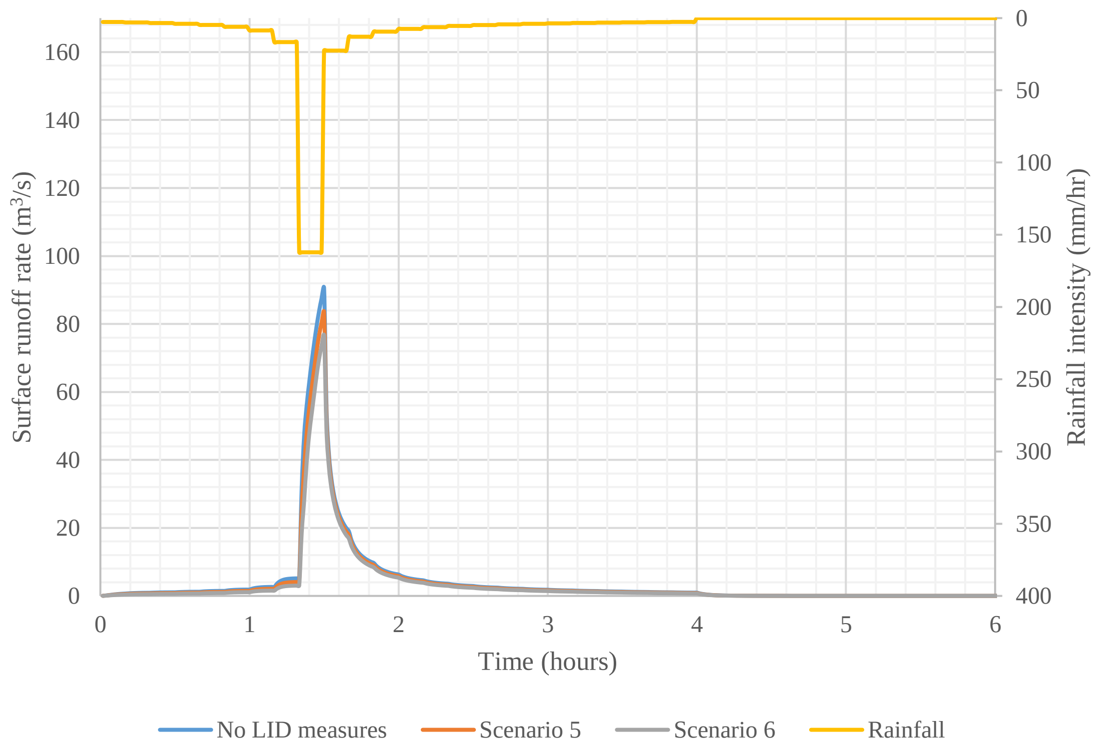

Figure 36: Runoff hydrographs for Scenarios 5 and 6 (10-year event)

