The Impacts of Stormwater Management on Hydromodification and Bedload Sediment Transport in a Gravel-bed Stream

by

Thommaso Victor Raso

A thesis
presented to the University of Waterloo
in fulfillment of the
thesis requirement for the degree of
Master of Applied Science
in
Civil Engineering (Water)

Waterloo, Ontario, Canada, 2017

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

I understand that my thesis may be made electronically available to the public.

Abstract

Global trends of increasing urbanization have led many researchers to attempt to quantify the resulting impacts on channel morphology and on the ecological health of urban channels. Overall, there has been a lack of research on the effectiveness of stormwater management facilities in reducing the potential for bedload sediment transport within the channel. This study aimed to characterize the impact of stormwater management facilities on bedload sediment transport potential within a particular urban stream.

A field study was undertaken along Morningside Creek, a tributary of the Rouge River in Toronto, Ontario. A tracer study using 300 stones with radio frequency identification tags was undertaken over a two year period, tracking the positions of the stones after major storm events. A hydrologic model was then prepared for the catchment area, detailing the stormwater management features.

A critical shear stress of 0.043 was determined for the threshold of particle mobilization, using a hiding factor to account for the increase in shear stress in the displacement of larger particles. The travel distances of the tagged particles were shown to follow a non-linear decreasing function with particle size. A variety of hydrometrics were measured based on the high resolution water level measurements taken within the creek. The cumulative excess shear stress was used as the basis for determining bedload sediment potential within the creek. A model of bedload sediment transport was developed by modifying the excess shear stress relationship first proposed by DeVries (2000). The hydrologic model was used to compare a variety of stormwater management scenarios to determine their effectiveness in reducing the potential for sediment transport within the creek. It was determined that the detention ponds in the Morningside Creek catchment provide a 33% reduction in bedload sediment transport potential. Analysis of the hydrologic model revealed that increases in imperviousness lead to a proportionate increase in bedload sediment transport. The hydrologic model also determined that for storms of a similar return period, longer storm durations generate larger bedload sediment transport potential.

Acknowledgements

First and foremost I'd like to thank God for providing me with the graces needed to complete this degree.

I would like to thank my supervisor, Bruce MacVicar for the endless support and mentorship in completing this degree.

I am thankful for the financial support provided by NSERC and MMM/WSP that ultimately made this thesis possible.

I would like to acknowledge the support I received from the TRCA, in particular Rita Lucero, Cameron Wilson, and Jamie Duncan and Mark Wodja from Matrix Solutions for providing data required to complete this thesis.

I would like to extend a huge thank you to Elli Papangelakis, Asal Montakh, Terry Ridgway, Julian Krick and the numerous co-op students who contributed their time and efforts out in the field collecting the tracer data and back in the office processing the data.

Finally, to my family and friends who provided the support I needed, from start to finish, in pursuit of this degree, thank you.

Table of Contents

List of Figures

List of Tables

Chapter 1
Introduction

Urbanization and land use changes have strong implications for the overall health and function of natural water courses. The growth of urban centres globally is leading to systematic changes in the natural environment, particularly impacting urban watercourses in what has been labelled by Meyer et al. (2005) as the 'urban stream syndrome' (Walsh et al., 2005). Symptoms include a flashier hydrograph, increased flooding and erosion, rapid geomorphic alteration and reduced biotic richness. Streams are dynamic systems that are subject to unsteady flows and sediment supply and are continuously changing in time. The relationship between channel flow and sediment transport in rivers has been a topic of research for many years with researchers seeking to understand the drivers causing the transport of sediment through fluvial systems (Lane, 1955; Schumm, 1969; Trimble, 1997). Despite growing research efforts to quantify the impacts of urbanization on natural water courses, the urban stream syndrome remains a prevalent issue.

Urbanization and increases in impervious land cover have been shown to alter the hydrologic regime and stream response, leading to greater peak flows (Hollis, 1975; MacRae, 1996) and storm runoff volumes (Bledsoe, 2002; Smith et al., 2013). Urban storm runoff creates flashier stream responses, resulting in rapid rising and falling water levels after storm events (Baker et al., 2008; Rosberg et al., 2017). In urban areas, the increase in erosion potential is greatest in small frequent events (MacRae & Rowney, 1992), with an increase in frequency of channel forming flows (Annable et al., 2010). Many studies have identified accelerated fluvial processes in urban streams often related to the large changes in hydrology due to urbanization (Doyle et al., 2000). Bedload transport is a crucial processes that largely defines the morphology of a stream (Church, 2006). In urban environments, the rate of stream modification can be of particular importance because of potential impacts to infrastructure located near streams. Modern stormwater management strategies, specifically detention and storage type facilities, have developed in response to urbanization in order to protect urban streams. There have been many difficulties in developing effective best management practices that protect urban streams against the changes in hydrology due to urban intensification. The highly variable nature of precipitation events, high variance in temporal solution effectiveness, mixed with an ever changing climate has made it difficult to find robust solutions. Intrinsic to stormwater management at the catchment scale is the complexity of the cumulative downstream impacts of stormwater management (McGuen 1974; Goff & Gentry, 2006).

Despite the best of intentions, the implementation of detention type facilities can often lead to increases in downstream erosion potential (MacRae, 1996; Booth & Jackson, 1997; Bledsoe, 2002). Another problem is the increase in urbanization leads to degradation of vital ecological conditions necessary for many sensitive species to inhabit these once natural streams (Walsh et al., 2005). The stream bed, comprised of larger bedload particles in semi-alluvial and alluvial systems, is habitat for many species, providing breeding and feeding grounds. The rate of bed disturbance is an important factor in the species richness of a stream environment (Townsend et al., 1997). Despite growing recognition of the impacts of urbanization, there continues to be a lack of field based assessment measuring the downstream impacts of stormwater management on erosion and sediment transport in urbanized water courses.

The aim of this research is to better understand the downstream impacts of stormwater management ponds on bedload sediment transport. The goal is to link hydrologic response with sediment movement in an urban stream at the individual storm event scale. Specific objectives include i) use field sediment tracking data to test available relations that link hydrologic response with sediment transport, ii) calibrate an event based hydrologic model to observed floods and iii) use the hydrologic model to assess scenarios with different stormwater management strategies, urbanization level and storm return period. The hope is that this study will improve the techniques for mitigating negative impacts of the urban stream system

This study looks at reach scale response to catchment wide alterations in land use and their impact on event scale hydrology and sediment transport. Whereas the majority of previous research has focused on larger temporal trends, the hypothesis of the current work is that a focus on the smaller scale will provide better resolution details of the driving forces behind bedload sediment transport and a better explanation of the channel instability that can result from urbanization. This work is part of a larger study seeking to understand urban hydrology and its impacts on bedload sediment transport and ecological stream health.

Chapter 2
Literature Review

2.1 Urban Rivers

The growth of urban centers is not a new phenomenon. From 1950 to 2014, the number of people living in urban areas increased by 24% and the urban population is projected to continue increasing (United Nations, 2015). With this shift toward further urbanization it is vital that we understand the impacts to streams and rivers caused by urban development. The impacts of urbanization on stream health has been a focus of research over past decades spanning across a variety of disciplines. The term "urban stream syndrome" was coined by Meyer et al. (2005) to describe the growing acknowledgment of the trend of ecological degradation in urban watercourses (Walsh et al., 2005). Chin (2006) took a global context approach and looked at the impacts that urbanization causes on river landscapes. The impacts of urbanization on river systems can be seen in four key areas: hydrology, geomorphology, water quality and stream ecology (Ladson et al., 2006). Figure 1 shows the complex relationship between these four areas. While motivated by the desire to minimize impacts on stream ecology, this study focuses specifically on the impacts of hydrology and geomorphology.

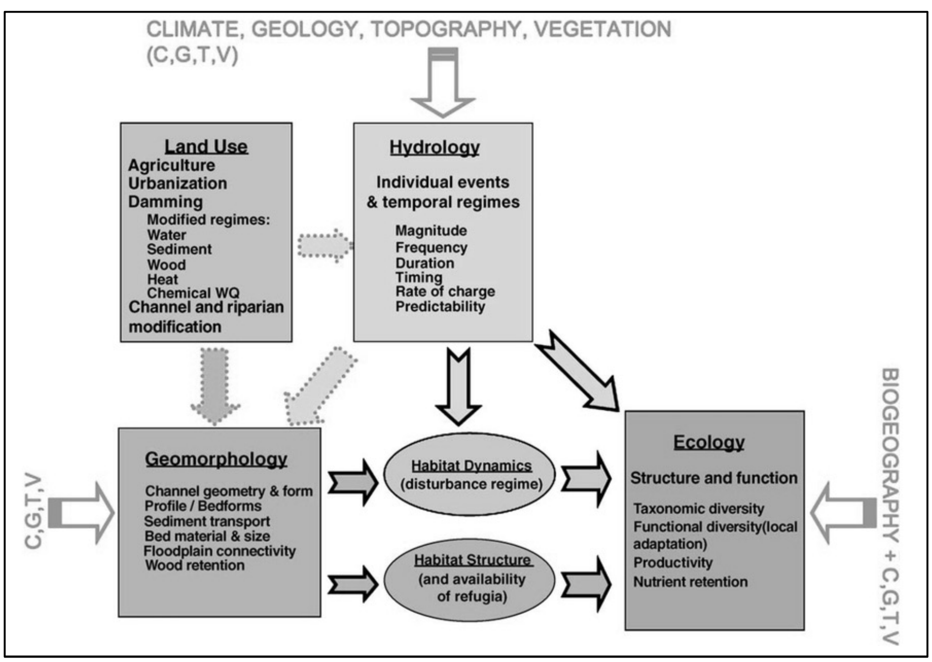


Figure 1: Conceptual diagram of the relationship between hydrology, geomorphology, and ecology and their responses to changes in land use (Poff et al., 2006)

2.1.1 Hydromodification in Urban River Systems

2.1.1.1 Imperviousness

Seemingly the largest focus of all urban river studies is the hydrologic response to increases in impervious land cover. Total imperviousness is strongly correlated with channel degradation (Leopold, 1968; Booth & Jackson, 1997). From a hydrologic perspective, an increase in impervious area results in decreased infiltration, higher runoff and a reduction in the time it takes for runoff to reach the channel (Galster et al., 2008). Increases in impervious land cover have been linked to increased runoff (Leopold, 1968; Dunne & Leopold 1978; McCuen & Moglen, 1988), bed and bank erosion (Wolman, 1967; Booth, 1990), channel incision (Chin, 2006) and loss of biological diversity (Townsend et al., 1997). It has been reported that even low levels of impervious cover, 5-20%, can result in degraded stream channels (Bledsoe, 2002; Poff et al., 2006). There are indications that there may be a threshold level of imperviousness, approximately 15%, beyond which there is a significant increase in the effects of degradation (Moscrip & Montgomery, 1997). Other factors, such as the length of time which a watershed has been urbanized, may also influence the geomorphic state of the channel (Doyle et al., 2000).

2.1.1.2 Storm Event Flows

The impact of urbanization can be seen on the event scale, altering the channel's response to a storm event. Urban streams tend to experience flashier peak flows, a faster rise and recession of flow due to storm runoff, as a result of a reduction in response time and increased total imperviousness (Leopold, 1968; Walsh et al., 2005; Trudeau & Richardson, 2016). Urban streams tend to have greater peak flow values (Baker et al., 2008; Mcguen & Moglen, 1988; Konrad et al., 2005, Poff et al., 2006) as well as increased flow exceedance frequencies (Dunne & Leopold, 1978; Booth & Jackson, 1997; Annable et al., 2012).

It has been shown that urbanization and increasing levels of imperviousness have a greater impact on the increase in peak flow of small, frequent events than in larger infrequent events (Booth, 1991; Konrad et al., 2005; Nehrke & Roesner, 2002). In smaller storms, the increase in peak flow is directly proportional to increased imperviousness (Nehrke & Roesner, 2002). Furthermore, the durations of frequent high flow events are shorter in higher levels of urbanization, indicating an increase in potential streambed disturbance (Konrad et al., 2005).

Annable et al. (2012) suggest that urban streams in comparison to rural streams show no increase in total annual discharge volumes; however, the annual cumulative volume in exceedance of bankfull stage increases in urban streams (Figure 2). This tends to agree with the concept of exceedance of mean annual streamflow, Q_{mean}, where a study by Konrad et al. (2005) showed that urban streams exceed Q_{mean} on less days than their rural counterparts. In general, research agrees that urbanization increases storm runoff volume, resulting in larger peaks, decreased response time, greater volumes conveyed above bankfull and greater frequency of sediment-transporting flows (Booth & Jackson, 1997).

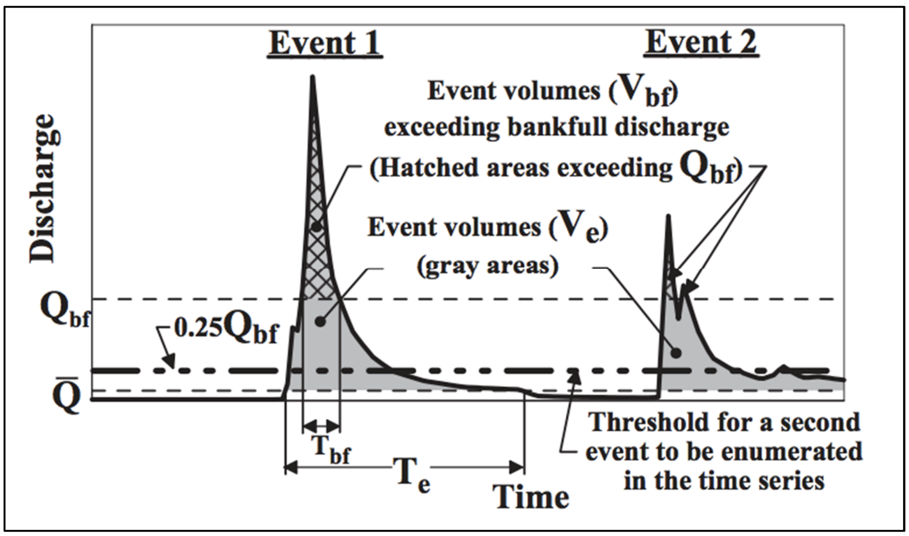


Figure 2: Conceptual hydrograph showing the distinction between 'event volume' and 'event volume only exceeding bankfull'. Although the cumulative V_e is the same between urban and rural streams, the cumulative V_{bf} is larger in urban streams. From Annable et al. (2012).

Many studies have recognized the positive trend between peak flow and erosive potential. However, analysis of the entire hydrograph provides a more complete understanding of the impacts of urbanization on streamflow. Recent studies by Trudeau and Richardson (2015, 2016) focus on the rising limb of the hydrograph and the rate at which flow accelerates toward a peak value. In an urban environment, the event flow acceleration of the rising limb during a storm event is greater than in rural systems as shown in Figure 3 (Trudeau & Richardson, 2016). This serves as evidence to the flashier peak flows seen after urbanization (Mcguen & Moglen, 1988, Leopold 1968). The studies by Trudeau and Richardson (2015, 2016) do not quantify the level of stormwater management present on the catchments studied, which could create great variance in how the hydrograph responds to runoff in an urban setting (McCuen & Moglen 1988).

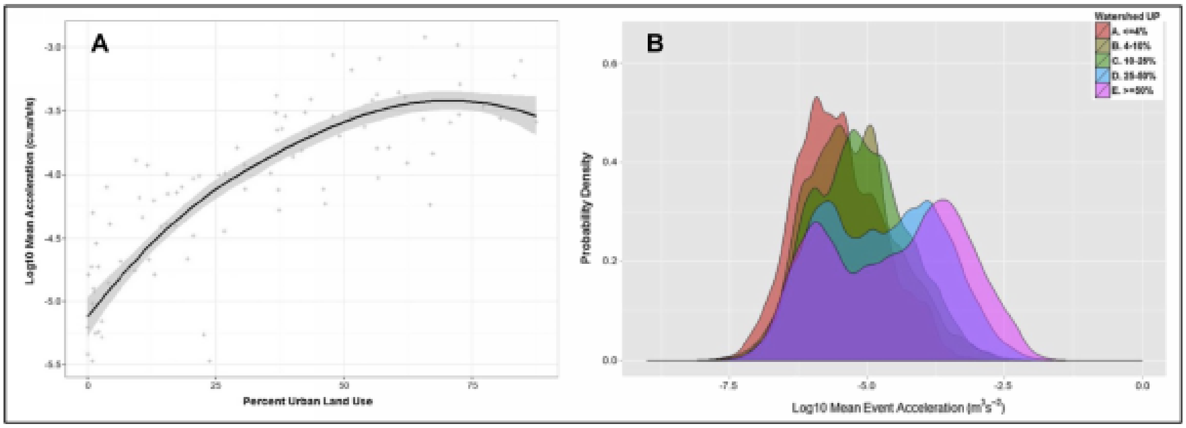


Figure 3: A) Increasing trend of mean event acceleration and percent urban land use. B) The probability distributions of mean event accelerations for varying levels of urban cover (Trudeau & Richardson, 2016).

Rosburg et al. (2017) use the Richards-Baker (RB) flashiness index (Baker et al., 2004) to describe the rapid rise and fall of stream discharge over time experienced in urban rivers.

$$RB = \frac{\sum_{i=1}^{n} |q_i - q_{i-1}|}{\sum_{i=1}^{n} q_i} \qquad (1)$$

The findings suggest an increase in flashiness with urbanization in all four urban watersheds studied similar to other studies (Annable et al., 2012). The study uses daily flow data, which while providing a reliable measure of stream flashiness (Baker et al., 2004), does not provide details on the rising and falling limbs of the storm hydrograph. Rosburg et al. (2017) discuss the likelihood of increased flashiness being tied to the introduction of stormwater conveyance systems. It is unclear as to what extent stormwater management techniques had been introduced in the catchments studied. Stormwater detention facilities can have a large impact on the reduction of flashiness (Booth & Jackson, 1997). However, the cumulative impacts of multiple detention facilities can actually create flooding problems, as peak flows can increase downstream (McCuen, 1974). Goff and Gentry (2006) concluded that a fully developed watershed with detention storage throughout will still experience greater than pre-development flows at certain points within the main channel. There is a lack of research that imposes field measurements downstream of detention facilities to measure cumulative impacts on channel response.

2.1.2 Morphological Adjustment

Lane (1955) first modelled the ability of a river to maintain equilibrium or geomorphic stability with the introduction of the following equation:

$$QS \propto Q_s D_{50} \qquad (2)$$

where Q is discharge, S is channel slope, Q_s is the sediment flux, and D_{50} is the median sediment diameter of the bed material. The equation qualitatively shows that a perturbation in any of the four parameters will cause a shift or response in the others (Figure 4), and that an alluvial channel will stabilize if the streamflow conditions remain consistent for a long time (Lane, 1955; Konrad et al., 2005). The balance of these conditions is linked to the stability of the channel (Church, 2006).

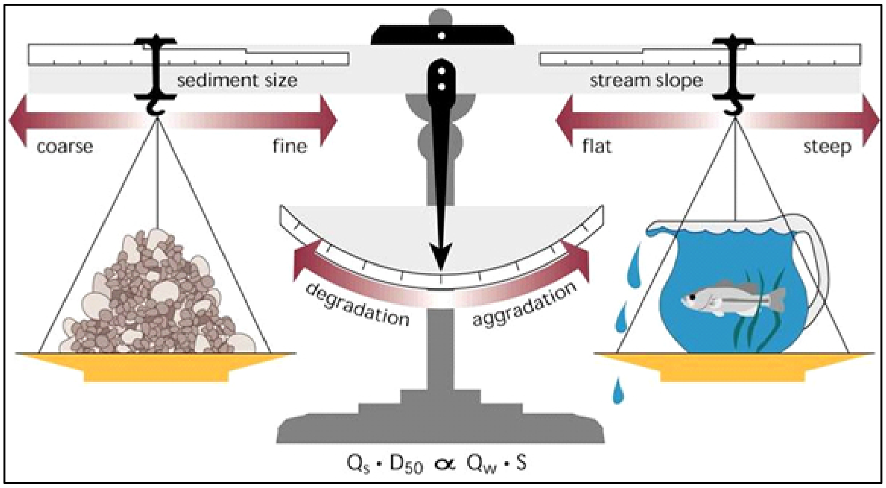


Figure 4: Lane's relationship (1955) presented visually by Rosgen (1996).