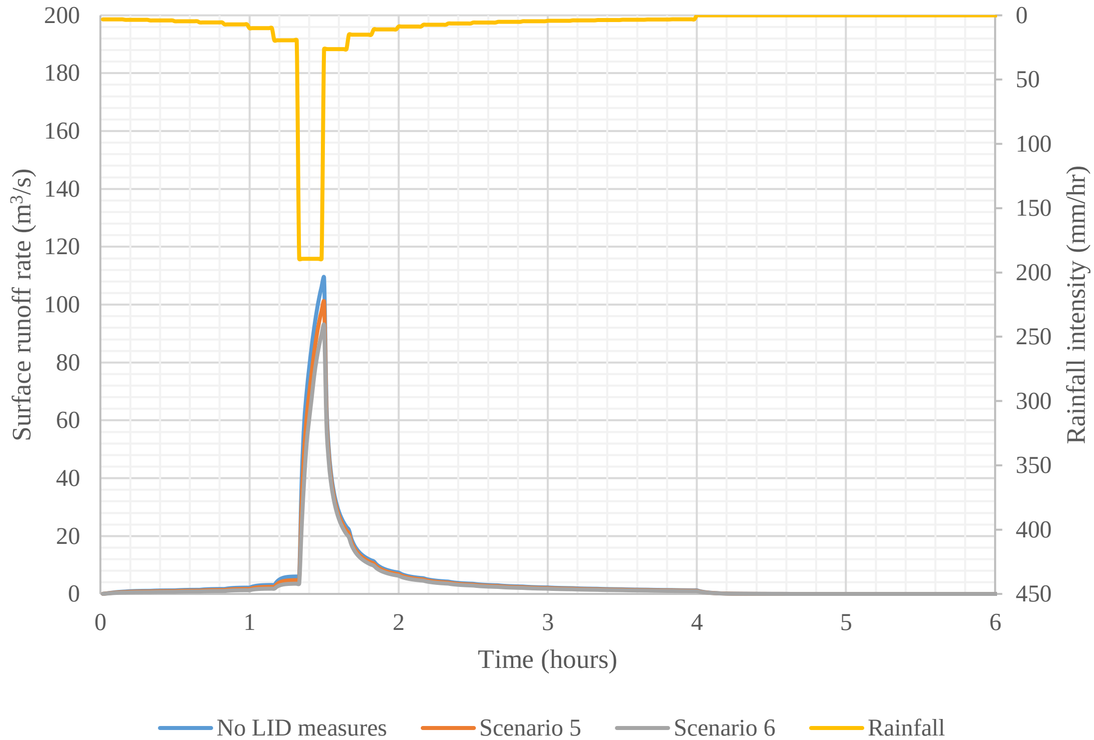

Figure 37: Runoff hydrographs for Scenarios 5 and 6 (25-year event)

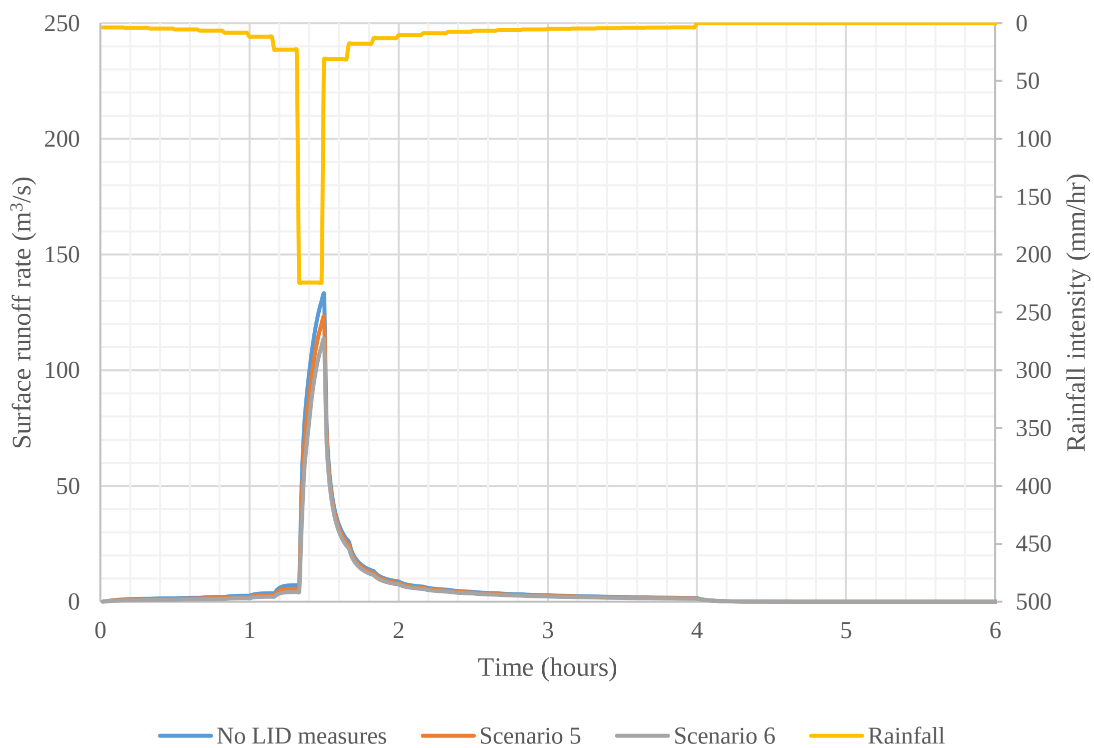

Figure 38: Runoff hydrographs for Scenarios 5 and 6 (50-year event)

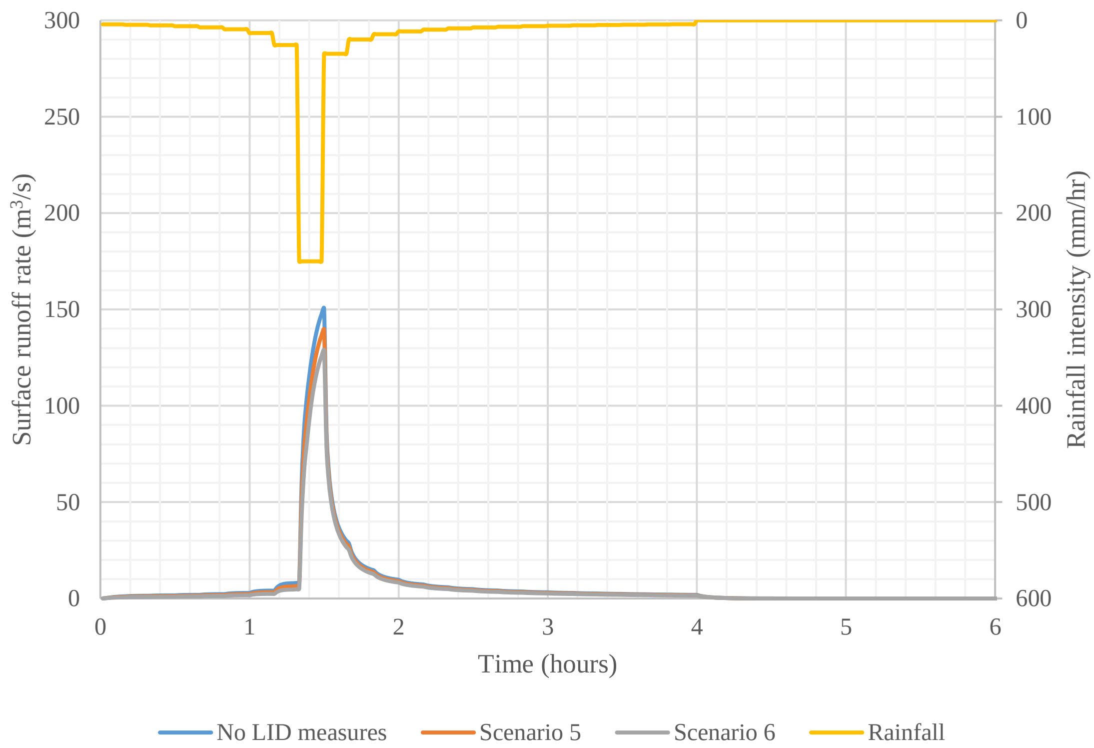

Figure 39: Runoff hydrographs for Scenarios 5 and 6 (100-year event)