In response to increases in urbanization, studies have shown that creeks will undergo a series of morphological adjustments. Typically, increases in peak flow and storm flow volumes result in wider and deeper channels (Dunne & Leopold, 1978; Booth, 1990; Galster et al., 2008). A river's morphology will also respond to changes in sediment supply. During the development phase of an urban area, construction can cause large increases in sediment production, as much as 80% of the basin's yield (Chin, 2006; Fusillo et al., 1977). After development has ceased, there is a decrease of sediment supply from the urbanized watershed due to the hardening of surfaces from land use changes, resulting in an increase of the amount of sediment eroded from the bed and banks of the channel (Doyle et al., 2000; Nelson & Booth 2002; Trimble, 1997; Poff et al., 2006). Additional factors, such as detention ponds, can act as sediment sinks in a watershed further reducing the sediment supply (Poff et al., 2006). A low sediment supply can lead to coarsening of the channel bed, potentially generating areas of immobile sediment (Yager & Schott, 2013).

Much research has been done in the area of determining the most geomorphologically significant flow in a river system, also referred to as the effective discharge. The effective discharge can be defined as the flow rate at which the most work is done in defining the hydraulic geometry of the channel (Leopold et al., 1964). Thus, the identification of bankfull flow became synonymous with effective discharge (Leopold et al., 1964). However, Leopold's work was developed in non-urban channels. A shift in the effective work curve may come as a result of urbanization as well as an increase in sediment transport potential for moderate flow events (MacRae & Rowney, 1992) creating debate as to whether the greatest effective work occurs at bankfull or mid-bank flows (MacRae, 1992). In urban environments, the frequency of midbankfull to bankfull events increases (MacRae, 1997; Annable et al., 2010) and are possibly the most geomorphically significant flows, leading to even greater rates of erosion (MacRae, 1992). This could be a consequence of the incised nature of most urban streams (Chuch, 2006) or in the difficulty of identifying what the bankfull stage is for a channel (Annable et al., 2010). In the development of this conclusion, MacRae (1997) looked at a stream with cohesive bed materials. The application of this understanding to alluvial channels with much larger particle sizes has yet to be studied.

2.1.3 Quasi-equilibrium

Leopold et al. (1964) proposed the idea that urban rivers can achieve a state of quasi-equilibrium after active development in the area has ceased. While studies show that channels can return to a theoretical state of quasi-equilibrium after urbanization (Leopold et al., 1964; Graf, 1977), the stable conditions can be difficult to achieve due to the complex nature of the processes (Chin, 2006). Konrad et al. (2005) suggest even after a period of adjustment urban streams will still undergo an increase in frequency and magnitude of streambed disturbance due to urban streamflow patterns. While gravel beds are generally stable under conditions of moderate duration flows, the increased magnitude of frequent high flow events in urban streams will result in greater streambed disturbance (Konrad et al., 2005).

The question of whether a stream is in a quasi-equilibrium state is important, with channel stability often becoming a definition of success in mitigating urban impacts. Grant et al. (2013) underlined the importance of the notion of equilibrium in regards to geomorphology, stating that while geomorphic processes and forces are not necessarily in equilibrium, the concept provides a reference point from which we can assess system behavior. However, Bledsoe (2002) pointed out that the concept of channel stability is subjective, due to the lack of an accepted definition or standard. The term stability can span disciplines, from geomorphology and bank stability, to channel hydraulics and the failure of culverts and bridges, and even ecology and bed stability as a requirement for habitat for aquatic organisms. Additionally, local context down to the channel reach scale can provide a variable definition of channel stability as certain reaches may have infrastructure concerns or provide habitat for sensitive species. It becomes increasingly important to define what channel stability looks like when trying to devise mitigation strategies, such as those in stormwater management.

2.1.4 Urbanization and Ecological Impacts

In response to growing urban centres and increasing development, many natural channels have experienced the negative effects of the "urban stream syndrome", resulting in major losses of aquatic habitat (Walsh et al. 2005). Even a 10% effective impervious area can result in loss of aquatic-system function (Booth & Jackson, 1997). The physical adjustments that a river undergoes in response to urbanization impact the survival of many aquatic biota.

Poff et al. (1997) list 5 critical components of the flow regime that influence the aquatic ecosystem in the channel: magnitude of flow, flow frequency, duration, timing, and rate of change. Native species have adapted to these natural conditions over time and alterations in any of these components can impact their ability to survive or give unfair advantage to invasive species (Poff et al., 1997). Townsend et al. (1997) found similar results when looking at bed-mobilizing events. The greatest species richness was found with intermediate frequency of bed-mobilizing events, suggesting that greater disturbances reduce biotic richness. Biota tend to respond better to a single large disturbance rather than smaller disturbances that occur at a higher frequency (Konrad et al., 2005; MacRae & Rowney, 1992). Macroinvertebrates have the capacity to respond to channel alterations, and can recover within a year in typical warm water streams (Whitaker et al., 1979). Flows at and above bankfull tend to occur more frequently with increasing urbanization, with sediment mobilizing events occurring multiple times a year (Annable et al., 2012). Therefore, preservation of diverse aquatic ecosystems is best approached through replicating the frequency of bed mobilization found in natural, stable channels (Doyle et al., 2000).

There is little research that focuses onthe ecological implications of hydromodification due to stormwater management controls. Mobley and Culver (2014) looked at optimizing the outlet configurations of stormwater detention facilities in order to minimize the impact on the ecological conditions in the channel. Their work focused on dry ponds, but the conclusions are transferrable; the hydrologic channel response to a storm event plays an important role in ecological stream health.

2.2 Bedload Sediment Transport

River systems are powerful morphological features carving into the landscape, not only conveying water but transporting sediment as well. The sediment transported by a channel is made up of a suspended load and a bedload, with the bedload fraction generally thought to range between 5 and 10 percent. The morphology of an alluvial river, in particular gravel-bed rivers, depends largely on the transport of bed material (Church, 2006). Depending on the frequency of formative flows, the river morphology can be redefined fairly frequently, including multiple times a year in highly urbanized watersheds (Annable et al., 2012). There is a critical link to be further explored between how the bedload fraction is transported and the morphological adjustments of gravel-bed rivers.

Particles resting on the bed of a channel are subject to applied forces from the flow; when these applied forces exceed the resisting forces, particles become mobile. The concept of tractive forces doing work on the bed and banks was first conceived by DuBoys (1879). Bedload particles, once set in motion, will travel by rolling, sliding, or saltation, remaining in contact with the channel bed. During a single storm event, particle travel distances are composed of multiple steps interspersed with periods of rest (Nikora et al., 2002). Einstein (1950) proposed that sediment transport was a stochastic process, and could be determined through statistical analysis. The research of mechanical forces that are at work on particles on a steam bed has been expanded upon over the years. Shear stress, turbulence, near bed variations in flow, and other mechanisms have been proposed as theories for initiation of particle movement.

2.2.1 Particle Mobilization

The stability of an alluvial channel is dependent on the coarse particles which define its geomorphic form. When these particles are mobilized the stability of the channel is decreased. Doyle et al. (2000) argues, “the recurrence of bed mobilization, while the most data intensive, is considered the most robust measure of geomorphic stability of those investigated, as it accounts for eroding forces, resisting forces, and hydrologic frequency of critical events”. There are not many studies that use the particle mobility parameter to assess channel stability (Macvicar et al., 2015), which is of much importance in assessing urban streams.

The initiation of particle motion can be understood through Newton’s first principle, F=ma = m*dV/dt. Rearranged to Fdt = mdV, where Fdt is the impulse and can be used to develop the tractive force method (Malcom & Smallwood, 1977). Shields’ theory (1936) has been used to identify the threshold conditions for the initiation of particle movement. Shields formulated incipient motion of a particle as a dimensionless ratio between bed shear stress and the submerged grain weight per unit area.

$$\boldsymbol{\tau}_* = \frac{\boldsymbol{\tau}_0}{(\boldsymbol{\rho}_s - \boldsymbol{\rho})\mathbf{gD}} \qquad (3)$$

where τ_* is the dimensionless Shields parameter for entrainment of the clasts of size D, ρ_s is the density of sediment, ρ is the density of water (1000 g/m^3), and g is the acceleration due to gravity (9.81 m^2/s).

Incipient motion of coarse particles is complicated by factors such as bed armouring (Church & Hassan, 2002; Yager & Scott, 2013), particle locking, grain sheltering (Church, 2006) and burial in reach-scale features like bars and riffles. Channel bed armouring, bed surface coarsening and areas of immobile sediment can be a common occurrence in urban streams as sediment supply is reduced and the transport capacity remains high (Yager & Scott, 2013). The hiding function proposed by Egiazaroff in 1965, was the first to provide a reasonable approximation of Shields parameter for mixed particle size beds, capable of including the effect of decreased mobility due to increasing grain weight, and increasing mobility due to protrusion of larger particles (Parker, 2008).

$$\frac{\tau_{sci}^*}{\tau_{scg}^*} = F_{hc}\left(\frac{D_i}{D_g}\right) = \left[\frac{\log(19)}{\log(19\frac{D_i}{D_g})}\right]^2 \qquad (4)$$

In the equation, Dg is the geometric mean particle size of the distribution. The hiding function provides a modified critical Shields parameter for particles; however, it has been shown that the function does not work well for very small particles, $D_i/D_g < 0.4$ and for the very coarse, rare particles in a mix (Parker, 2008).

2.2.1.1 Partial Mobility

Bed particles in a gravel bed river can experience partial, selective, and equal mobility. Partial mobility occurs when the coarse fraction of the bed surface material size distribution is not presented in the mobile material (Parker, 2008). Selective mobility occurs when all size particles are mobilized, but the size distribution of the bedload is finer than the bed surface. Equal mobility occurs when all size fractions of the bed material are present in equal proportion in the mobile material. To determine differential mobility conditions, Wilcock and McArdell (1993) suggested indexing the ratio of the boundary shear stress to the shear stress required to move a particular grain size; commonly the 84th percentile of the bed material because it is associated with the coarse tail of the grain size distribution (Venditti et al., 2015). Most gravel-bed rivers exhibit partial mobility during flows below bankfull, but have selective mobility during bankfull flows (Wilcock & McArdell, 1997; Church & Hassan, 2002; Macvicar et al., 2015; Phillips & Jerolmack, 2014; Venditti et al., 2015). Research has given evidence that size-selective mobility holds for gravel-bed rivers (Ashworth & Ferguson, 1989; Church & Hassan, 2002).

2.2.2 Particle Travel Distance

Important in the measure of channel stability is not only whether a particle has moved but also how far it travels before it comes to a rest. Particle travel distance is essential to quantifying bulk sediment transport and erosion rates. Past research has focused on creating relationships between travel distance and particle size (Church & Hassan, 1992), flow characteristics (Church & Hassan, 2002), near bed turbulence (Yager & Schott, 2013), and channel morphology (Pryce & Ashmore, 2003; Lamarre & Roy, 2008).

Church and Hassan (1992) propose assessing travel distance by grouping particles by size class and comparing mean travel distances to the mean travel distance of particles in the median size class. They developed the following equation,

$$\frac{\overline{L_i}}{\overline{L_{D50}}} = 1.77\left(1 - \log\frac{\overline{D_i}}{D_{50}}\right)^{1.35} \tag{5}$$

where $\overline{L_i}$ is the mean travel distance for particles in a size class i, $\overline{D_i}$ is the corresponding mean particle size, and $\overline{L_{D50}}$ is the geometric mean travel distance of particles in the median size class.

It is generally agreed that there is a non-linear relationship between scaled movement distance and scaled particle size (Church & Hassan; 1992). Macvicar et al. (2015) used the proposed relationship of Church and Hassan in a study with a large number of stones (n = 443) over a wide range of grain sizes. However, one of the limitations was the small sample size for each size class of tracer particles. Vasquez-Tarrio and Menendez-Duarte (2014) found the relationship proposed by Church and Hassan (1992) was generally a good fit in a study using 1,960 tracers in 3 size classes to develop the relationship between particle size and travel distance.

Similar to mobility rates, a variety of hydrometrics have been found to scale with particle travel distance. Studies have correlated particle travel distances with shear stress (Wong et al., 2007; Phillips & Jerolmak, 2014), excess stream power (Schneider et al., 2014; Houbrechts et al., 2015), cumulative excess stream power (Hassan et al., 1992; Lamarre and Roy, 2008; Schneider et al., 2014), and dimensionless impulse (Phillips et al., 2013).

Pryce and Ashmore (2003) state longer travel distances are likely defined in part by the channel morphology and not solely the hydraulic characteristics of the flow and that the relative influence of these two factors is based on the magnitude of the flow event. The control of bed morphology on particle movement has been seen in studies by Haschenburger (2013) in large floods.

2.2.3 Relating hydrometrics with bedload sediment transport

2.2.3.1 Peak Excess Shear

Channel competence is defined as the ability of the stream flow to mobilize a particle of a given size quantified by the Shields number (Church, 2006). Researchers argue excess shear is better suited for channels having variable substrate size, which is common for natural gravel-bed channels (Doyle et al., 2000). Others believe that sediment motion cannot be accurately predicted by exceedance of critical stress (Yager & Scott, 2013).

Erosion rate was related to excess shear stress as described by Foster et al. (1977) with the following equation:

$$\boldsymbol{\varepsilon = k_d(\tau_O - \tau_C)^a} \qquad (6)$$

where, ε = erosion rate (m/s), k_d = erodibility coefficient (m^3/N•s), a = exponent, typically 1.5, τ_o = applied shear stress on the soil boundary (Pa) and τ_c = critical shear stress (Pa). Researchers have used this concept of excess shear stress to generate estimates of erosion potential for urban areas before and after development (Pomeroy et al., 2008). If erosion rates are a function of excess shear stress, by extension it would appear that bedload sediment transport could be governed by the same principle. Pomeroy et al. (2008) looked at the impact of SWM controls on erosion potential. Their study used hydrologic models to generate measures of the change in erosion potential between pre and post development. Their estimations of erosion rates were based on Foster's equation and they used a critical shear stress from empirical data for colloidal alluvial silts. However, their study did not include direct field measurements of erosion potential or sediment transport undertaken.

2.2.3.2 Specific Stream Power

Stream power is a measure of the energy used to cause geomorphological changes within a channel (Bagnold, 1966). The following equation defines specific stream power:

$$\boldsymbol{\omega = \frac{\gamma QS}{w}} \qquad (7)$$

where γ = specific weight of water, Q = dominant discharge, S = channel slope and w = channel width. Researchers have used a measure of maximum specific stream power to predict particle travel distances (Hassan & Church, 1992; Houbrechts et al., 2015). Similar to peak excess shear, the concept of maximum specific steam power does not account for the duration of competent flow capable of transporting sediment.

2.2.3.3 Duration of Competent Flow

The geomorphic effectiveness of a given channel discharge is related to its duration and its magnitude relative to the cumulative discharge conveyed by the channel (Wolman & Miller, 1960). This relates to the geomorphically significant flows identified by Leopold et al. (1964), and the understanding that frequent small magnitude floods transport most of the sediment in a channel because of the cumulative duration of these flows. Konrad et al. (2005) found that flow duration, instead of frequency, may be a better index of geomorphically effective flows in gravel bed channels because over time, sediment transport exhausts the supply of mobile particles from the channel bed and banks.

2.2.3.4 Cumulative Effective Work

The work being done by the flow on the channel bed can be integrated over time using the hydrometric of cumulative effective work index (CVC, 2012). The cumulative effective work index, Wi, is measured in units of N/m, and is calculated as:

$$\boldsymbol{Wi} = \sum(\boldsymbol{\tau_i} - \boldsymbol{\tau_c})\boldsymbol{V}\Delta\boldsymbol{t} \qquad \textbf{(8)}$$

where τ_c is the critical shear stress for the D_{50} size class and V is the mean channel velocity for that time step, Δt. The measure of Wi is similar in nature to measures of cumulative excess stream power; however, some regulatory authorities in Ontario such as the Credit Valley Conservation Authority (CVC, 2012) state that it is the preferred measure for assessing erosion because the velocity rises with increasing storm flows making it more sensitive to extreme floods, which researchers have shown is in agreement with predictions of sediment transport (Garcia, 2008).

2.2.3.5 Impulse

It has been shown that particle entrainment is related to both the magnitude of the mobilizing forces and the duration of competent flow experienced. Another time integrated metric that has been related to thresholds of particle entrainment is impulse, defined as the product of shear stress magnitude and duration (Diplas et al., 2008). In order to quantify the time-integrated fluid momentum in excess of threshold, Phillips et al. (2013) use a similar concept, defining the dimensionless impulse as:

$$\mathbf{I}_* = \int_{t_s}^{t_f} \frac{(U_* - U_{*C})dt}{D_{50}}, U_* > U_{*C} \qquad (9)$$

where U_* = the shear velocity (m/s) and U_*c = the critical shear velocity. The equation is valid under the assumption of normal flow conditions. Here t_s, represents the start of a flood, and t_f represents the end of a flood of interest. The integral is only calculated over the record of $U_* > U_*c$, where the shear velocity is above critical, as sub-threshold flows do not transport sediment (Booth & Jackson, 1997).

2.2.3.6 Excess flow Energy Expenditure

Other research has looked at similar time integrated measures such as total excess flow energy expenditure (Haschenburger, 2013; Papangelakis & Hassan, 2016). Following the work of Haschenburger (2013), total excess flow energy is defined as:

$$\Omega_T = \rho g S \int_{t_s}^{t_f} (Q - Q_c)\, dt \qquad (10)$$

where Q is the channel discharge and Q_c is the critical discharge for particle entrainment. Haschenburger (2013) found a power function best described the relation between particle travel distances and excess flow energy expenditure in a study on Carnation Creek, in a bar-riffle-pool morphology.

2.2.3.7 Cumulative Excess Shear Stress

Another time integrated hydrometric, is cumulative excess shear stress (CESS). DeVries (2000) defines CESS as:

$$\mathbf{CESS} = \int_{t_s}^{t_f} \frac{(\tau - \tau_{ci})^{1.5}}{D_i p_i} dt \qquad (11)$$

where τ_{ci} is the critical shear stress for particle size D_i, comprising $100p_i$% of the surface grain size distribution. Figure 5 shows conceptually the calculation of CESS for a specific storm event.

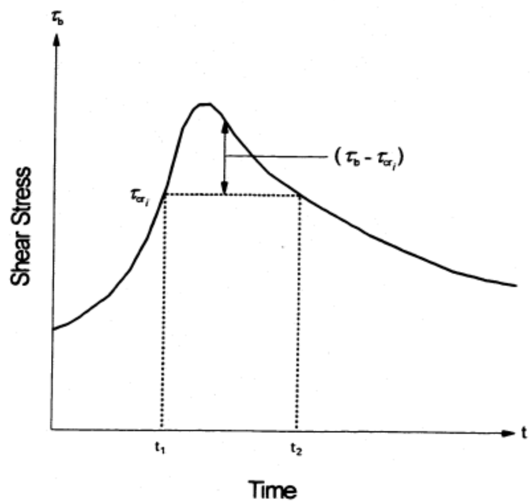


Figure 5. A schematic of the excess shear stress, shown for a period of time, t_1 to t_2, where the threshold shear stress is exceeded for the particular grain size (DeVries, 2000).