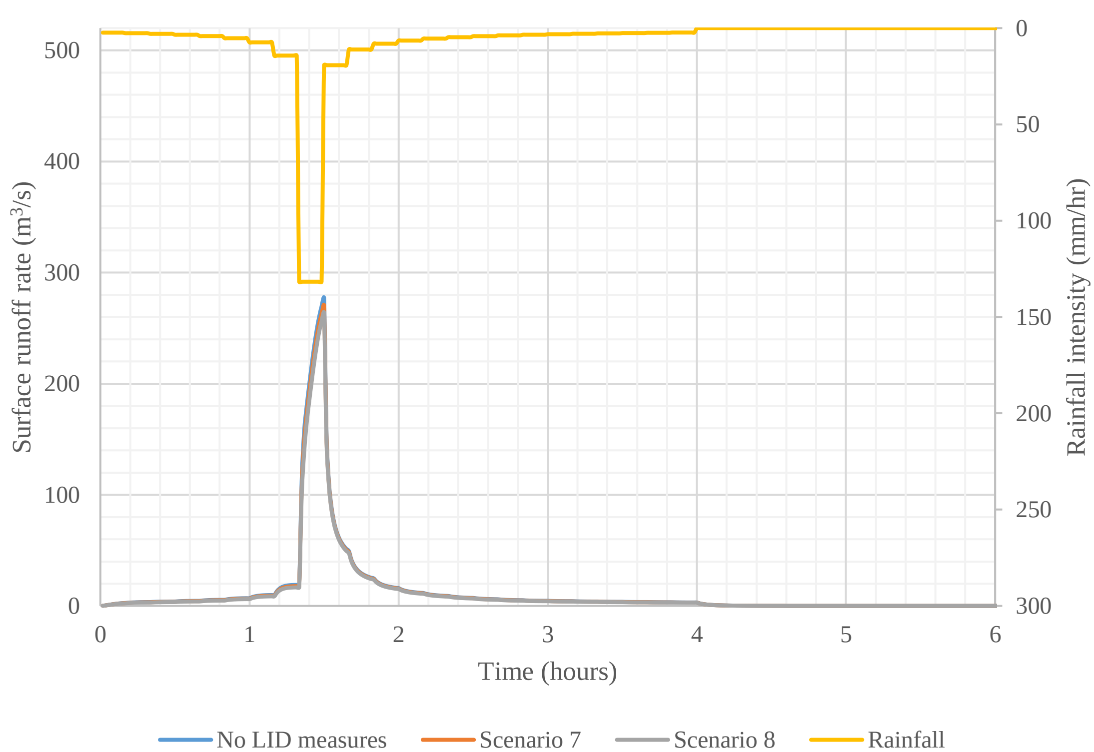

Figure 40: Runoff hydrographs for Scenarios 7 and 8 (5-year event)

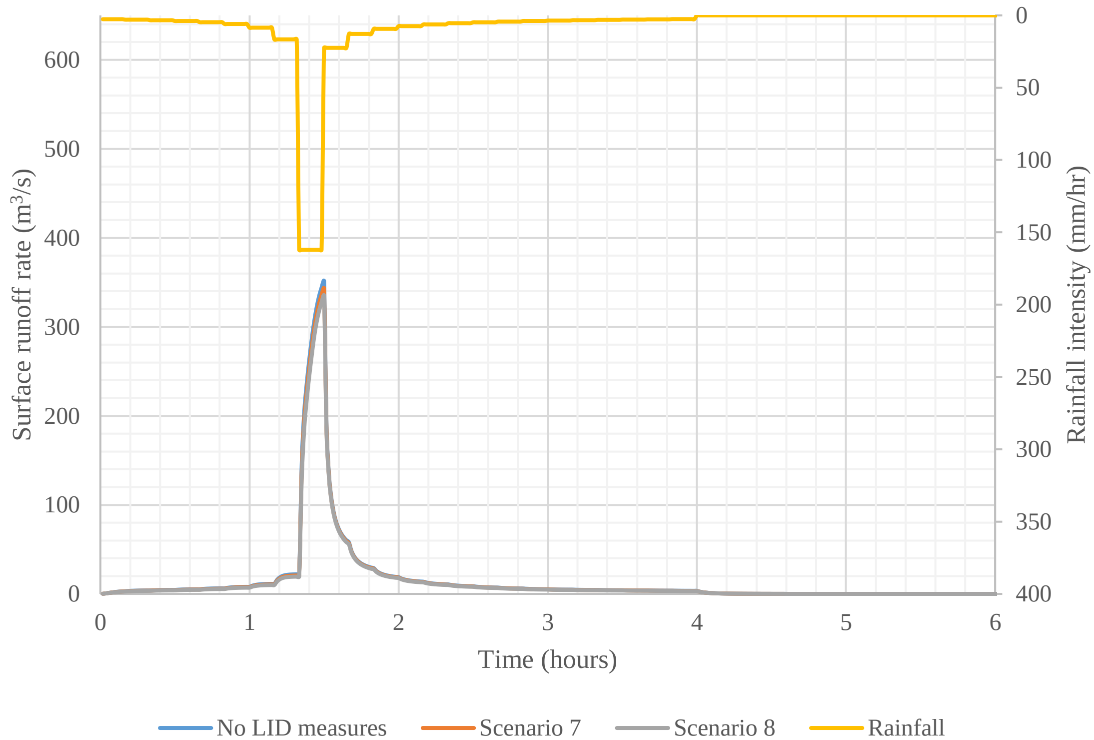

Figure 41: Runoff hydrographs for Scenarios 7 and 8 (10-year event)