DeVries (2000) hypothesized that particle travel distance was a linear function of cumulative excess shear stress for particles experiencing full mobility and non-linear for partial mobility. In his field studies he proposed the following equation, fit to data from two field sites:

$$\mathbf{L_{xi}} = \mathbf{0.0001CESS_i} + \mathbf{34.1}\left(\mathbf{1} - \left(\mathbf{0.0000333CESS_i} + \mathbf{1}\right)^{-\mathbf{5}}\right) \qquad (12)$$

where L_{xi} is the travel distance for a specific particle size and $CESS_i$ is the corresponding cumulative excess shear stress for the specific particle size. Further flume experiments by Devries and others developed a linear relationship between the time integrated excess shear stress and the mean particle travel distance, shown in Figure 6.

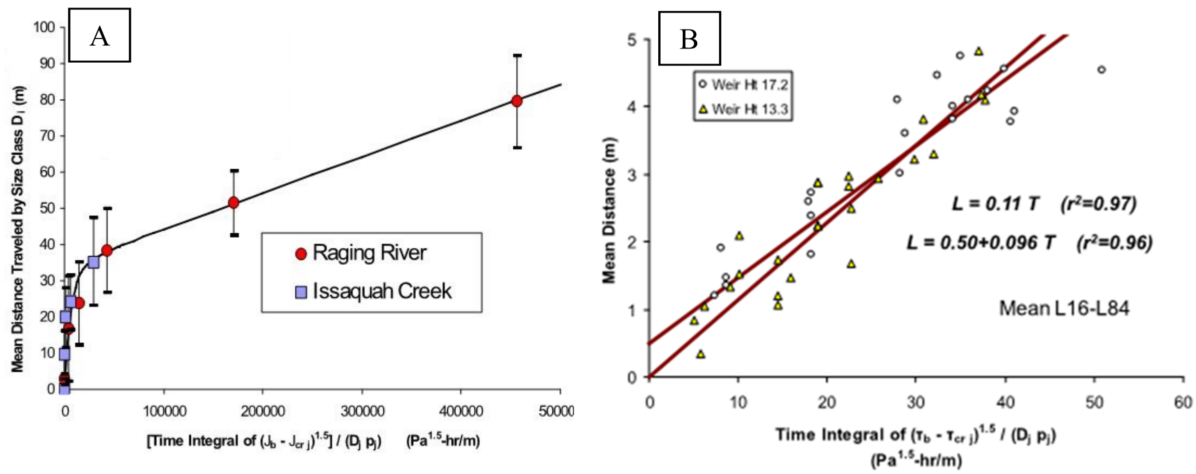


Figure 6: Cumulative excess shear stress versus the mean travel distance of tracer particles. A) Results from a field study. B) Laboratory flume experiment with results for particles in the 16 to 84 percentile of the size distribution. Taken from a poster presented by DeVries et al. at AGU 2006.

2.2.4 Measuring Bedload Sediment Transport using Tracers

There are numerous methods for measuring bedload sediment transport within a river, both direct and indirect. Eularian methods, such as Helley-Smith samplers and bedload traps, measure the volumetric loading at a given location over time. Lagrangian methods such as tracer particles track particle displacements along the extent of a river. This study uses the method of tracers to measure particle movement to infer bedload transport. The tracking of individual tracer particles is a non-intrusive method that does not alter the flow conditions or the transport of sediment, such as a bedload trap. Additionally, while bedload traps can overfill in large events (Sear et al., 2003), tracer particles remain a feasible method for tracking any event large enough to initiate particle mobility. The tracer method has evolved over time, from painted stones, to magnetic stones to active transmitters and finally to passive integrated transponders (PIT) (Nichols, 2004). The use of Radio frequency identification (RFID) for tracking stones with PIT tags was outlined in detail in a methods paper by Lamarre et al. (2005), outlining the advantages and limitations of the technology.

2.2.4.1 Radio Frequency Identification Passive Integrated Transponders

The use of RFID PIT tags to study particle movement in alluvial systems is an attractive method due to its many advantages. The PIT tag is a small glass tube containing a coil of wire and remains passive until it is activated by an electromagnetic signal. An alternating electric current is passed through a second coil of wire, the antenna, creating an alternating magnetic field. Based on the number of coils and strength of the current, the strength of this magnetic field can be adjusted, thus adjusting the distance required to activate the PIT tag (Lamarre et al., 2005). The PIT tag can be encrypted with a unique number allowing the tracking antenna to identify each individual particle with its unique code, referred to as RFID. The PIT tag is designed to withstand vibrations and shock and is insensitive to temperature changes and humidity (Lamarre et al., 2005). The tags are relatively inexpensive, and come in a variety of sizes, the most common ranging from 8 to 23 mm. The PIT tag is inactive and does not require a battery, having an estimated life span of approximately 50 years (Allan et al., 2006).

2.2.4.2 Limitations of RFID technology

Limitations of the RFID technology can be seen where tracer particles experience deep burial and in channels with very high sediment transport rates. Channels with high scour and a deep active layer can make it difficult to recover tagged particles (Lamarre et al., 2005). The transmission signal of the antenna has a limited range, often between 0.5 and 1.0 m depending on the set up and equipment being used. If a particle is buried deeper than the range of the antenna, the PIT tag will not be activated and the RFID reader will not locate the particle. The interference of grains on the detection limits of the antennae is of little concern based on the work of Schneider et al. (2010) who found that when detecting buried PIT tags, water and sediment have only a minor shielding effect. Other limitations include channels with high transportation rates where particles can travel great distances during a single minor storm event. In these conditions, the length of travel can be a limitation if the particle travels beyond the study limits in a single event (Lamarre et al., 2005). It also limits the time scale of the monitoring, as the tracer particles do not remain in the study area for very long. Particle locking or imbrication can drastically vary the results from expected outcomes.

2.2.4.3 Seeding location and Tracer Quantity

An important variable in a tracer study is the determination of location for injection of the tracer particles. Sear (1996) performed a 2-way ANOVA test to determine the effects of injection position on travel distance and burial depth. Unsurprisingly, the tests revealed that the injection location of the tracer particle had a significant effect on both the travel distance and burial depth. Many studies seed stones loosely on the surface (Houbrechts et al., 2015) and others adopt a method of replacing existing stones with tagged ones in an effort to have a more natural initial position (Vasquez-Tarrio &

Menendez-Duarte, 2014). Often the stones are artificially mobile during the first mobilizing event and so this event is not used in further analysis (Lamarre & Roy, 2008).

Another key factor to be considered in a tracer study is deciding what portion of the grain distribution will be tracked. Lamarre et al. (2005) tagged stones in the middle of the size distribution, as they were limited by the ability to insert a PIT tag into smaller stones and the difficulty in transporting larger stones to the laboratory for drilling. The disadvantage of tagging smaller particles is the increased probability of travel results in particles travelling beyond the study limits rapidly, limiting the temporal scale of the study.

The number of particles tagged can have an important outcome on a study. Gravel-bed rivers can express large variability in bed forms and clast orientation which can effect particle movement, drastically changing the mechanics of sediment transport at the grain scale. In order to overcome this, a large number of particles should be tagged to provide opportunity to gather sufficient data from which meaningful conclusions can be drawn (MacVicar, 2015).

2.2.5 Estimating Bulk Sediment transport

There are various methods for calculating bulk sediment transport. Church (2006) refers to the process of estimating sediment transport using morphological change as the inverse problem.

2.2.5.1 Spatial Integration Method

Included in these inverse methods of sediment transport calculations is the spatial integration method (SIM) which utilizes the following equation: (Hassan et al., 1991; Haschenburger & Church, 1998):

$$\mathbf{Q_b = v_b d_s w_s (1 - \lambda) \rho_s} \qquad (13)$$

where Q_b is the bulk sediment transport (kg/s), v_b is the virtual velocity of particles (m/s), d_s is the active layer thickness (m), w_s is the active layer width (m), λ is the porosity of sediment and ρ_s is the density of sediment (kg/m^3). The three parameters that need to be measured in the calculation of bulk sediment transport are virtual velocity, active layer thickness (DeVries, 2002) and active layer width (Ashiq, 1999). Each of these parameters has been studied individually as well as collectively in an effort to properly estimate sediment transport.

Virtual velocity of a particle is the particle path length divided by the time of travel. There are varying opinions and methods of measuring the virtual velocity and numerous studies focusing on solely particle path length (Pryce & Ashmore, 2003). In some studies, the virtual velocity is determined by taking the total distance travelled over many events and dividing by the total time, including the periods of rest (Bradley & Tucker, 2012). In other cases, the virtual velocity is calculated over a single event, as the distance travelled from particle entrainment to final deposition, often including multiple steps and rests, and dividing by the duration of competent flow (Haschenburger & Church, 1998; Milan, 2013; Houbrechts et al., 2015).

Active layer thickness is the depth of the substrate which can become active during an event and is also variable in its definition and measure. It can be estimated using the burial of tracer clasts (Hassan & Ergenzinger, 2003) or scour pins (Laronne et al., 1994). Techniques for using the PIT tags to measure active layer thickness include approximating it with the maximum burial depth (Sear et al., 2003), or estimating it as twice the D_{90} (DeVries, 2003). When using PIT tags to estimate bulk sediment transport it must be assumed that the active layer is well represented by the tracers in the study (Sear et al., 2003). Laronne et al. (1992) argue that the active layer is consistently underestimated in particle tracer studies; however, a number of studies have shown that the estimated active layer depth is comparable between tracer studies and scour chain measurements (Hassan, 1990; DeVries, 2003). Additionally, error can be introduced if the active layer depth is widely variable across the study reach (DeVries, 2003). Haschenburger and Church (1998) developed a method to estimate the uncertainty in using the SIM method to calculate bulk sediment transport, indicating the percentage of error associated with each of the measured parameters. The method of Haschenburger and Church (1998) suggests that the largest percentage of error in estimating sediment transport rates is derived from estimating the virtual velocity.

Sear et al. (2003) used aluminum passive tracers to assess the accuracy of the spatial integration method in a gravel-bed stream. More recent studies have used PIT tags in place of aluminum tracers; however, in this particular paper, they compared the SIM method with experimental results of bulk sediment transport using pit traps. The range of estimates for the sediment transport rate varied by up to three orders of magnitude depending on the assumptions made in the SIM method (Sear et al., 2003).

Vasquez-Tarrio and Menendez-Duarte (2014) use a tracer study to evaluate nine bedload equations in a coarse-bed mountain stream. The results found that the bedload equations reported higher estimates of transport than those determined by the tracers. Although this study looked at a mountain stream, the low sediment supply parallels with many urban creek studies.

2.2.6 Particle tracking in urban environments

While the method of particle tracking as a means of measuring bedload sediment transport has been widely practiced, both in laboratory and field experiments, few studies focus on urban environments.

Field data gathered in urban environments needs to be fitted to these theoretical models to provide confidence in applying these models to urban systems. A large enough sample size must be recovered to have meaningful results when fitting these models (MacVicar et al., 2015). Challenges in particle tracking through urban environments include flashier urban storm response, higher frequency of formative flows, and limited sediment supply which can lead to bed armouring.

Urban streams have been said to have more consistent bedload movement (Annable et al., 2012), but this does not necessarily equate to larger bulk transport volumes over time. In applying the spatial integration method to an urban setting, the hope is to establish an indicator of stream stability in order to measure the effectiveness of stormwater management practices in the study. Further particle tracking studies are required in urban channels to identify whether urban channels exhibit similar particle transport mechanics as other channels. Additionally, these studies will provide insight into whether urban channels have the ability to achieve a state of quasi-equilibrium and the time frame necessary to achieve this. Further, by conducting a field study in a catchment heavily influenced by stormwater controls, the impact of these controls on bedload sediment transport can be monitored against theoretical and laboratory conclusions.

2.3 Stormwater Management

The practice of stormwater management has greatly evolved over the past half century. Modern practices have shifted from a focus on conveyance and flood control to a more holistic approach of mimicking the natural retention properties of pre-development conditions. Stormwater management is a constantly evolving set of practices designed to mitigate the negative impacts of urbanization on natural channels. The Wet Weather Flow Management Master Plan (2006) of the City of Toronto, states the goal of stormwater management quite clearly: “To reduce, and ultimately eliminate the adverse effects of wet weather flow on the built and natural environment in a timely and sustainable manner, and to achieve a measurable improvement in ecosystem health of the watersheds.”

Focusing on Ontario, in March 2003, the Ministry of the Environment implemented the Stormwater Management and Planning Design Manual, a guideline for stormwater management in the province (MOE, 2003). Building upon these guidelines, many municipalities have constructed regulations and approaches defining how developers would implement stormwater management practices within their jurisdictions. For example, the City of Toronto implemented the Wet Weather Flow Management Guideline (WWFMG) in 2006 to provide detailed design parameters for stormwater management.

In general, stormwater management covers three areas of concern, water quantity, water quality and water balance. In large part it deals with standards for new developments and how the control of storm runoff will be addressed. The goals of a stormwater management plan could include flood control, peak flow attenuation, volume control, water quality, and erosion protection of downstream watercourses. A best management practice (BMP) refers to any practice or facility implemented based on current best technologies and feasible practices. BMPs may change over time as new technologies emerge and new practices are developed.

2.3.1 Stormwater Management controls

Common to all urban SWM strategies are the use of storm sewers that convey runoff efficiently out of the urban centre. In doing so, storm sewers reduce the lag time of the channel response to a storm event (Hirsch et al., 1990, Paul & Meyer, 2001). The reduction in lag time due to stormwater conveyance is likely the main factor behind the correlation between imperviousness and channel degradation (Walsh et al., 2005). In order to combat the increased conveyance efficiency, stormwater management moved towards the implementation of BMPs that increase lag time and reduce peak flows.

SWM controls can come in many forms usually broken down into the following categories: lot level, conveyance controls, and end-of-pipe facilities (MOE, 2003). Lot level controls are designed to target runoff before it leaves the site. Examples of lot level and conveyance controls include rooftop storage, reduced lot grading, and grassed swales. The added advantage of these controls over other SWM controls is the ability to reduce runoff volumes through water reuse or infiltration. Low Impact Development (LID) typically promotes reversing the increased storm runoff volumes which are a common impact of urbanization.

End-of-pipe controls are the last BMP in the treatment train approach and will be much larger in capacity and can receive runoff from large areas. These facilities may be built downstream of other controls or as a single treatment facility. Examples of end-of-pipe facilities include wet ponds, wetlands, and infiltration galleries.

2.3.1.1 Detention Facilities

In Ontario, detention facilities, also referred to as SWM ponds, are the most commonly used end-of-pipe stormwater management facility (MOE, 2003). A SWM detention facility/pond is often designed to return post-development peak flows to pre-development flow rates, a technique commonly referred to as peak shaving (Baker et al., 2008). The storage is provided in the form of either an online detention facility or an offline facility. An online detention feature creates a backwater effect with a controlled release providing storage directly in the channel and adjacent floodplains. The downstream impacts of an online facility can be much more prevalent than an offline facility due to the control over the entire upstream contribution. The MOE (2003) discourages the use of online facilities due to concerns for wildlife movement and fish passage. An offline detention facility provides a storage volume for a given contributing drainage area of the channel. The offline facility provides a storage volume and outlet control that releases stormwater at a single point into the receiving channel. In highly urban areas, these offline facilities often take the place of headwater tributaries that have been replaced with sewer conveyance systems.

2.3.2 Peak shaving

The concept of reducing the peak flow seen in urban hydrographs is referred to as peak shaving (Baker et al., 2008). This practice reduces peak flow in the receiving channel by providing a detention feature, such as a wet pond, to store large volumes of runoff before releasing them at a slower rate over an extended period of time (Figure 7). There are various strategies in designing detention facilities in order to achieve different objectives. A detention facility that controls post-development flows to pre-development levels for the two to 100-year return period storms is commonly referred to as the zero runoff increase (ZRI) approach. Such facilities are ineffective in controlling erosion, due to discharging erosive flows for a longer duration and at an increased frequency, while the outlet fails to attenuate smaller, frequent storms (Roesner et. al, 2001). The ZRI facility lacks the ability to reduce runoff volume and the frequency at which erosion events occur (McRae, 1992). Studies have shown that stormwater management facilities can actually increase the duration of erosive discharges in the receiving channel (Baker et al. 2008; Bledsoe, 2002) and more specifically that peak attenuation of the two-year return period storm can exacerbate downstream erosion (McCuen, 1979; McCuen & Moglen, 1988; MacRae, 1997). In combination with increased runoff volumes due to urbanization, even when peak flows match pre-development flows these flows extend over a longer duration increasing the total impulse (Figure 8), resulting in increased erosion (McCuen & Moglen, 1988).

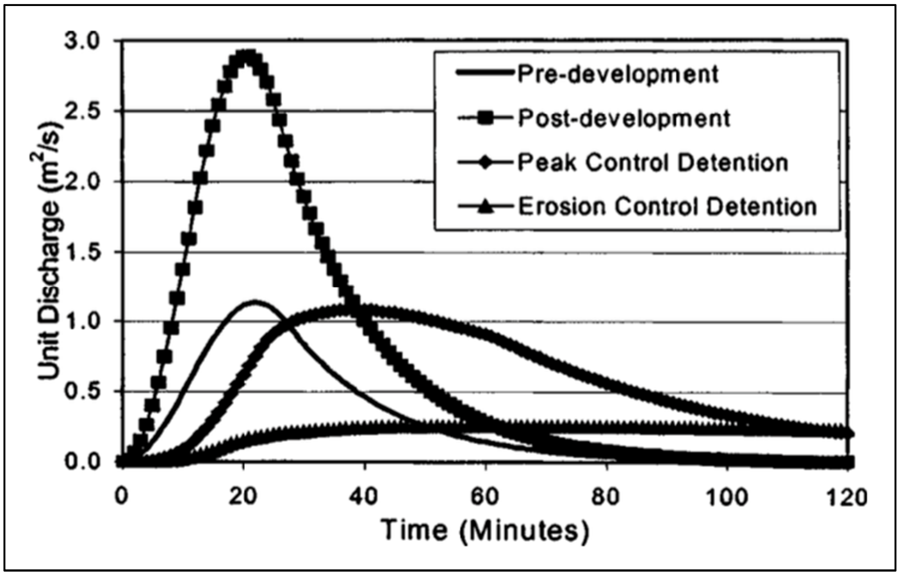


Figure 7: Comparison of measured pre-development and post-development hydrographs and modelled peak control and erosion control SWM hydrographs. From Bledsoe (2002).

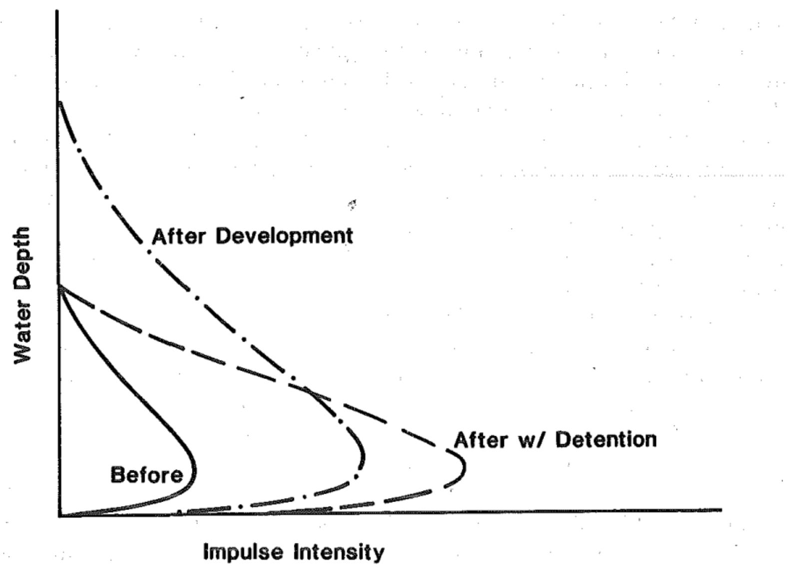


Figure 8: Impulse intensity vs. water depth. Taken from McCuen & Moglen (1988).