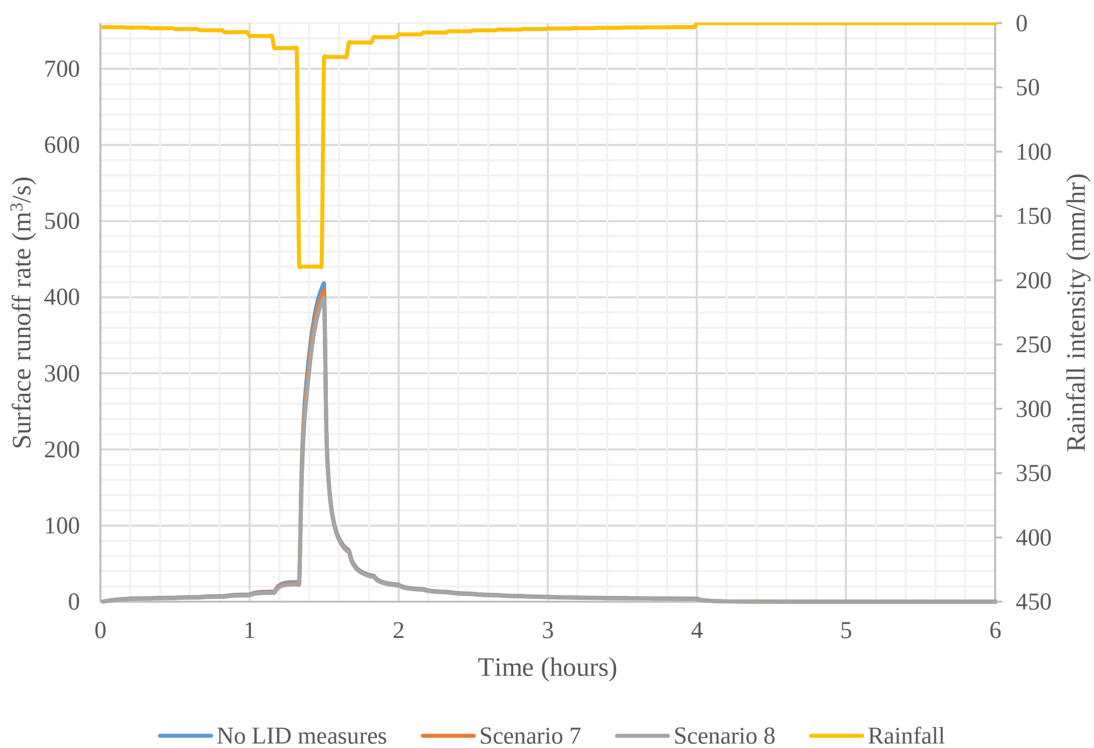

Figure 42: Runoff hydrographs for Scenarios 7 and 8 (25-year event)