2.3.3 Multi Criteria Approach

In Ontario, policies have included an erosion control measure, based on a hydrologic approach of controlling the runoff volume generated from a 25 mm storm event and releasing it over a period of 24 to 48 hours (Figure 9c). It provides a greater control than the ZRI method, providing storage volume and time of retention up to two times greater (McRae, 1992). This approach does not account for the boundary material of the receiving channel (MacRae & Rowney, 1992). McRae (1993) proposes a method of distributed runoff control, with the intention of minimizing channel erosion by maintaining the erosion potential of the channel boundary materials in the pre-development conditions (Figure 9d). Distributed runoff control remains largely theoretical, requiring a field assessment in the pre-development condition to assess the hydraulic stress and erosion potential of channel boundary materials (Baker et al., 2008).

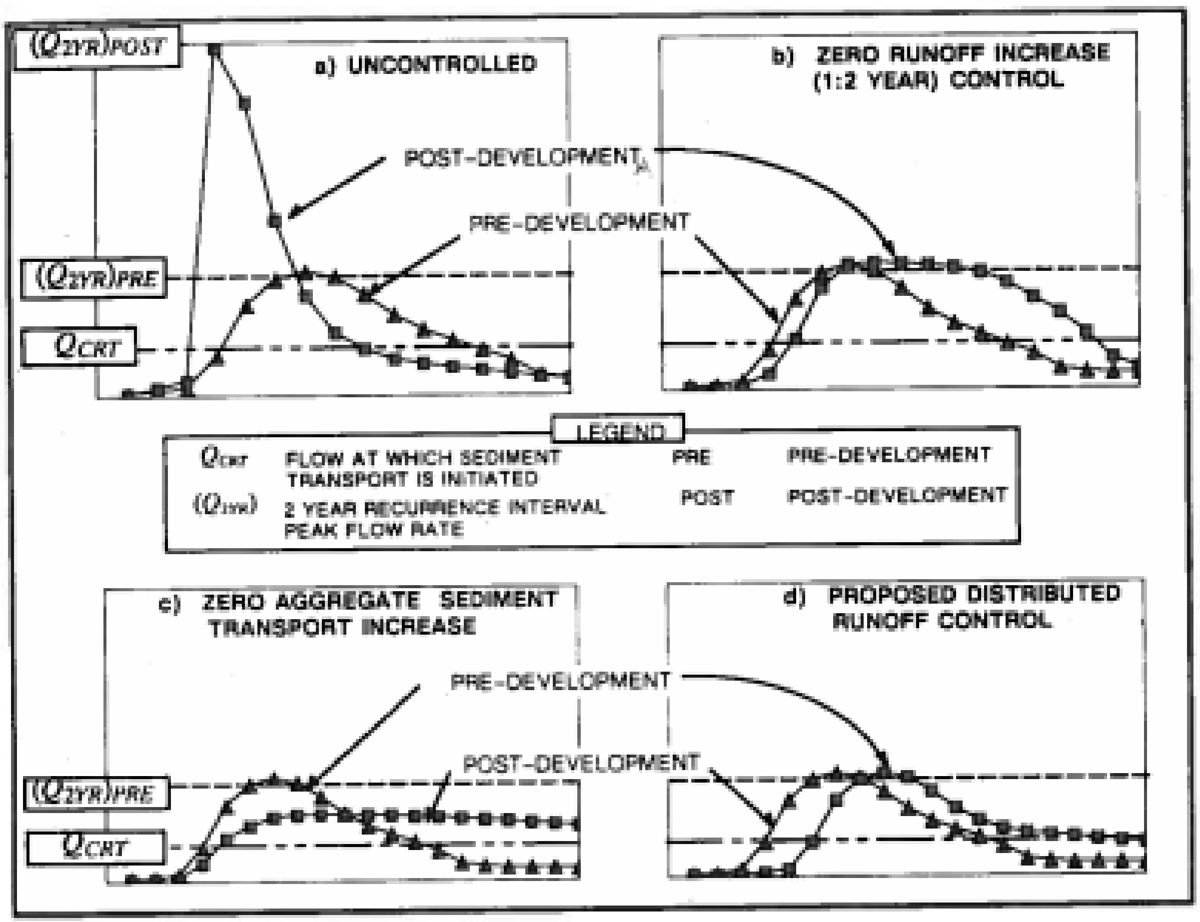


Figure 9: Visualization of 2 year return storm hydrographs for various forms of stormwater management controls. Taken from MacRae & Rowney (1992).

The impact of stormwater detention on the flow characteristics of the channel is directly related to the stability of the channel and can actually increase the potential for bedload sediment transport (McCuen & Moglen, 1988). There is a lack of field measured data on the movement of bedload particles as a result of the increase in impulse due to SWM features.

2.3.3 The Cumulative effects of multiple SWM detention features

McCuen (1974) was one of the first to recognize the cumulative downstream impacts of multiple detention facilities in a watershed. Research has shown that while controlling peak flow at the outlet of detention facilities, peak flow may not be controlled at all points within the downstream channel due to the cumulative effect of timing and magnitude (McCuen, 1974; Goff & Gentry, 2006). Goff and Gentry (2006) also found that the effectiveness of detention decreased with an increasing percentage of development within the watershed. Bell et al. (2016) investigated the overall impact of stormwater management mitigation on various instream channel response metrics. Their data showed that metrics which included information about stormwater control measures did not appear as primary predictors of hydrologic response, suggesting that these stormwater controls were insufficient in their influence on watershed scale hydrologic response. Recent work has seen the development of hydrologic models to measure and simulate the channel response of these complex stormwater management networks (Beck et al., 2017). Overall, field and modelling data on the cumulative impacts of stormwater detention facilities and their relation to erosion potential in the form of bedload sediment transport is scarce.

2.4 Summary of Research Gaps

The research presented here highlights the current knowledge and understanding of the dynamics of bedload sediment transport in urban rivers and the complex nature of hydromodification due to stormwater management systems. There is a general understanding of channel response in urban settings, can be complicated by the presence of SWM features (Annable et al., 2012). Based on the literature reviewed the following gaps have been identified:

i) A lack of field data measuring bedload sediment transport in urban channels with stormwater management controls, in particular detention facilities

ii) A poor understanding of the cumulative effects of a SWM system on the hydrology and sediment transport in the receiving channel

iii) A general inability to predict the marginal impacts of different SWM best management practices, in particular end-of-pipe designs such as detention facilities and volume reduction structures

A recent review by Hawley et al. (2013) summarized the current research need by stating that, "…having an improved understanding of the mechanisms by which stormwater management influences channel structure is imperative, such that policy may be more informed by fluvial process and may have a greater probability of positively affecting stream integrity."

Chapter 3
Methods

3.1 Site Description

3.1.1 Site selection

Morningside Creek was selected as one of three creeks in part of a broad study on sediment transport in urban rivers. The site was selected based on its similarities to the neighbouring sites in regards to its longitudinal slope, bed surface particle size distribution, and distance from Lake Ontario. Morningside Creek is a tributary of the Rouge River, located in the City of Toronto, Ontario, Canada (Figure 10). The development of the Morningside Creek catchment happened gradually, beginning in the 1980s. The drainage area of Morningside Creek is 21.1 km^2 and is approximately 45% impervious cover, with land use being predominantly suburban housing developments and institutions. The headwaters begin in Markham, Ontario, just north of Steeles Avenue.

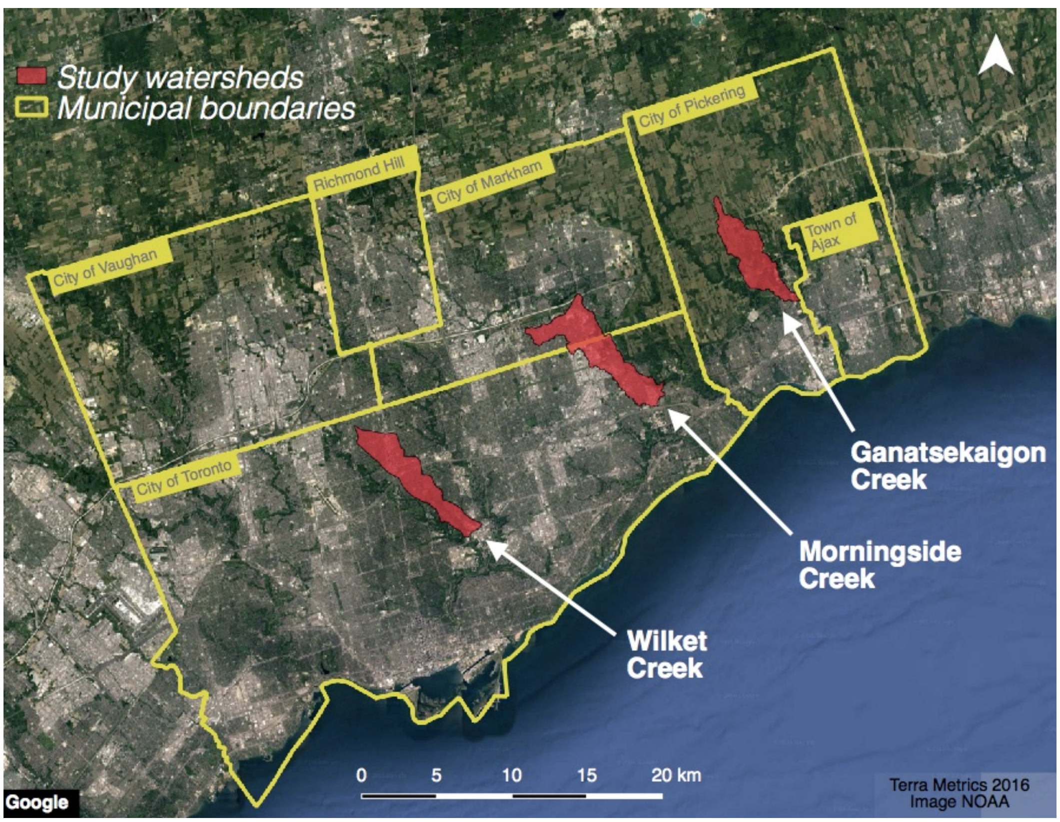


Figure 10: Location map showing the study catchments included in the broader project scope (Figure credit: E. Papangelakis)

Morningside Creek has been geomorphically altered over the years as land areas adjacent to the creek underwent development (MESP, 1999). A restoration project was undertaken on the middle reach of Morningside Creek, from the Tapscott Diversion structure to just upstream of the railway crossing (Figure 11). The restoration consisted of natural channel design and the lowering of the channel invert (MESP, 1999). The main objectives of the restoration project were to increase channel capacity, in order to ensure the conveyance of bankfull flows and sediment conveyance rather than deposition, and to ensure the dissipation of high flow energy onto the floodplain (Schaeffers, 1999). The study reach selected is downstream of this restored reach.

The ecological health of Morningside Creek has been the focus of recent studies. According to the Rouge Fisheries Management Plan (2011), surveys during the years of 2007 to 2009 noted a decline in the abundance of Redside Dace. Redside Dace was added to the Species at Risk in Ontario List in 2009 (MNRF, 2014) and is currently classified as an endangered species with its habitats targeted for protection. Redside Dace spawn in streams with gravel bars and riffles, with faster flowing water and larger particles (MNR, 2011). Beavers are common in the watershed and tend to build channel-spanning dams that impact both water and sediment flows.

3.1.2 Hydrology and Geomorphology

The drainage area of Morningside Creek is part of the Great Lakes Basin, with typical annual precipitation totaling 793mm[a]/ 840 mm[b], (a-Stat Can, 2006;b-WWFMG, 2006) mostly as rainfall between the months of May and November. The underlying surficial geology is a predominantly sandy silt till of the Newmarket Till (Sharpe et al., 1997). The study reach is 200 m in length, approximately 2 km upstream of the confluence between Morningside Creek and the Rouge River, with a contributing runoff area of approximately 17.1 km^2. A forested undeveloped buffer exists on both banks of the channel, with the creek running between two steep sloping valley walls at a slope of roughly 1H:1V. The reach morphology is best described as riffle-pool dominated with a longitudinal gradient of 1.02%. The bankfull width and depth of a typical section were measured as 6.5 m and 0.6 m respectively. Figure 12 shows the detailed cross-sections that were surveyed.

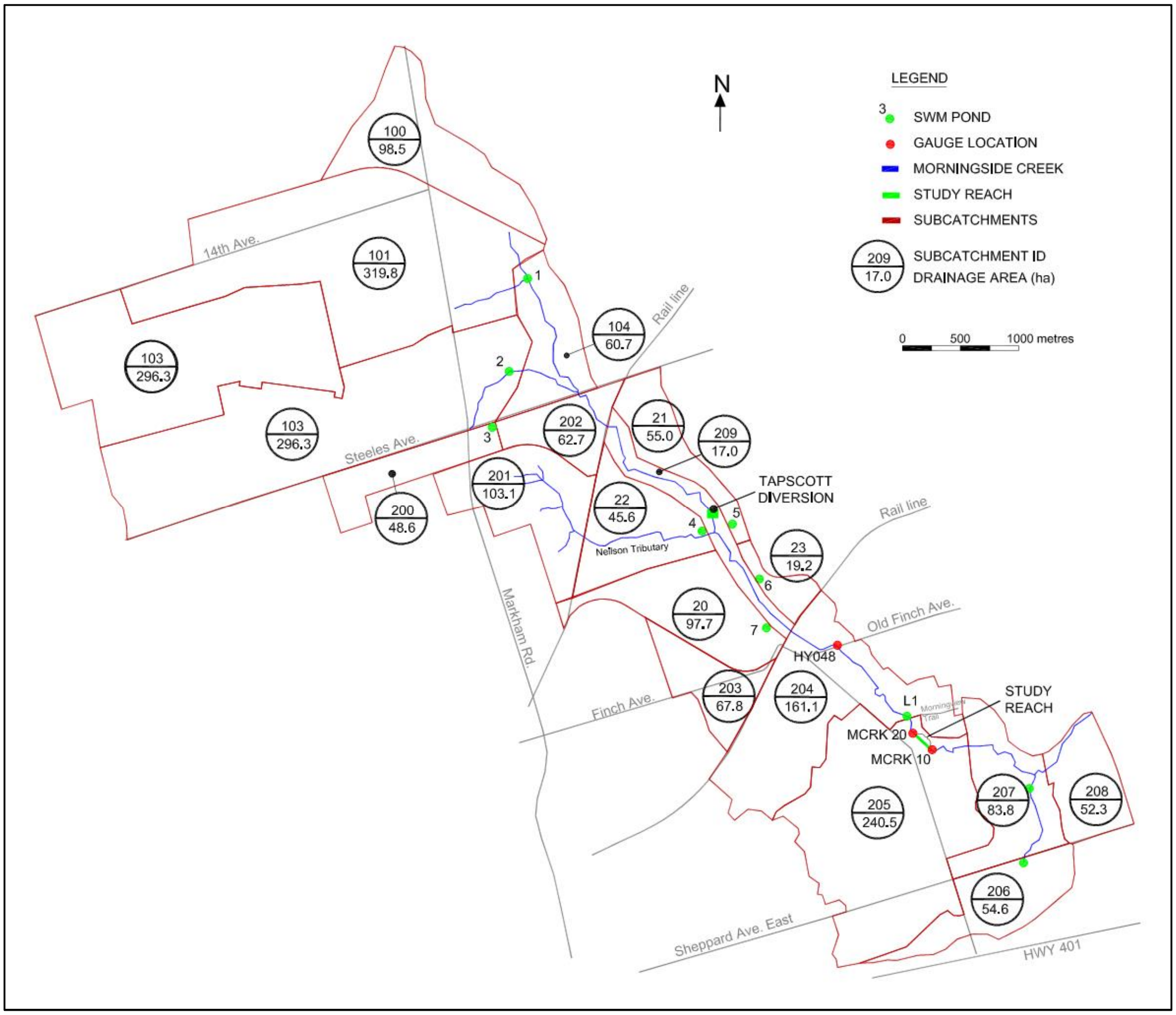


Figure 11: Morningside Catchment with SWM features, gauges and study reach

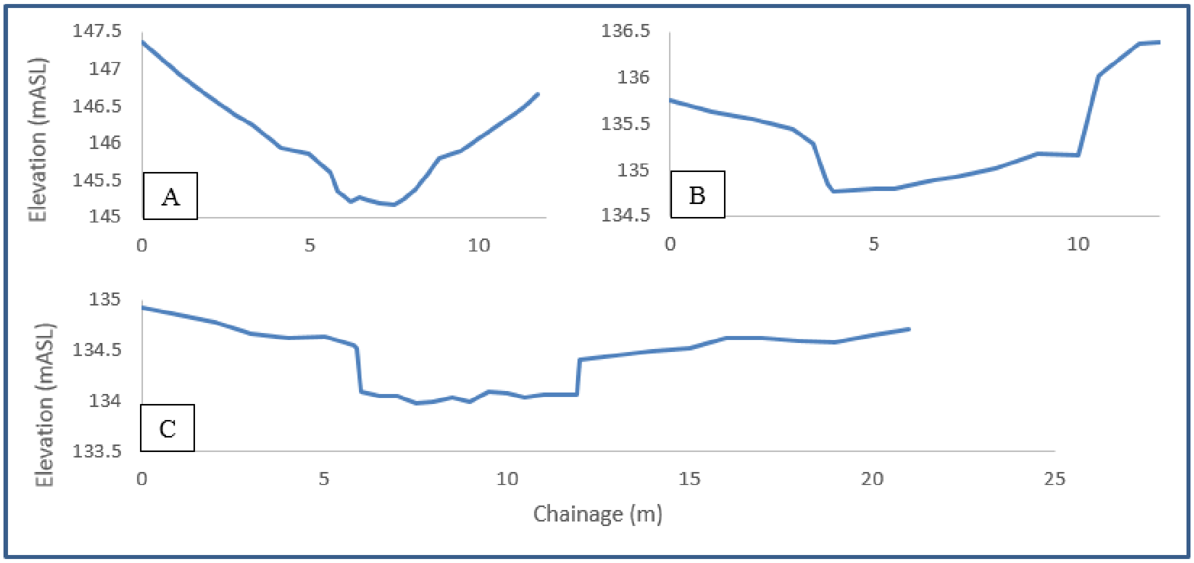


Figure 12: Cross-sections of A. MCRK HY048, B. MCRK 20, and C. MCRK 10

The bed surface has a well-mixed grain distribution, dominated by larger gravels and cobbles having a D_{50} and D_{84} of 40 mm and 99 mm, respectively. The grain size distribution is plotted in Figure 13. Within the limits of the study reach, the creek has a good connection to a floodplain, extending out on both banks. The planform description of the reach is gently meandering with a large range of radii of curvature. The longitudinal bed slope in the study reach is 0.0102 m/mas measured over a series of riffle crests (Figure 14).

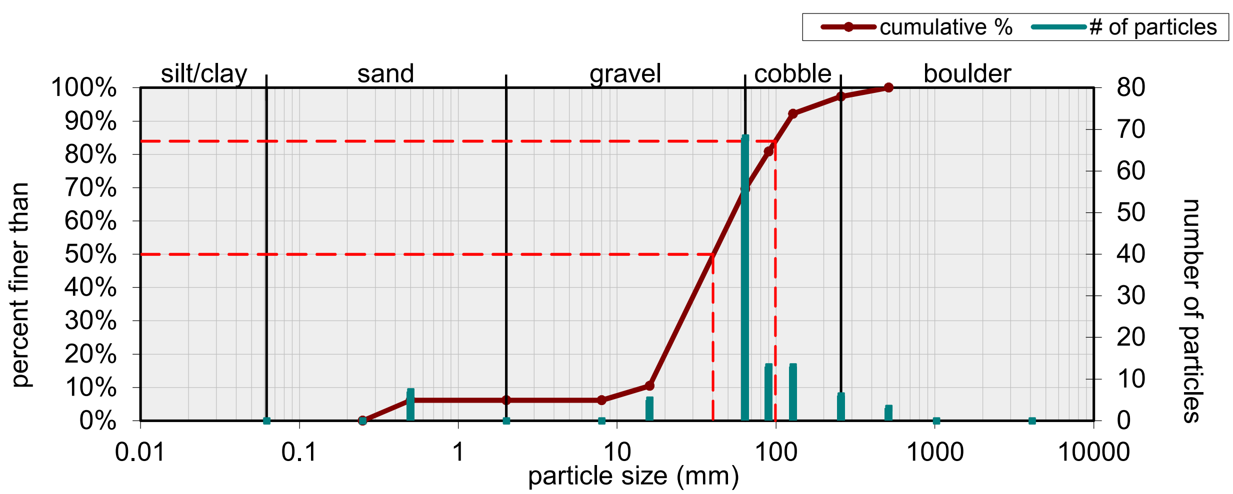


Figure 13: Channel bed surface material grain size distribution of Morningside Creek

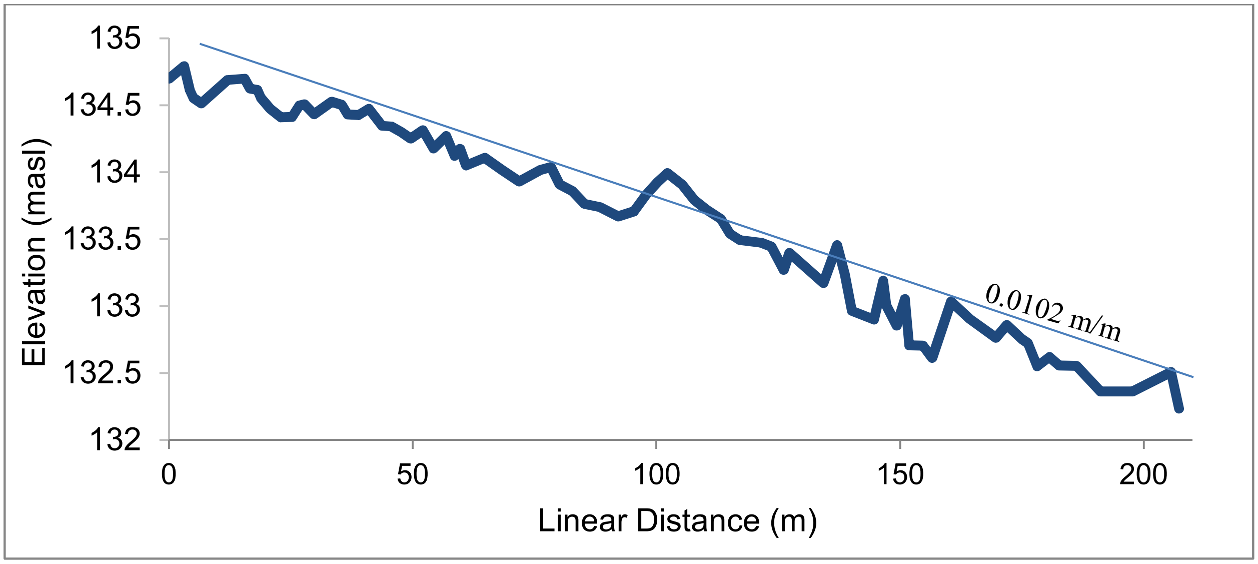


Figure 14: Longitudinal profile of Morningside Creek study reach

3.1.3 Stormwater Features of Morningside Creek

A combination of online and offline stormwater management facilities and a flow diversion structure are located throughout the Morningside catchment and along the creek (Figure 11**Error! Reference source not found.**). The headwaters are controlled by SWM detention ponds that outlet directly into the creek. The Tapscott diversion structure has a strong hydraulic impact on the downstream flows in Morningside Creek, diverting a large portion of storm flows to the Rouge River (MESP, 1999). The diversion structure is directly upstream of Seasons Avenue and has a contributing drainage area of 11.5 km^2, Table 1 gives the designed flow diversion values for the weir.

Table 1: Flow diversion values at Tapscott Diversion in m^3/s (Valley Design Report, 2002)

Total Incoming Flow	To Rouge River	To Morningside Creek	Percent Diverted
0	0	0	0
1.4[(1)]	0.7[(1)]	0.7[(1)]	50%
3.0[(1)]	2.0[(1)]	1.0[(1)]	67%
5.7	3.1	2.6	54%
13.2	9.6	3.6	73%
18.9	14.3	4.6	76%
25.9	20.1	5.8	78%
31.9	25.4	6.5	80%
38	30.5	7.5	80%
115	95.8	19.2	83%

(1) Theoretical flows determined through model calibration

The confluence of the Nielson tributary with Morningside Creek is downstream of the Tapscott Diversion. The Morningside Heights development area, (highlighted in Figure 15) is located on the east side of Morningside Avenue between the two rail lines and is controlled by four stormwater management ponds, referred to as Hydro West, Hydro East, Silvercore, and Morningside. Storm runoff from upstream drainage areas is conveyed to each pond through a storm sewer system servicing the developments. Each detention pond was designed as an offline facility to capture and detain a 2 year, 33 mm storm event for the objective of erosion control. All storm runoff from larger events was designed to pass through the ponds, effectively uncontrolled, using the large overflow weir structure. The details of each pond are summarized in Table 2. Values for the four ponds in Morningside Heights were determined from the review of stormwater management design reports prepared by Schaeffers Consulting Engineers (2001).

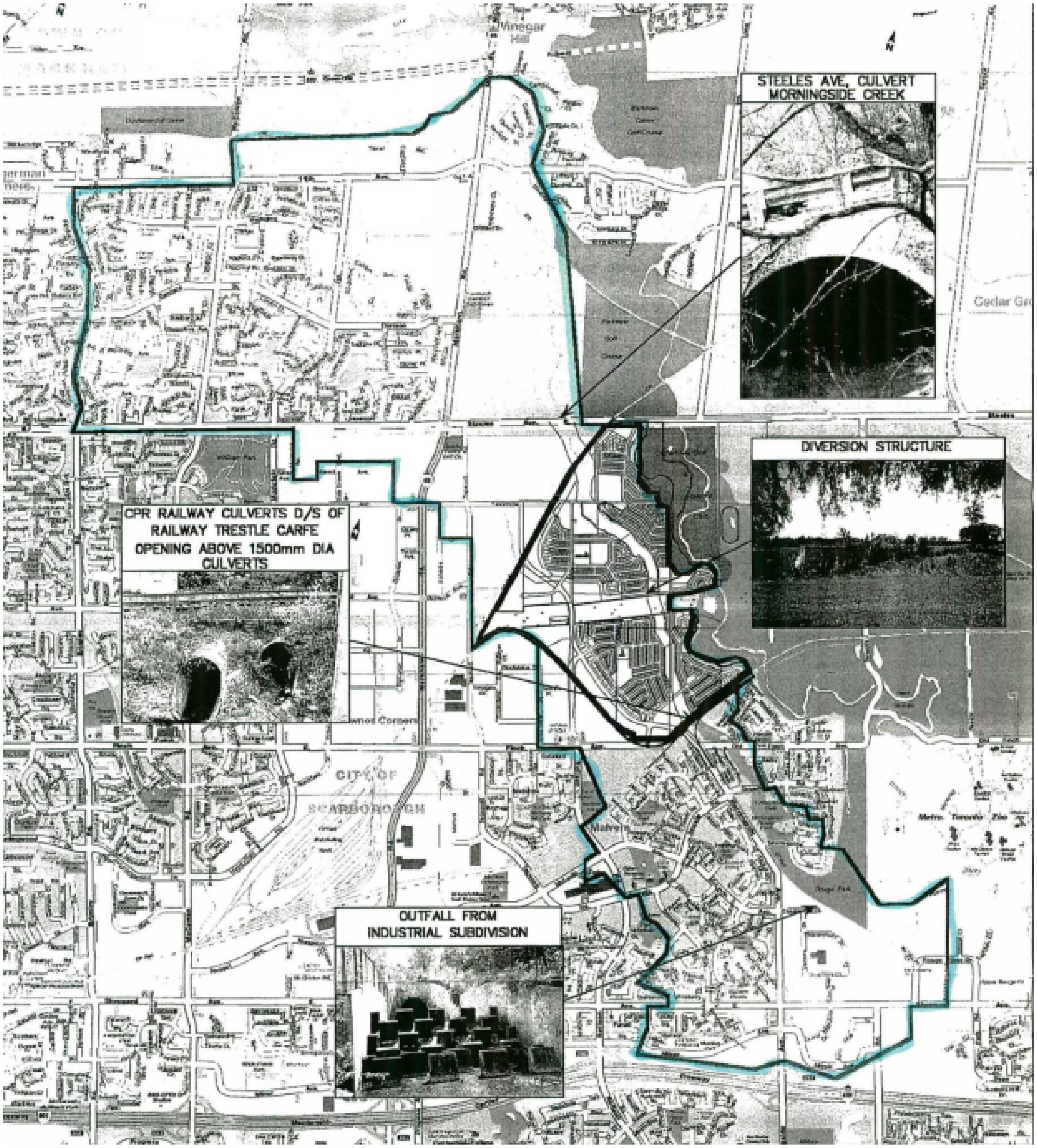


Figure 15: Morningside Catchment with Morningside Heights outlined. Figure from MESP (1999) prepared by Schaeffers Consulting Engineers.

Table 2: Summary of detention ponds in the Morningside Creek catchment upstream of the study reach

ID	Facility Name	Drainage area (ha)	Permanent Pool Storage (m^3)	Active Storage (m^3)	Outlet Orifice Diameter (mm)	Maximum Controlled Outlet Release Rate (m^3/s)	Length of Overflow Weir (m)	Maximum Release Rate (m^3/s)
1	Markham North Pond & Wetland	702.5	-	221,000	-	-	-	32.5
2	Markham South Pond	344.1	-	61,000	-	-	-	17
3	Tapscott Industrial Park	97	-	29,500	-	-	-	15
4	Hydro West	45.6	8,540	15,280	190	0.084	25	7.09
5	Hydro East	55	9,403	17,775	200	0.086	22	6.25
6	Silvercore	19.2	3,795	6,615	140	0.036	5	1.44
7	Morningside	97.7	19,096	29,885	275	0.18	45	19.59

Upstream of the study reach, located immediately upstream of Morningview Trail, is an online stormwater pond, referred to as Pond L1. The facility is controlled by a small culvert with a ditch inlet grate and drop structure for larger flows. The outlet, a large box culvert, can be seen on the downstream side of Morningview Trail, where energy is dissipated through a wide apron with baffle blocks.

3.2 Field instrumentation

3.2.1 Precipitation

Precipitation data was obtained from the Toronto Region Conservation Authority (TRCA). The data from the Milne Dam weather station was selected due to the proximity of the gauge to the catchment, located at an Easting, Northing of 639672, 4858742, approximately 8 km northwest from the study site. The 5 min precipitation data received from the TRCA was only available for the period of April 22, 2015 to December 7, 2015, April 14, 2016 to December 5, 2016, and April 15, 2017 to June 15, 2017. Supplemental daily precipitation data from Environment Canada's Buttonville Airport Weather Station, located at 43°51'39" N, 79°22'07 " W, was used to fill in the yearly record (Table 3).

Table 3: Annual Precipitation Trends and Values

Year	Recorded Period	Total Annual Rainfall	Number of Days with Precipitation	Number of Storm Events exceeding 25 mm	Ratio of storms exceeding 25 mm
2015	January 1 to December 31	729.2	146	6	4.1%
2016	January 1 to December 31	688.3	144	2	1.4%
2017	January 1 to June 15	507.6	79	3	3.8%
Expected Average	January 1 to December 31	793/840	-	-	5%

The average annual precipitation in the City of Toronto is 793mm[a]/ 840 mm[b] (a-Stat Can, 2006; b-WWFMG, 2006). The exceptionally low total precipitation in 2016 resulted in very low base flows in Morningside Creek. The frequency of occurrence of a 25 mm volume storm event or greater is approximately 5% of annual occurrences in the City of Toronto according to the WWFMG (2006). During the period of study of 2015 to 2017, the occurrence of larger storm events was less frequent than expected. Details of the most significant storm events are given in Table 4. The storm intensity was calculated by dividing the total precipitation volume by the total storm duration. The storms were plotted versus the theoretical IDF curves for Morningside Creek and the return period of each storm was determined through visual comparison to the plotted curves (Figure 16).

Table 4: Characterization of major storm events

Date	Total Volume (mm)	Total Duration (hr)	Average Intensity (mm/hr)	Return Period (1 in X year)
6/22/2015	29.2	4.1	7.2	<2
6/27/2015	36.2	10.8	3.4	<2
9/19/2015	20.6	1.1	19.0	2
10/28/2015	47.8	16.8	2.8	2
3/31/2016	8.8	5.2	1.7	<2
6/11/2016	13.2	0.8	15.8	<2
7/25/2016	21.4	2.2	9.9	<2
8/13/2016	17.4	2.9	6.0	<2
4/15/2017	7.4	3.2	2.3	<2
4/20/2017	14.6	4.7	3.1	<2
4/30/2017	32.6	19.4	1.7	<2
5/4/2017	50.0	30.2	1.7	2

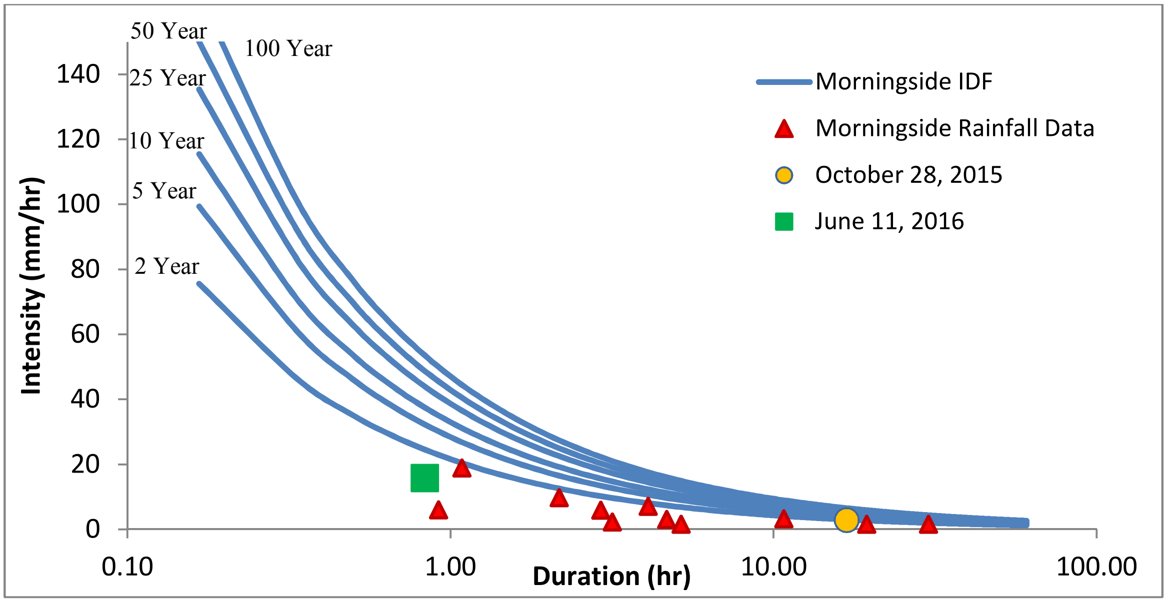


Figure 16: Measured storm events during the period of study from January, 2015 to June, 2017 plotted against the intensity duration frequency curves of Morningside Creek produced by the Ministry of Transportation Ontario IDF Curve Lookup

The largest significant precipitation event recorded during the period of study was approximately a 2 year return period event. The distribution of durations was quite large, with a number of short, high intensity events like June 11th, 2015, as well as longer, low intensity events like the October 28, 2015 storm (Table 4).

3.2.2 Flow Measures

The site was instrumented with in-stream pressure transducers to measure water level. The gauges were HOBO 13-Foot Fresh Water Level Data Loggers (model U20-001-04), with a calibrated accuracy of ±3mm and a depth range of 0 – 4m. Two gauges were set up within the study reach: MCRK 20 located upstream of the first seeded riffle; and MCRK 10 located 200 m downstream. A third water level gauge, MCRK HY048, was placed on the upstream side of Old Finch Ave, located 1.35 km upstream of the study reach (**Error! Reference source not found.**). The TRCA also has a water level gauge a few metres upstream of the culvert at Old Finch Ave. An additional transducer to measure atmospheric pressure was placed at the study site. The gauges were set to collect data at varying intervals, either 1 or 2 min intervals from April through November, and 7 min intervals for the remaining winter months.

A rating curve was developed to translate water level to total channel discharge (Figure 17). The discharge curve was created using Manning's flow resistance equation with a detailed cross-section survey and estimation of Manning's n values based on the bed surface material. Field measurements of discharge were done using the SonTek FlowTracker, an in-stream acoustic Doppler velocimeter. The technique measures 1D velocity at width intervals within the channel and uses those measurements to estimate the total discharge for a given stage. Measurements were taken on three different days at varying water levels to better define the stage-discharge relationship. The theoretical discharge rating curve was then adjusted to better fit the field measured data. All flow measurements were recorded in low flow conditions, and all high flow values remain theoretical. In the calculation of the theoretical rating curve, Manning's n values of 0.065 and 0.11 were used for the channel and floodplains respectively. The selected Manning's n value within the channel is higher than most reported values for gravel-bed rivers, but is reflective of the low flow conditions under which it was measured.