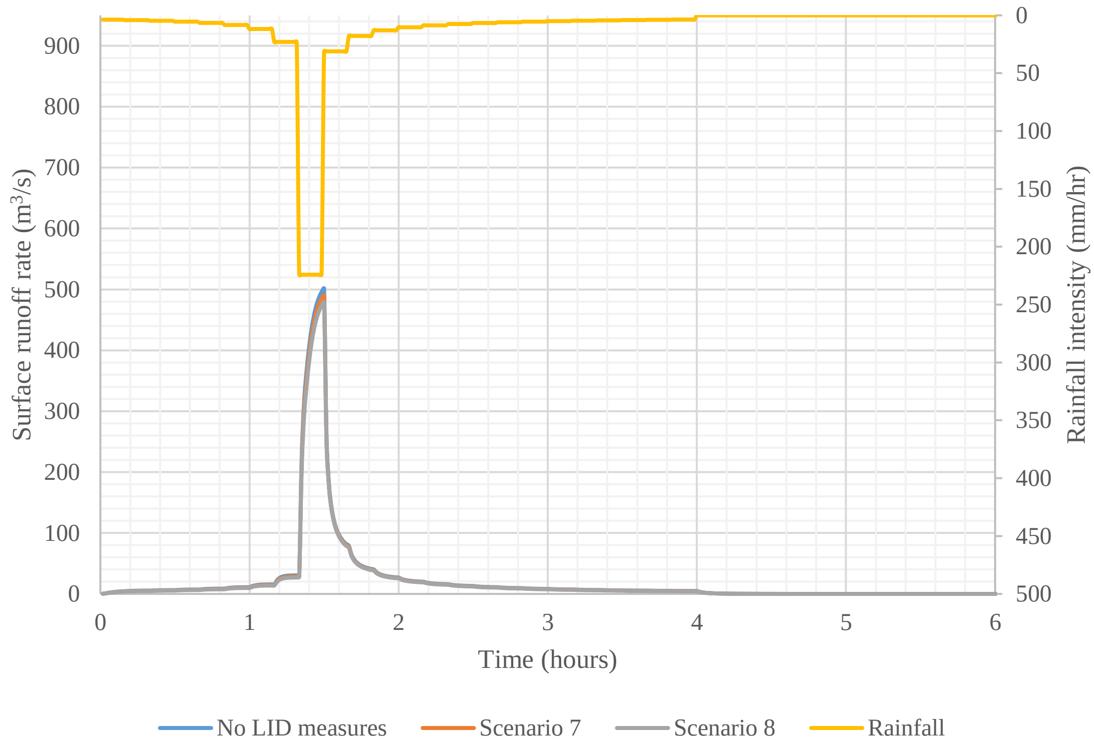

Figure 43: Runoff hydrographs for Scenarios 7 and 8 (50-year event)

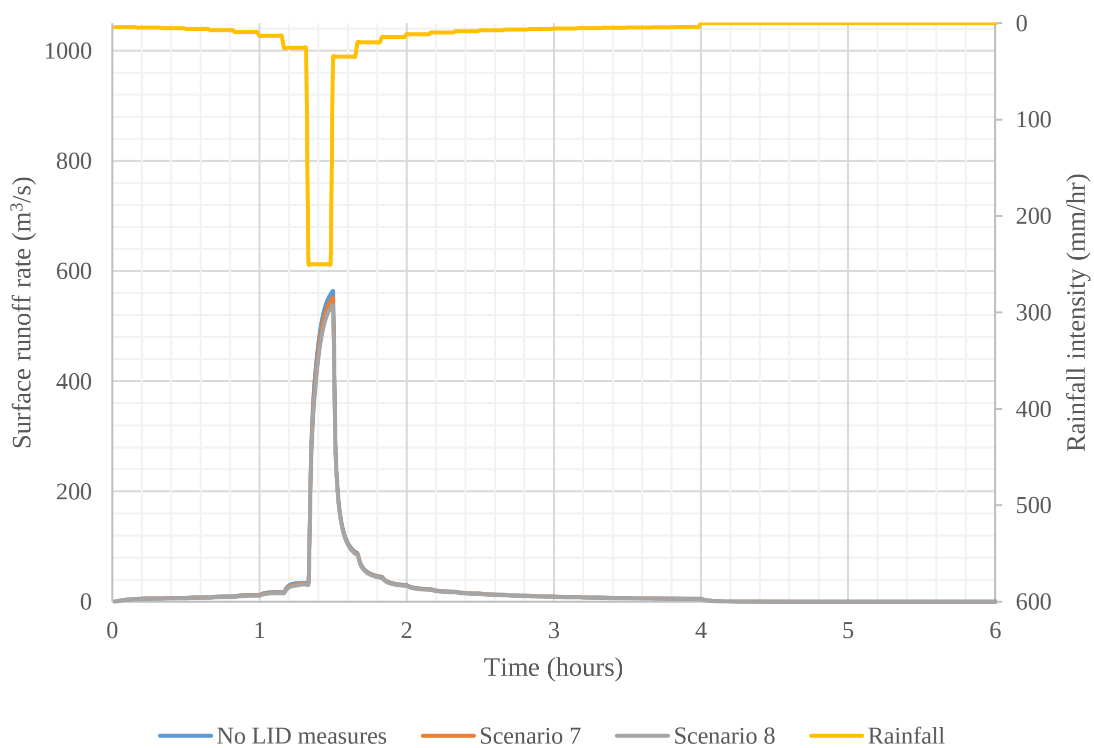

Figure 44: Runoff hydrographs for Scenarios 7 and 8 (100-year event)

Appendix G: Description of files on USB

This section discusses the files saved on the USB provided. These files represent all of the results obtained during the research of this thesis.

Land Use Analysis

This folder contains files related to the land use analysis component of this research. The folder "Aerial Photos" contains the original aerial photographs collected from the TRCA, as well the files associated with modifying and digitizing the photographs (broken down by time period). The folder "Center Points for Aerial Photos" contains a shapefile that can be opened in ArcMap to view the center point locations of all aerial photographs collected from the TRCA (was used to give a general idea of where photographs were located to aid in georeferencing). The folder "Outlines" contains the shapefiles associated with creating the outlines of the Black Creek watershed and 31 subcatchments in ArcMap. The file "Black Creek Historical Analysis" is the main file that can be opened in ArcMap to view the land use analysis work. This folder relates to Chapter 5 of this thesis paper.