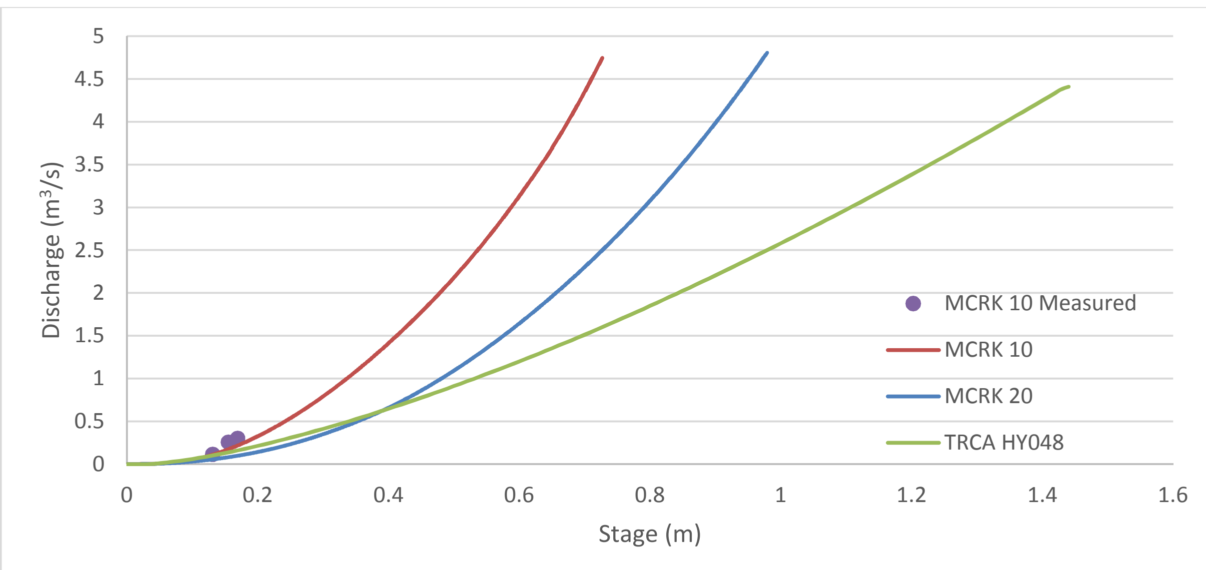


Figure 17: Rating Curves for MCRK 10, MCRK 20, and MCRK HY048

3.2.3 Particle Tracking

The particle distribution of the bed was determined using a Wolman (1954) pebble count, with a sampling size of 200 points. Following recommendations of MacVicar et al. (2015) stones were collected and sorted into 3 size classes/bins belonging to the half φ bins 5-5.5, 6-6.5, and 7-7.5, which correspond to the surface D_{50}, D_{75}, and D_{90} size classes. Each stone was drilled and a passive integrated transponder (PIT) tag was inserted with a unique radio frequency identification (RFID) number. A total of 300 stones were tagged, 150 in the D_{50} size class, 32 – 45 mm, 100 in the D_{75} size class, 64 – 90.5 mm and 50 in the D_{90} size class, 128 – 181 mm. For calculation purposes, the median value was taken from each size class range, giving values for the D_{50}, D_{75} and D_{90} as 38.5 mm, 77.25 mm, and 154.5 mm respectively. In August 2015, two consecutive riffles were seeded with 150 stones each. The stones were distributed across the riffle in 25 rows, with 6 stones per row about 25 to 50 cm apart. Each row was seeded with 3 D_{50}, 2 D_{75}, and 1 D_{90} size class stones. The particles were seeded on the bed surface, replacing a stone of similar or slightly larger size, in an effort to match the imbrication and hiding effects of the existing bed condition.

The tracking period recorded for this study was from August 9, 2015 to May 11, 2017. Efforts were made to track the position of the tagged particles after major flow events in order to capture mobilization at the event scale. This was not always possible due to storms in close succession, in which case the positions were recorded as often as possible. A total of 7 tracking events were completed over the course of the study. In each tracking event, the PIT tagged stones were located using a combination of a large loop antenna and a smaller stick antenna manufactured by Oregon RFID. The recovery range of the large and small antenna was 1.0 m and 0.2 m respectively. The position of each stone was recorded using a Sokkia SET650RX total station and known benchmarks previously inserted along the study reach. By confirming all positions with the stick antenna, the position of the stone was assumed to be accurate to within 0.2 m. The RFID technology allowed for identification of the stones without removal and minimal disruption to the bed and hence the stones were left in-situ throughout the duration of the study.

3.3 Field Data Analysis

3.3.1 Hydrologic Analysis

In order to understand the hydrologic response in Morningside Creek and its relative impact on bedload sediment transport, a number of hydrometrics were calculated. A number of studies have shown a relationship between the channel flow and the sediment transported through the channel; however, the best measure of flow for modelling sediment transport in unsteady flow is greatly debated. A number of measures were therefore calculated to find the best relation in Morningside Creek.

3.3.1.1 Runoff Ratio

The runoff ratio was calculated as a way of normalizing the total storm runoff volume by the size of the storm event, similar to the procedure used by Bell et al (2016). The equation for runoff ratio is:

$$\boldsymbol{Runoff\ Ratio = \frac{Total\ Runoff\ Volume/Catchment\ Area}{Total\ Event\ Precipitation}} \qquad \textbf{(14)}$$

The runoff ratio provides an indication of the overall hydrologic response of the catchment across storm events. It also allows for a normalized comparison across other field sites and other findings with different size catchments. The limitation of this equation is the inability to account for the spatial distribution of rainfall across the catchment area, thus assuming a homogenous distribution of rainfall.

3.3.1.2 Time of Exceedance

The calculation of time of exceedance, T_e, is also commonly referred to as the duration of competent flow. In this study, T_e refers to the length of time for which the shear stress is above the critical shear stress required to initiate movement of the D_{50} particle size class. For all hydrometrics using time integrals, the time of exceedance follows this definition.

3.3.1.3 Cumulative Effective Work Index

Cumulative effective work index, Wi, is given in Equation 8. The Wi is an erosion index that is sensitive to large floods due to its inclusion of the velocity term, which increases with rising flood stage, making it a strong predictor of sediment transport (Garcia, 2008). The Wi was selected as a hydrometric based on its merit and its widespread use by SWM practitioners, outlined in the SWM guidelines of both the TRCA and Credit Valley Conservation (CVC).

3.3.1.4 Dimensionless Impulse

Another time integrated metric used to determine sediment transport is impulse, the product of shear stress magnitude and duration (Diplas et al., 2008). Dimensionless impulse, I_*, was calculated using Equation 9 (Phillips et al., 2013).

3.3.1.5 Cumulative Excess Shear Stress

The CESS was calculated using Equation 10 for each of the three tracer size classes and was compared to the field data presented by DeVries (2000). Doyle et al. (2000) argue that the best metric for gravel-bed rivers with variable substrate size is excess shear stress, which takes into account both the eroding and resisting forces. The CESS is a measure of the excess bed shear experienced by the particles in the channel over the period of competent flow. The equation uses an exponent of 3/2, which is similar to other sediment transport equations, such as the Meyer-Peter-Muller equation (1948). DeVries (2000) suggests that there exists a relation between particle travel distances and CESS, Equation 12. Particle tracking data from Morningside Creek is plotted against DeVries equation, as well as being used to provide a modified equation.

3.3.2 Sediment Transport Analysis

In order to assess bed particle mobility in the study reach of Morningside Creek, the total fraction of mobile tracer particles was calculated for each tracking event. The fraction of mobility (F_m) was calculated as the ratio between the number of tracers that moved (n_m) and the total number of tracers recovered or inferred (n_f).

$$\boldsymbol{F_m = \frac{n_m}{n_f}} \tag{15}$$

A tracer was considered moved if its position was at least 0.4 m downstream from its previous recorded position. A threshold of 0.4 m was selected as it is twice the detection limits of the tracking antenna and thus provides confidence that the tracer has in fact moved. Tracer positions could be inferred for tracers, which though not recovered in the tracking event in question, were recovered in subsequent tracking events and whose position had not changed. Thus, the tracer's position could be inferred in the previous tracking period.

The location of each particle was converted from Cartesian coordinates to a stream-wise normal coordinate system, using a method similar to Legleiter and Kyriakidis (2006). Particle travel distance was calculated as the downstream distance travelled compared to the channel thalweg. The average travel distance, $\bar{L}$, of the three stone classes was calculated. Following the work of Church and Hassan (1992) a normalized travel distance was also calculated for each stone size class and was plotted against Equation 5.

Bulk sediment transport can be measured inversely using the spatial integration method (Hassan et al., 1991; Haschenburger & Church, 1998) using Equation13. The three parameters that need to be measured in the calculation of bulk sediment transport are virtual velocity, active layer thickness and active layer width. Bulk sediment transport volumes were not calculated for Morningside Creek, and this metric was only used as a means of comparing bulk sediment transport in the modelled scenarios. In each scenario, the virtual velocity was measured as the total distance travelled by the D_{50} particle size class for that single storm event. It was assumed that the active layer width and thickness were constants among the modeled scenarios. With this assumption, the change in virtual velocity across the modelled scenarios is proportional to the change in bulk sediment transport. As such, the impact of varying SWM strategies could be related to bulk sediment transport for Morningside Creek.

3.4 Hydrologic Model Set-up

3.4.1 Background

A hydrologic model was developed for the Morningside catchment area. A Visual Otthymo model was received from the TRCA for the Rouge River watershed, last updated and calibrated in 2001. The model was received as an Otthymo 89 file and was imported into Visual Otthymo Version 2.4 (VO2). Three scenarios of the model were received: an existing model for pre-2001 land use, a committed development model, and a complete development model. The chosen scenario was the committed development, as it seemed best suited for analysis of the current state of the watershed. The existing scenario was outdated, lacking current information regarding land use and SWM facilities that have been developed. The complete development scenario was detailed as an ultimate condition where potential development may occur and did not reflect the current state of the watershed.

Due to some unknown computing artifacts, when the model was imported there were some unresolved errors. The following updates were made to the model to rectify some of the errors. The ROUTE CHANNEL routine requires the Manning's n value for the main channel segment to be input as a negative value; the values were updated with the magnitudes left unchanged. The STANDHYD commands were changed to match the TRCA model, which uses the United States Department of Agriculture Soil Conservation Service (SCS) method. The model was run to compare the results with the given results from the TRCA (Table 5). Small discrepancies in flow were attributed to updated flow routing calculations in the newer version of the software.

Table 5: Comparison of peak flows and time to peak flow (TP) from TRCA reported summary values and the rectified VO2 model.

Node	Description	5 year storm event				100 year storm event			
		TRCA Summary		Rectified VO2 Model		TRCA Summary		Rectified VO2 Model	
		Q_p (m^3/s)	TP (hr)	Q_p (m^3/s)	TP (hr)	Q_p (m^3/s)	TP (hr)	Q_p (m^3/s)	TP (hr)
843	Random U/S Test node	16.663	7.25	16.663	7.25	41.283	7.00	41.283	7.00
894	Rouge River, U/S of Morningside confluence	71.778	17.25	71.775	17.25	163.811	15.25	163.807	15.25
900	Upstream of Tapscott diversion	9.552	7.50	9.553	7.50	27.064	7.00	27.064	7.00
901	Morningside Creek, D/S of diversion	13.499	5.00	13.499	5.00	25.745	7.00	25.746	7.00
999	Rouge River, D/S of diversion	72.460	17.25	72.456	17.25	165.551	15.25	165.547	15.25
23	Downstream of study reach	16.36	6.50	16.363	6.50	51.36	5.50	51.360	5.50
903	Confluence of Morningside and Rouge	69.541	19.75	69.539	19.75	161.195	17.00	161.191	17.00
957	Final Node of Rouge River	85.523	22.25	85.521	22.25	201.718	19.75	201.715	19.75

3.4.2 Discretization of Morningside Creek Hydrologic Model

In order to develop a well calibrated hydrologic model, further detail was added to the hydrology model for the Morningside Creek subcatchment. In order to achieve this goal, revisions and additions were made, largely based on the Qualhymo model prepared by Schaeffers Consulting Engineers as part of the Valley Design Report (2002). One revision was the addition of detail to the SWM features. The TRCA model incorporated lumped ponds into a single reservoir and, while able to account for storage and discharge, the TRCA model failed to reflect the time shift expected in the flow discharged from each individual pond.

The four stormwater ponds in the Morningside Heights development and their upstream drainage areas were discretized to more accurately reflect the time shift of pond outflows (**Error! Reference source not found.**). The Hydro West, Hydro East, Silvercore and Morningside SWM ponds, known respectively as TRCA ponds 311.0, 311.1, 311.2 and 311.3 were inserted downstream of the DIVERT HYD command, which represents the Tapscott diversion on the north side of Seasons Drive. The parameters of the catchment areas were chosen to mimic the existing catchments of the model (Table 6). The rating curves of the ponds were developed using the information provided by the SWM reports for the design by Schaeffers Consulting Engineers (2001).

Table 6: Uncalibrated parameters of the added subcatchment nodes. TIMP is the total impervious fraction, SLPP/SLPI is the subcathment slope, CN is the curve number value and Ia is the initial abstraction for pervious areas.

	Hydro West	Hydro East	Silvercore	Morningside
Node Reference	22	21	23	20
Area	45.6	55	19.2	97.7
TIMP	0.631	0.416	0.524	0.524
SLPP/SLPI	0.97	0.42	0.99	0.99
CN	76.7	76.7	62.3	62.3
Ia	5.0	5.0	5.0	5.0

Modifications to the existing catchment and routing commands were necessary to reflect the discretization. The ROUTE CHANNEL 767, representing the main channel of Morningside Creek, was originally a length of 4817 m, which is approximately the length of Morningside Creek from the Tapscott diversion to the confluence. During the discretization, the 767 node was divided into smaller route channel commands, whose sum equaled the original length of 4817 m.

3.4.3 Model Calibration

The model was calibrated to match existing flow measurements for two storm events, October 28, 2015 and June 11, 2016. In order to determine the accuracy of the model, the Nash-Sutcliffe model efficiency coefficient (Nash & Sutcliffe, 1970) was calculated.

$$\boldsymbol{E = 1 - \frac{\sum_{t=1}^{T}\left(Q_m^t - Q_o^t\right)^2}{\sum_{t=1}^{T}\left(Q_o^t - \overline{Q_o}\right)^2}} \qquad \textbf{(16)}$$

where Q_m^t= modelled discharge, $\overline{Q_o}$= mean of measured discharges and Q_o^t = measured discharge at time t. An efficiency of 1 (E=1) would correspond to a perfect match, where the modeled discharge equals measured values. A result of E<0, would indicate that the observed mean discharge is a better predictor of instantaneous discharge than the model. Therefore, the closer the efficiency is to a value of 1, the stronger the model calibration.

The hydrologic model was calibrated using two storm events, October 28, 2015 and June 11, 2016, the largest storm during the study period and a typical smaller high intensity event, respectively. The June 11, 2016 storm event is significant as it is shown later to be a threshold event for bedload sediment transport in the creek. The model was calibrated to match the discharge measured at two gauge locations, MCRK 10, the downstream limits of the study reach and TRCA HY048, located on the upstream side of Old Finch Avenue. The Nash-Sutcliffe model efficiency coefficient was calculated for each event and at both locations to assess the model's accuracy in predicting discharge. The results of the model calibration are presented in Table 7.

Table 7: Calibration Results for VO2 model of Morningside Creek

		TRCA HY048		MCRK 10	
		Measured	Modelled	Measured	Modelled
October 28, 2015	Nash-Sutcliffe E		0.914		0.811
	Peak Discharge (m^3/s)	1.85	2.00 (109%)	2.97	2.52 (85%)
	Total Flow Volume (m^3)	251,048	274,205 (109%)	333,624	364,123 (109%)
June 11, 2016	Nash-Sutcliffe E		0.762		0.762
	Peak Discharge (m^3/s)	0.84	0.74 (88%)	0.91	0.96 (106%)
	Total Flow Volume (m^3)	92,926	74,587 (80%)	72,602	86,162 (119%)

In calibrating the model, it was determined that there was a strong dependency of the hydrograph response to the online SWM ponds and the Tapscott Diversion structure. The falling limb of the hydrograph was very sensitive to the configuration of the L1 online pond. This is evident in the difference in the falling limb seen in the October 28 storm event (Figure 18), where the flow seems to remain quite high before quickly tapering down to baseflow. Additionally, during the calibration, it was found to be necessary to account for natural detention features in the section of creek upstream of the Tapscott Diversion to match the timing and magnitude of the flood wave at downstream locations. Site reconnaissance confirmed the presence of multiple beaver dams and wood jams upstream of the diversion, creating small reservoirs along the length of the creek.

The Nash-Sutcliffe coefficients were strong for both the October 28 storm event (E = 0.91 and 0.81 for TRCA HY048 and MCRK 10, respectively) and the June 11 storm event (E = 0.76 for both locations). The slightly lower result for the June 11 event could be attributed to a number of factors.

For example, the assumption of catchment wide homogeneous rainfall, especially for the short summer rainfall event, can be a poor assumption. The hydrograph at MCRK 10 does plot well for the June 11 storm, seen in Figure 18. There is a very minor time shift in the modeled versus measured data, which can greatly impact the Nash-Sutcliffe coefficient. However, for the purpose of testing the scenarios, the hydrographs for both events are quite robust.

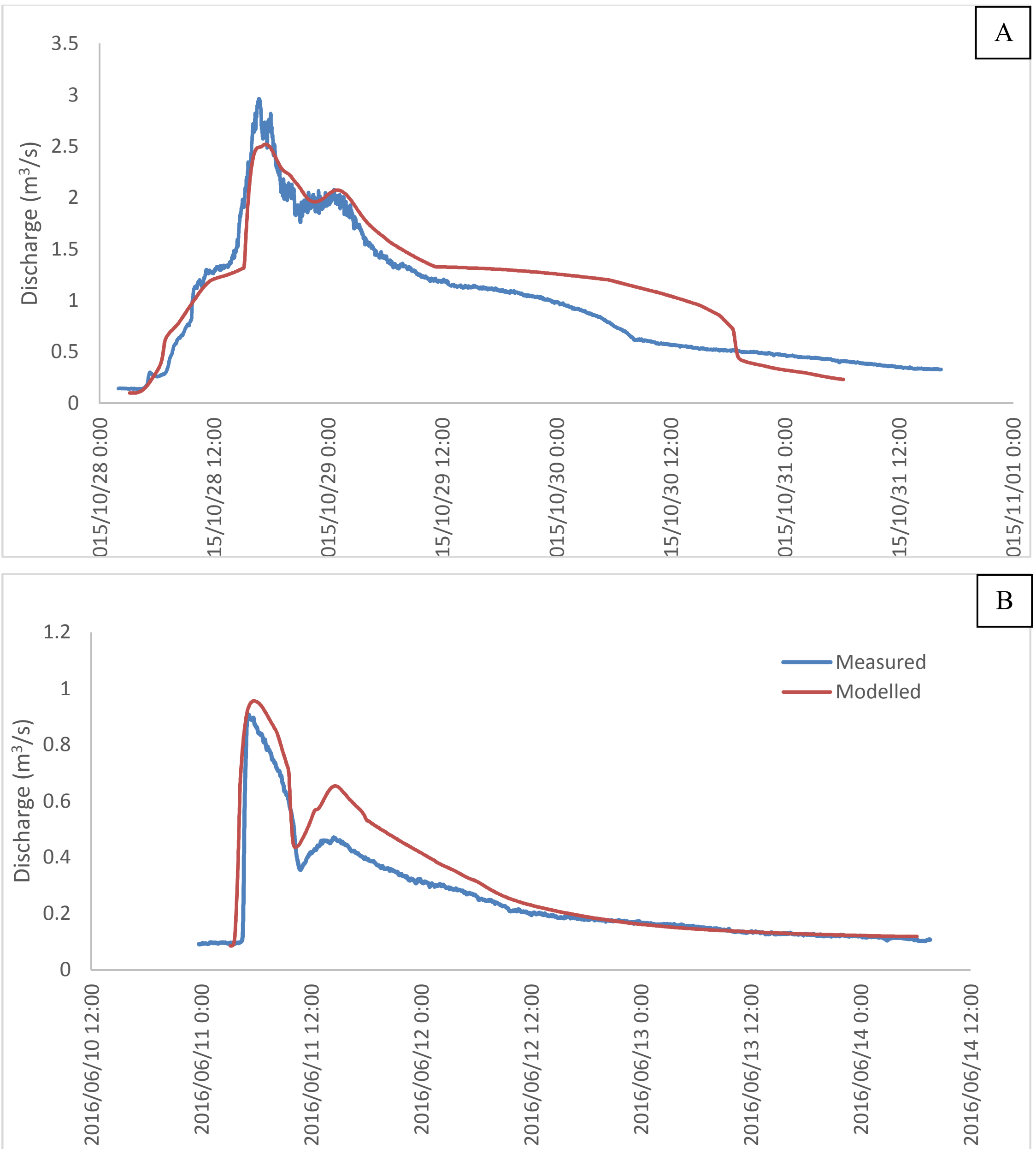


Figure 18: Calibration results for (A) October 28, 2015 and (B) June 11, 2016 storm events.

3.4.4 Model Validation

The model was validated using two storm events, July 24, 2016 and May 4, 2017. The July 24 storm event was selected because it represents a short duration high intensity event, typical of summer storms in the area. The May 4 storm was a large event that exceeded threshold. The validation was performed using the measured flow values at MCRK 10 and the results are presented in Table 8 and Figure 19.

Table 8: Model validation results

		Measured	Modelled
June 24, 2016	Nash-Sutcliffe E		-1.676
	Peak Discharge (m^3/s)	0.839	1.037 (124%)
	Total Flow Volume (m^3)	84,557	135,279 (160%)
May 4, 2017	Nash-Sutcliffe E		0.253
	Peak Discharge (m^3/s)	2.197	2.357 (107%)
	Total Flow Volume (m^3)	275,505	386,864 (140%)

Model validation resulted in Nash-Sutcliffe efficiency values of -1.676 and 0.253 for the June 24, 2016 and May 4, 2017 event, respectively. The negative efficiency value would provide very little confidence in the model; however, it is important to consider the other variables as well. The model overpredicts flow and runoff volume, which suggests that the modeled storm precipitation may have been overestimated, especially under the assumption of homogeneous rainfall across the catchment area. Additionally, with the smaller flashier storm, the Nash-Sutcliffe value is very sensitive to the timing of the spikes in the hydrograph. It should be noted that in the days leading up to May 4, 2016 storm event, there were a few smaller precipitation events totaling 35 mm of precipitation. The event based model was not capable of accounting for the antecedent conditions for the May 4, 2016 event, making it difficult for the model to reproduce the measured hydrograph. Similar to the October 28, 2015 storm event, the model overpredicts flow in the May 4, 2016 storm event during the drawdown phase.

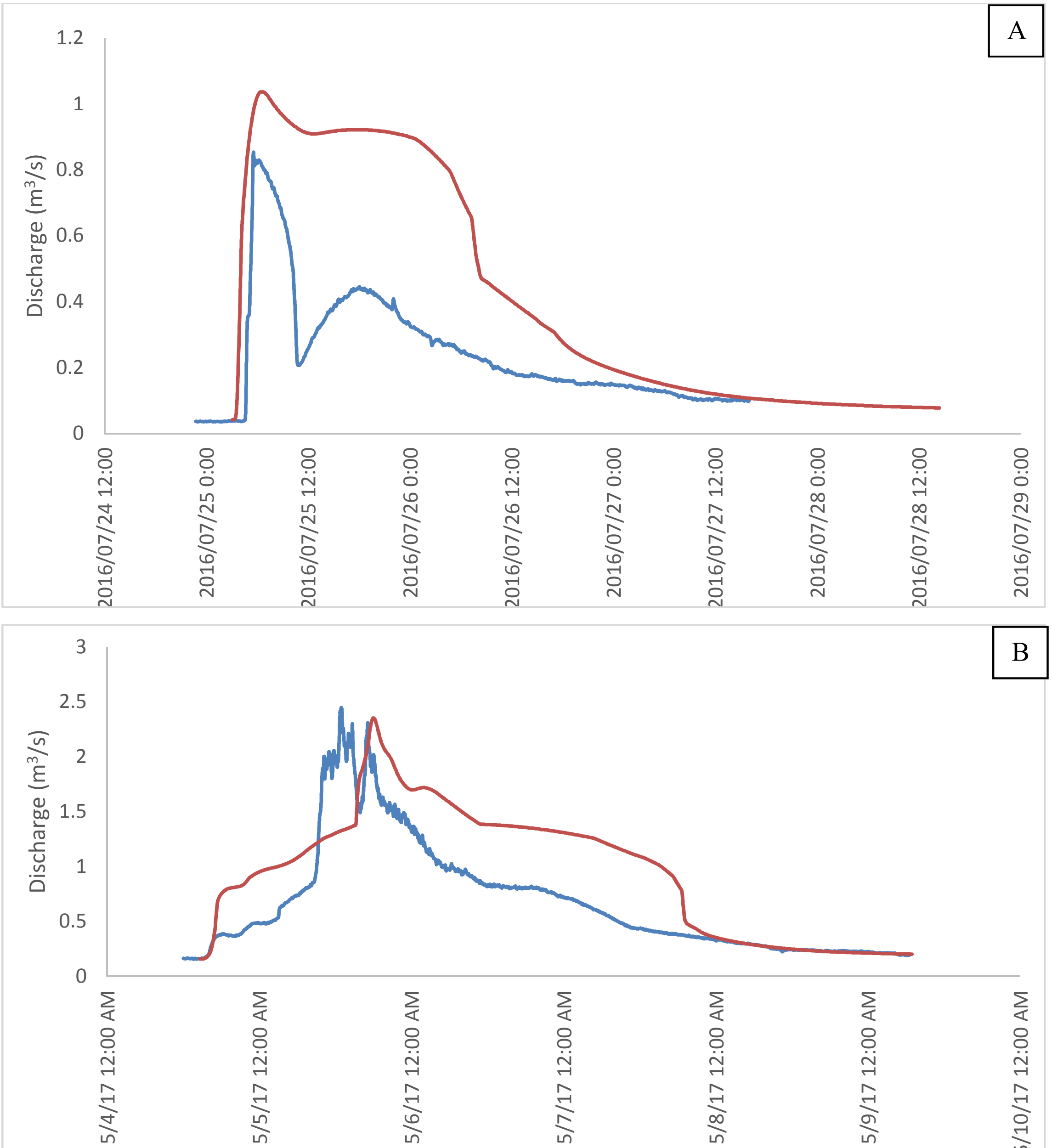


Figure 19: Model Validation Results. Hydrographs for the storm events A) July 24, 2016 and B) May 4, 2017

Chapter 4
Field Results

4.1 Runoff

The hydraulic response in Morningside Creek was recorded using the instream water level gauges. The shape of the hydrographs for MCRK 10 and 20, the two gauges located within the study reach, were relatively similar. The channel response recorded at the upstream gauge, MCRK HY048, is distinct in its shape in comparison to the response seen at the two gauges of the study reach. Bankfull elevations were estimated based on visual interpretation and markings along the banks and floodplain and bankfull discharges were found using the calculated rating curves. Bankfull flows were estimated as 1.55 m^3/s and 1.12 m^3/s at cross-sections MCRK 10 and MCRK 20 respectively.

The response for each storm is unique in its magnitude, duration and pattern, which are functions of the precipitation event, antecedent moisture conditions and the SWM facilities. The following two storms were chosen to be analyzed in detail, October 28, 2015, and June 11, 2016, as they represent one of the largest storms during the study period in terms of total precipitation and a typical smaller high intensity event, respectively. The hydrographs and hyetographs for the two storm events are shown in Figure 20.

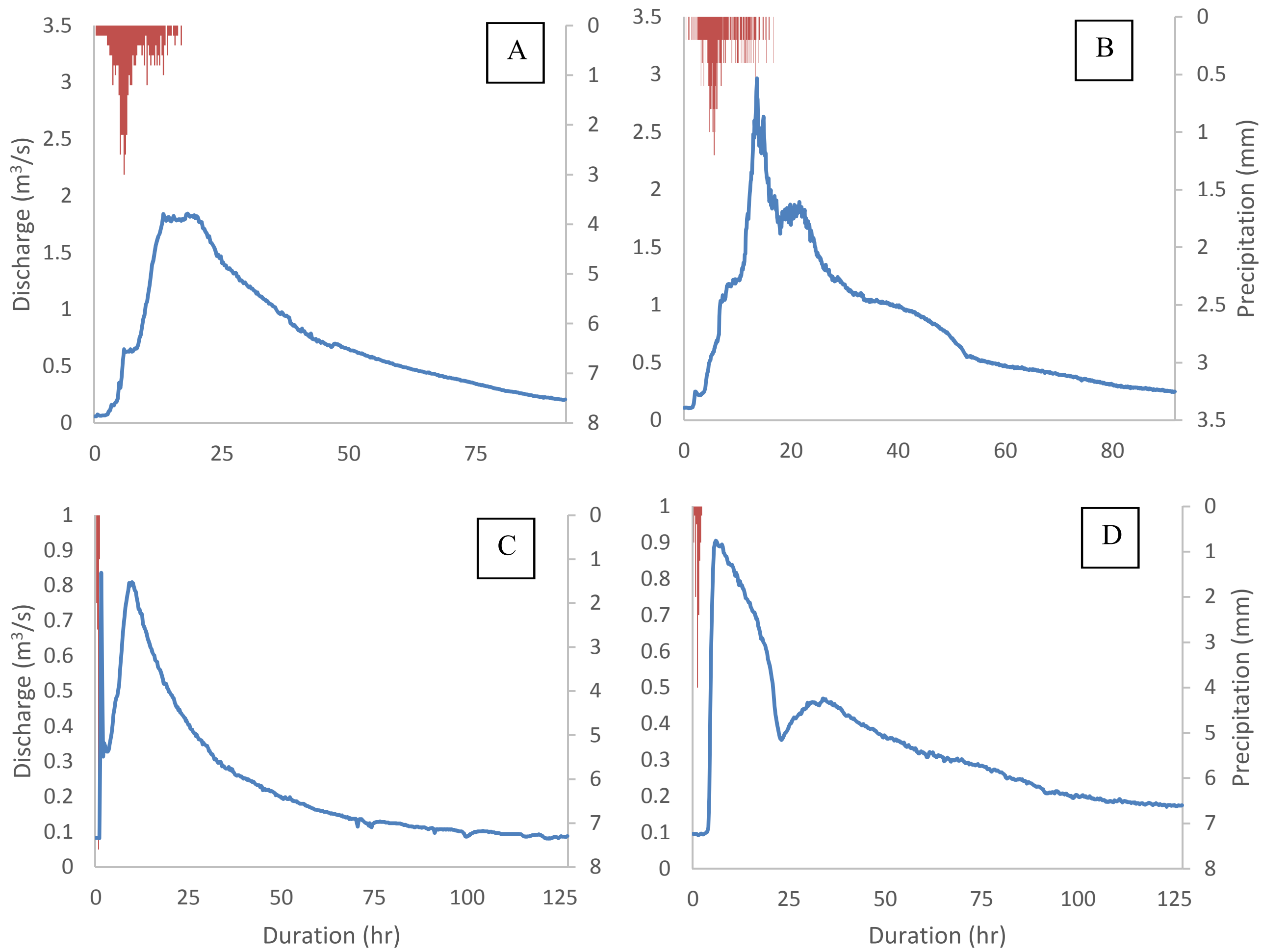


Figure 20: Hydrographs for Morningside Creek. A) October 28, 2015 measured by the TRCA at HY048 B) October 28, 2015 measured at MCRK 10 C) June 11, 2016 measured by the TRCA at HY048 D) June 11, 2016 measured at MCRK 10.

The hydrographs recorded in high temporal resolution have complex shapes, with multiple peaks and inflection points. The impact of the online pond, Pond L1, located on the upstream side of Morningview Trail is quite visible in comparing the hydrograph at HY048 to MCRK 10. The duration of the initial peak is extended significantly as a result of the online detention facility. The initial peak appears not to decrease in magnitude (comparing Figure 20C and D), this can be attributed to additional flow entering the channel between the two gauges. Common to the MCRK 10 hydrographs for both storm events is the slow falling limb that is typical of a SWM detention pond. In Figure 20B, an inflection point is apparent in the falling limb, which is likely attributed to the water level at which discharge through the overflow grate ceases and the online pond returns to an orifice controlled system.

The short-duration high-magnitude pulse at the beginning of the hydrograph of the smaller June 11, 2016 event shows that there is a significant portion of uncontrolled runoff entering the study reach of Morningside Creek (Figure 20C and D). This flow pattern was also seen in other small measured events, July 24, 2016 and August 13, 2016. The sharp spike in flow, with a very short duration, seen at the HY048 gauge, suggests that there is uncontrolled runoff entering upstream of this gauge. The second peak seems to occur approximately 9 hours after the initial peak. The 9 hour gap between peaks seems to be consistent regardless of the storm size or magnitude of the peak flow and can be seen in the October 28, 2015 event as well. It is hypothesized that the second peak is delayed by the channel routing and SWM features upstream of the Tapscott diversion. It appears that there are two response waves, the first being uncontrolled runoff and then a second controlled release of lesser peak magnitude and longer duration. The second peak also appears to accelerate at a lesser rate, taking longer to reach its peak value. This could be indicative of a longer time to concentration of a larger subcatchment area upstream or it could be attributed to the SWM controls in the upstream portion of the catchment.

The hydrograph of October 28, 2015 (Figure 20A), does not have as flashy of a response as the peak seen in June 11, 2016. This is most likely attributed to the longer duration, lower intensity storm event. The hydrograph does have the same double peak, while not as drastic as the June 11 storm. The double peak in this case could be attributed to the precipitation pattern, which shows a variation in intensity and two identifiable peaks, or the individual response of the upstream SWM detention ponds, which were designed for a 33 mm event. Given that the October event exceeded this design event, it likely created an overflow response from one or more ponds, resulting in the plateau of higher flow seen at HY048. Additionally, HY048 is located at a culvert, and the plateau may be related to backwater effects, creating an artificially consistent high water level.

4.2 Particle Tracking

The particle racking field work was spread throughout the calendar year, with the primary goal to record the positions of the stones after major storm events in order to determine event scale displacements. A total of seven tracking periods were recorded (Table 9). The recovery rate, P_R, of the tagged stones was relatively high, with total recovery rates ranging from 88 to 97 percent. The recovery rates did drop slightly over time, as would be expected, however with the inclusion of inferred positions, the recovery rates remained fairly stable. Recovery rates were generally higher for the largest size class. The recovery rates and mobile fraction, F_m, for each of the tracking events are shown in Table 9. The tracer positions are displayed visually in relation to the channel thalweg in Figure 21. The first tracking period was excluded from further analysis due to artificially high mobility rates. This is typical of tracer studies, as despite best efforts particles tend to have loose configurations on the bed that are not representative of true particle to bed interactions (Lamarre & Roy, 2008).

Table 9: Bedload sediment transport measurements for each of the tracking events

Tracking Date	All Size Classes			Size Class 1 32 - 45 mm (150 Stones)			Size Class 2 64 - 90.5 mm (100 Stones)			Size Class 3 128 - 181 mm (50 Stones)		
	P_R	F_m	LD_{50}	P_R	F_m	$\bar{L}$	P_R	F_m	$\bar{L}$	P_R	F_m	$\bar{L}$
Oct 25, 2015	97%	27%	2.3	99%	35%	2.3	92%	24%	2.3	100%	6%	1.3
Nov 11, 2015	92%	57%	14.8	93%	64%	13.9	89%	61%	7.7	98%	31%	2.6
May 03, 2016	94%	17%	2.1	94%	22%	2.2	92%	15%	1.3	98%	4%	0.7
June 22, 2016	92%	4%	0.9	91%	4%	0.9	90%	2%	1.1	98%	4%	0.5
Aug 19, 2016	93%	3%	1.0	94%	3%	1.0	90%	3%	0.9	94%	0%	0.0
Apr 12, 2017	95%	10%	1.2	95%	11%	1.2	94%	6%	0.8	98%	12%	1.5
May 11, 2017	88%	27%	6.4	89%	33%	10.7	87%	31%	6.7	88%	0%	0.0

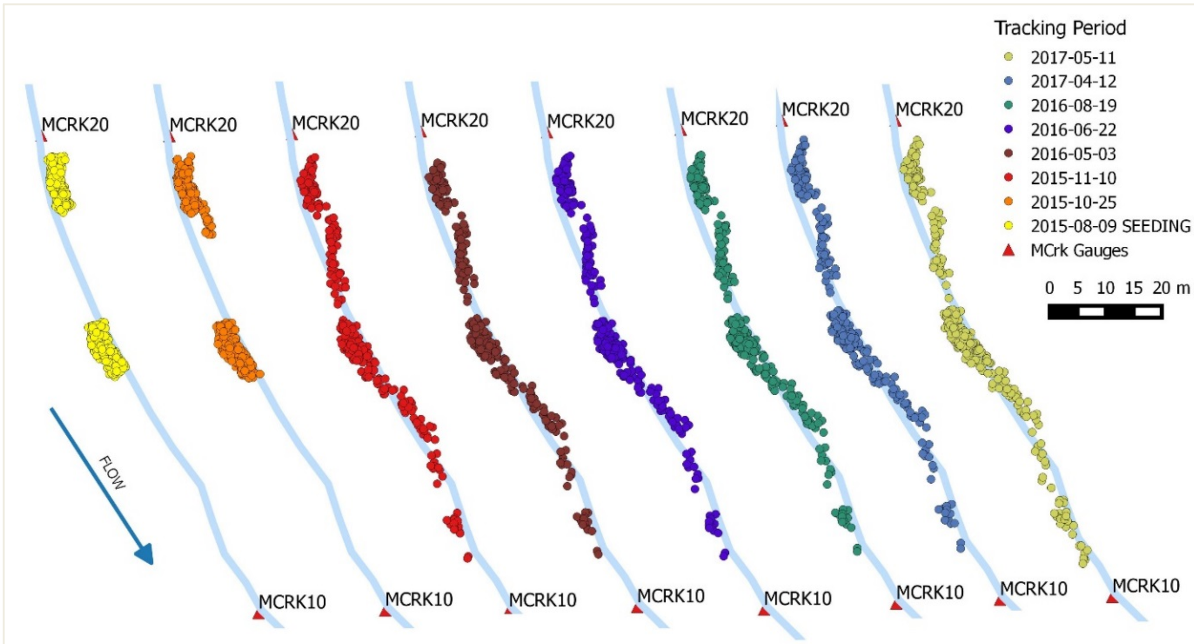
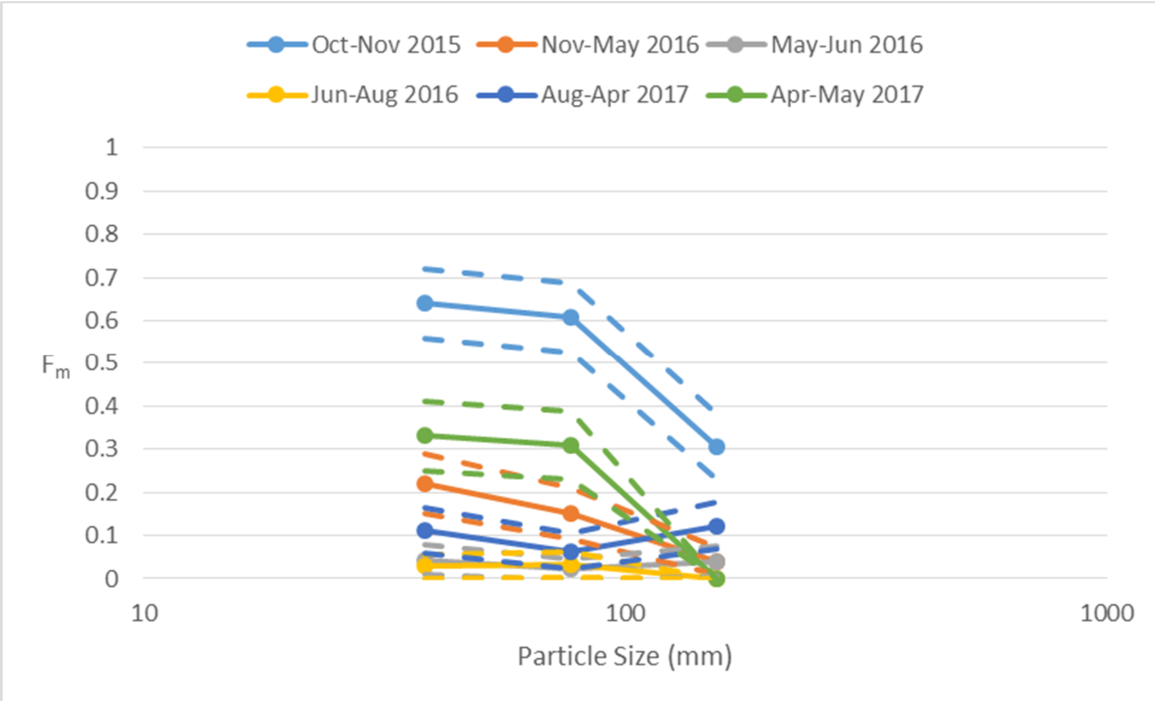


Figure 21: Visual overview of particle locations and movement over time. The blue line indicates the surveyed thalweg of the study reach.