Modeling

This folder contains files related to the modeling component of this research. The PCSWMM file for each historical and LID scenario simulation will allow you to view the model setup. All other files and folders relate to each simulation (created by PCSWMM) and should not be modified. This folder relates to Chapters 5 and 6 of this thesis paper.

Site Visit

This folder contains pictures from a site visit to Black Creek.

Current Time Simulations

These Excel files present the PCSWMM output results from the current time modeling simulations (LID scenarios). Information such as precipitation, infiltration, runoff, peak runoff, and runoff coefficient for each subcatchment are listed. Information such as max flow, time of max, lateral inflow volume, and total inflow volume for each node in Black Creek are listed. Details for each LID measure (inflow, evaporation loss, infiltration loss, surface outflow, drain outflow, initial storage, final storage, and continuity error), as output by PCSWMM, is presented. The subcatchment runoff and node inflow hydrographs are also included. Scenarios 1 and 2 correspond to Scenarios 1 and 2 of this thesis, Scenarios 3, 4, and 5 are extra scenarios not included in this thesis, and Scenarios 6, 7, 8, 9, 10, and 11 correspond to Scenarios 5, 6, 7, 8, 4, and 3, respectively, of this thesis. Scenarios labelled with a "_2" represent results from the modified LID parameters analysis. These files relate to Chapter 6 of this thesis paper.

Historical Breakdown of Hydraulic Structures

This Excel file presents the various hydraulic structures built into the PCSWMM model. The names of these structures are listed as well as which time period they were last seen in the historical aerial photographs. This file relates to Chapter 5 of this thesis paper.

Historical Results

This Excel file summarizes the information from the "Historical Simulations" Excel files, and presents plots of the hypothetical outlet hydrographs for Black Creek from 1949-2015. This file relates to Chapter 5 of this thesis paper.

Historical Simulations

These Excel files present the PCSWMM output results from the historical modeling simulations. Information such as precipitation, infiltration, runoff, peak runoff, and runoff coefficient for each subcatchment are listed. Information such as max flow, time of max, lateral inflow volume, and total inflow volume for each node in Black Creek are listed. The subcatchment runoff and node inflow hydrographs are also included. These files relate to Chapter 5 of this thesis paper.

Land Use Analysis

This Excel file summarizes the land use analysis work. By time period, each subcatchment contains general information, as well as the total area of each land use category calculated in ArcMap from the digitization phase, the total imperviousness, and the percent difference in land use from time period to time period. Plots of the change in total imperviousness over time, for each subcatchment, is also included. This file relates to Chapter 5 of this thesis paper.

LID Costing and Specification Sheet

This Excel file provides some general information about real-life examples of LID measures (size, cost, etc.). Specifications for each LID unit used in the simulations of this research (as required by the PCSWMM model) is also included. This file relates to Chapter 6 of this thesis paper.

Scenario Results

This Excel file provides a summary of the effects of each LID scenario on subcatchment parameters (infiltration, runoff, peak runoff, runoff coefficient) and on node parameters located between each channel segment of Black Creek (max total flow, time to max, lateral inflow volume, total inflow volume). Final plots are also included. This file relates to Chapter 6 of this thesis paper.

Simulation Scenarios

This Excel file provides specifications for each LID scenario. The number of each LID measure, total cost, subcatchment total imperviousness after implementation of each scenario, and percent of impervious area treated by each LID measure is included. This file relates to Chapter 6 of this thesis paper.

Curriculum Vitae

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Post-secondary Education and Degrees:	The University of Western Ontario London, Ontario, Canada 2009-2014 B.E.Sc.
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Related Work Experience:	Research Assistant The University of Western Ontario 2013
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Publications:

Kokas, T., S. P. Simonovic and A. D. Binns. 2016. Flood Risk Management in Canadian Urban Environments: A Comprehensive Framework for Water Resources Modeling and Decision-Making. London, Ontario, Canada: *Water Resources Research Report no. 095, Facility for Intelligent Decision Support, Department of Civil and Environmental Engineering*. 54 pages. ISBN: (print) 978-0-7714-3117-3; (online) 978-0-7714-3118-0.

Presentations:

Kokas, T., A. D. Binns and S. P. Simonovic. 2016. Evaluating the effects of urbanization in the Black Creek subwatershed. Montreal, Canada: *69th National Conference of the Canadian Water Resources Association (CWRA)*, 25-27 May 2016.