4.2.1 Mobility and Travel distance with respect to grain size

Based on the tracking results, correlations could be established between the bedload sediment transport and the particle grain sizes (Figure 22). There is a negative correlation between grain size and both F_m and $\bar{L}$. This suggests that not only do the smaller particles become entrained more easily but they also travel further once transport is initiated. The low mobility rate and short travel distances experienced in this tracking period could then allow for the confidence intervals associated with the data to explain the variation to the trend.

Focusing on the two tracking periods with the greatest movement, October – November 2015 and April – May 2017, there is a strong trend between grain size and both F_m and $\bar{L}$. Both of these tracking periods were after significant rainfall events, exceeding Q_c. In both these events we see almost equal mobility rates for the D_{50} and D_{75} size classes, and partial mobility beyond the D_{75} size class as the D_{90} is significantly less mobile (Wilcock & McArdell, 1993). Interestingly, there was zero mobility of the largest size class in the April – May 2017 tracking period. This could be a result of the flow remaining below the threshold for the initiation of motion for these larger particles.



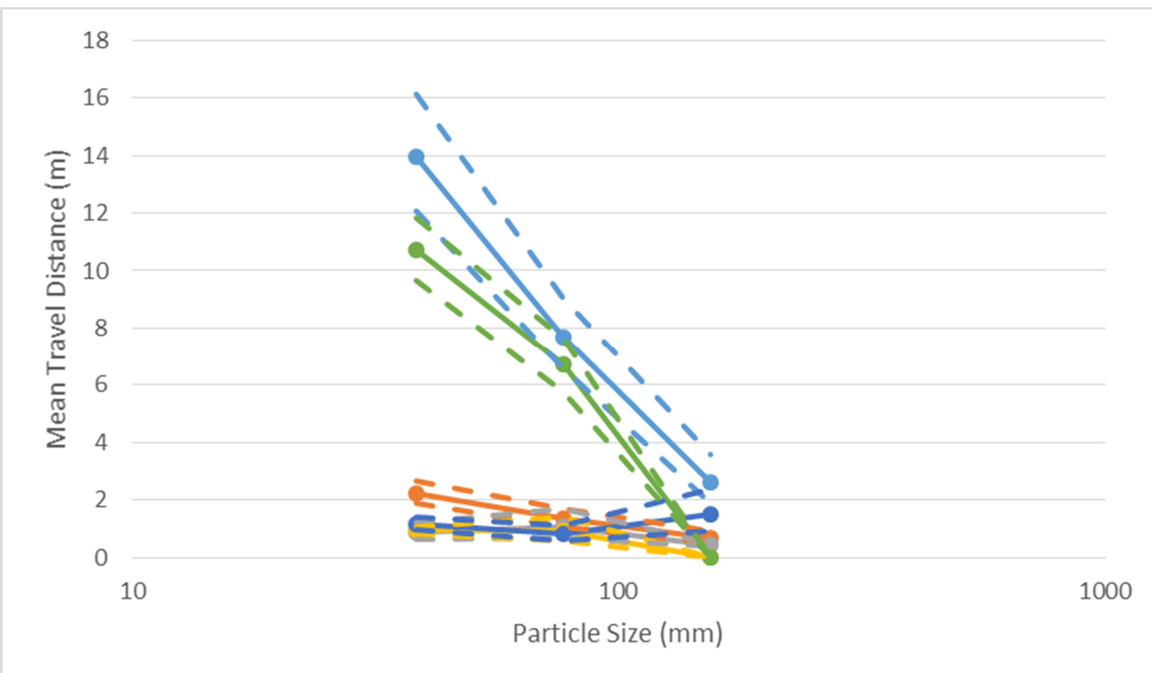


Figure 22: The mobile fraction and average travel distance plotted against grain size class for all tracking periods excluding the first (Aug to Oct, 2015). The dashed lines represent the 95% confidence intervals.

The relative travel distances and relative particle sizes were plotted in Figure 23 against the curve proposed by Church and Hassan (Equation 5). For the event scale tracking periods of Oct 26 – Nov 11, 2015 and Apr 12 – May 11, 2017, the data fits very well with the proposed relationship. The other tracking periods are below or very near the calculated threshold for particle entrainment. The movements in these tracking periods are likely influenced by local bed conditions.

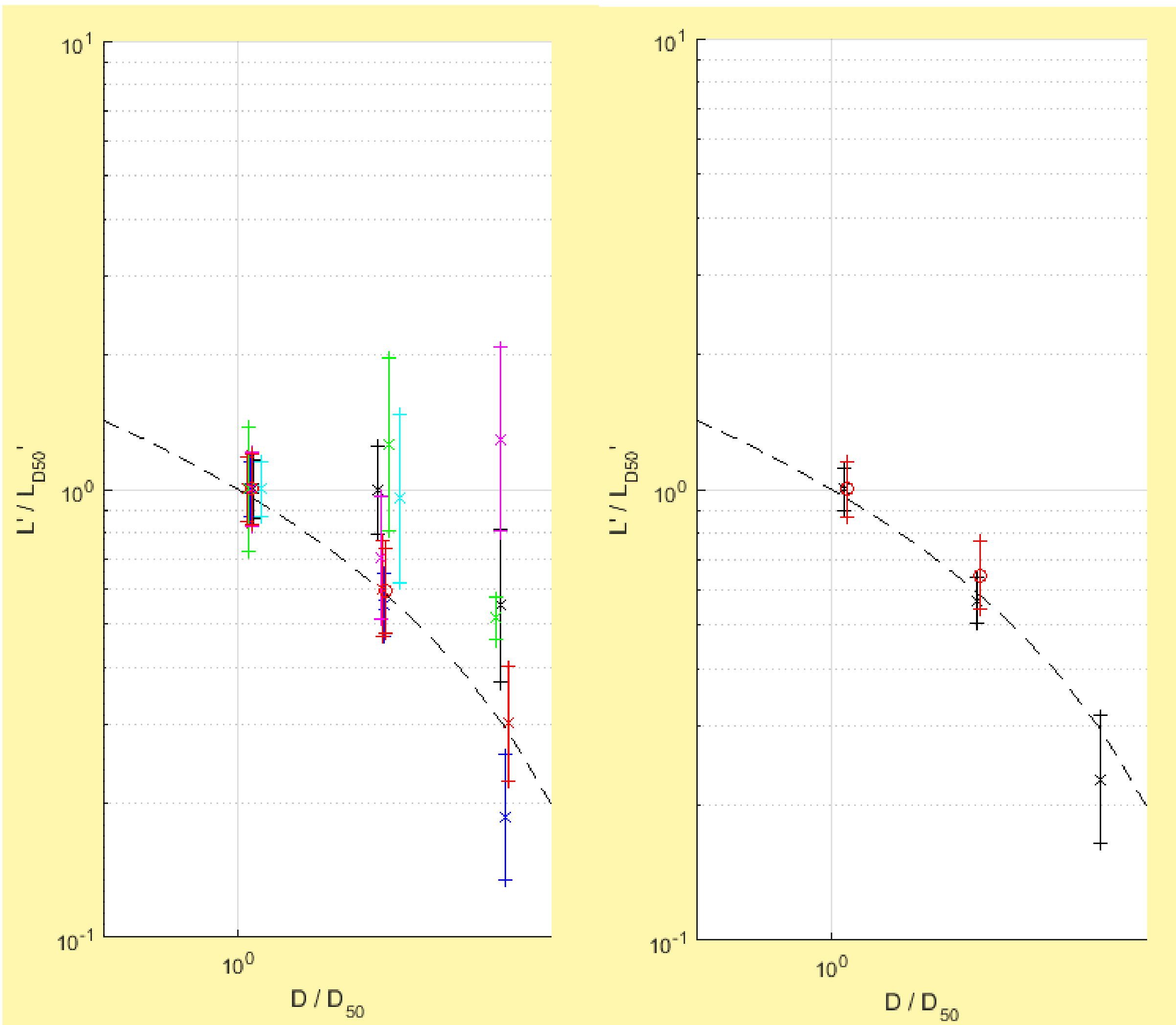


Figure 23: Relative travel distance for each size class as a function of relative grain size and their 95th percent confidence intervals. A) All the tracking periods B) Data for the singular captured event tracking periods, Oct 26 – Nov 11, 2015 and Apr 12 – May 11, 2017. The dashed line is Equation 5 (Church and Hassan, 1992).

4.3 Hydrometrics of Bedload Transport

The calculated hydrometrics are based on stage level recorded at MCRK 10 and the discharge rating curve. The results from MCRK 20 were skewed by the presence of a beaver dam for the first few months of the study, which created a small reservoir in the gauge location and as such gave artificially high water level readings. The upstream gauge HY048 is not representative of the flow conditions that the study reach would be subject to and so was not used in this analysis.

4.3.1 Field Established Threshold Values

The two tracking periods over the summer of 2016, from May 4 to June 22 and June 23 to Aug 19, had very minimal bedload sediment transport with particle mobility rates of 4% and 3% and mean travel distances of 0.9 m and 1.0 m, respectively. Based on the low rate of mobility and small travel distances, it was assumed that the flow was at its threshold or critical value. The peak flow, Q_p, occurred during the June 11, 2016 storm event. The peak flow depth was measured at both gauges, MCRK 20 and 10, at the upstream and downstream limits of the study reach respectively, and used to generate a corresponding shear stress. From the average of the two gauges, a critical shear stress threshold of 26.5 Pa was calculated for the study reach, below which it was assumed there would be no mobilization of the D_{50} size class or greater. This threshold equates to a dimensionless shear stress value τ^*_c of 0.043, based on a D_{50} = 38.5 mm, and a critical flow, Q_c = 0.99 m^3/s at MCRK 10 (Table 10). The dimensionless shear stress matches closely with typical threshold values for gravel bed rivers proposed by Buffington and Montgomery (1997) of $\tau^*_c \sim 0.030$ to 0.073 and Church (2006) of 0.045.

Table 10: Threshold values for particle initiation

Gauge	Max Depth	Hydraulic Radius	τ (Pa)	τ^*_c
MCRK 20	134.302	0.2530	25.315	0.0406
MCRK 10	135.205	0.2764	27.657	0.0444
AVERAGE	-	-	**26.486**	**0.0425**

Based on the hiding factor developed by Egiazaroff (1965), Equation 4 was used to calculate the critical shear stress and Shields parameter for the D_{75} and D_{90} size classes (Table 11).

Table 11: Results of hiding factor on critical shear stress

Size Class	Representative size (mm)	Unadjusted τ_c	Adjusted τ^*_c	Adjusted τ_c
D_{50}	38.5	26.486	0.0425	26.486
D_{75}	77.25	53.142	0.0278	34.758
D_{90}	154.5	106.285	0.0196	49.059

The calculated values correspond well with observed mobility rates in the field. Particles in the D_{50} and D_{75} size classes had $F_m > 0.1$, in the same tracking periods. However, $F_m < 0.1$ for the 2016 summer events, thus considered as being at the threshold and can be seen plotting below the critical threshold limit in Figure 24.Figure 24: Shields Diagram with the Egiazaroff hiding factor calculated based on a T*cg of 0.043. Field-measured values of peak shear were plotted for each tracking event. Particles in the D_{90} size class were mobilized in the October to Nov 2015 tracking period, and no movement in the summer of 2016 and May 2017, tracking periods. This corresponds with those events plotting below the critical threshold. The winter tracking periods were excluded from the plot, due to poor accuracy in measuring shear stress. Water level measurements during the winter months were impacted by the effects of ice and snow cover on the creek, resulting in poor estimates of shear stress.

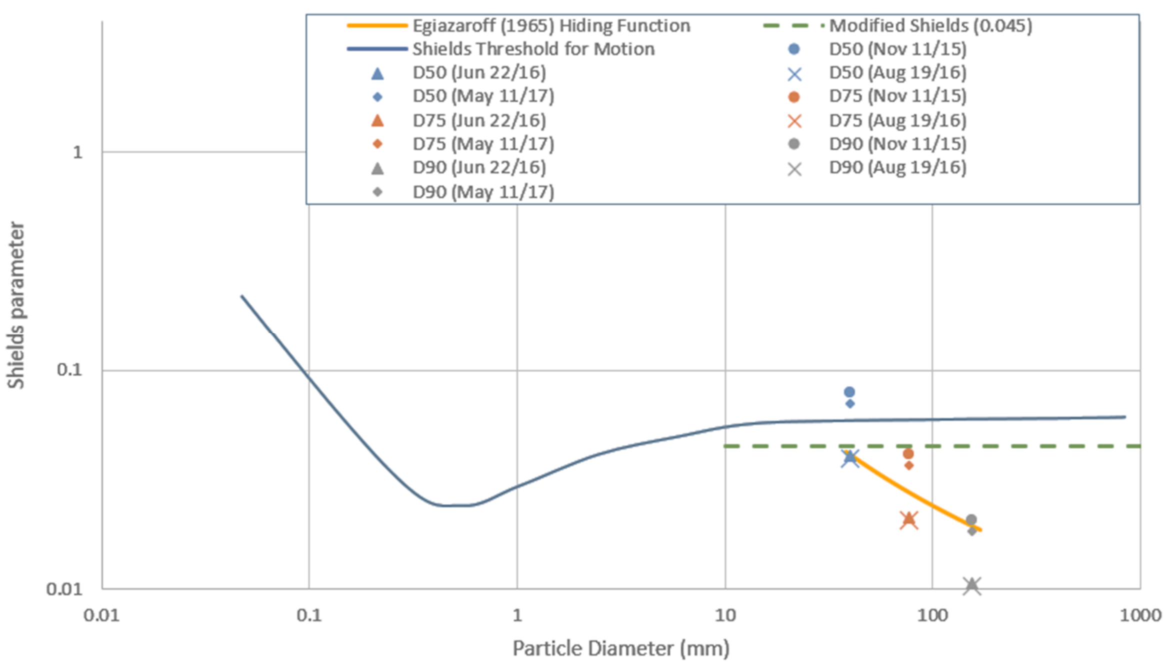


Figure 24: Shields Diagram with the Egiazaroff hiding factor calculated based on a T*cg of 0.043. Field-measured values of peak shear were plotted for each tracking event.

4.3.2 Relationships between hydrometrics and particle movement

The relationships between bedload sediment transport and the hydrometrics were analyzed. Each hydrometric was compared to both F_m and $\bar{L}$. Tracking periods that included the winter months were excluded from the plots, as there were multiple flow events and poor estimates of shear stress during these periods. The hydrometrics were calculated based on the water levels observed at the MCRK 10 gauge and are presented in Table 12.

Table 12: Hydrometrics calculated for each tracking period based on the water level measurements at MCRK 10 gauge. The winter tracking periods are highlighted.

Tracking Period	Total Precipitation (mm)	Q_p (m^3/s)	T_e (hr)	Wi (N/m)	CESS ($Pa^{1.5}$-hr/m)
Aug 09, 2015 – Oct 25 2015	172.6	0.998	2	648	2
Oct 26, 2015 – Nov 11 2015	71.6	3.162	35	665,533	35,696
Nov 12, 2015 – May 03, 2016	300.7	1.948	38	417,873	3082
May 04, 2016 – June 22, 2016	76.4	0.938	0	0	0
June 23, 2016 – Aug 19, 2016	49.8	0.898	0	0	0
Aug 19, 2016 – Apr 12, 2017	462.7	4.746	251	32,561,512	103,224
Apr 12, 2017 – May 11, 2017	134.8	2.609	22	522,849	14,202

4.3.2.1 Cumulative Effective Work Index (Wi)

The Wi was calculated for each tracking period using Equation 8. There was a positive trend between both F_m and $\bar{L}$ and the hydrometrics of peak flow and cumulative effective work for the data that was collected (Figure 25); however, only two periods had values greater than 0 for Wi, making it difficult to assess the significance of the relationships with either F_m or $\bar{\bar{L}}$.

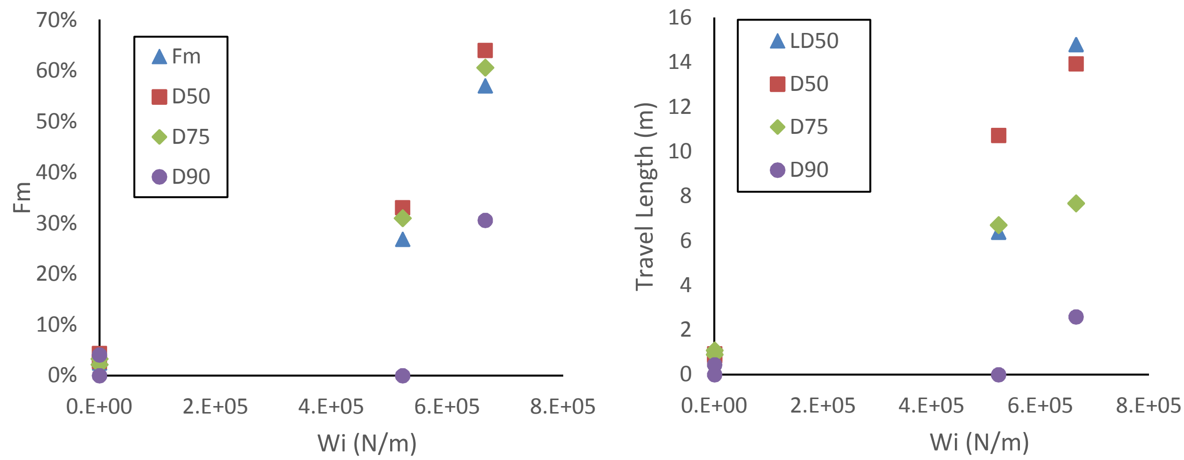


Figure 25: Wi vs. Mobility Rate and Particle Travel Distance

4.3.2.2 Cumulative Excess Shear Stress (CESS)

Following the work of DeVries (2000), the CESS was calculated for the two major tracking events, from October to November, 2015 and April to May, 2017. The CESS was calculated for each of the three tracer size classes, using the adjusted τ_c value, based on the hiding factor. The data obtained for Morningside Creek is plotted in Figure 26. The relationship proposed by DeVries (2000) for his field work at the Raging River and Issaquah Creek is displayed on the chart. The results from Morningside Creek plot well below DeVries' relationship, showing shorter travel distances for similar measured CESS. The mean travel distances were multiplied by F_m to provide a true mean travel distance including the non-mobile fraction. It should be noted that the study sites in DeVries' work were significantly larger, having 2 year flood peak flow rates more than 10 times larger, as well as having larger surface D_{50} particle sizes and steeper bed slopes. The relationship for particle travel length and CESS was modified and fit visually to the Morningside Creek data, with the following result:

$$\mathbf{L_{xi}} = \left[\mathbf{0.0001CESS_i} + \mathbf{4}\left(\mathbf{1} - (\mathbf{0.00015CESS_i} + \mathbf{1})^{-5}\right)\right]/\boldsymbol{F_m} \qquad (17)$$

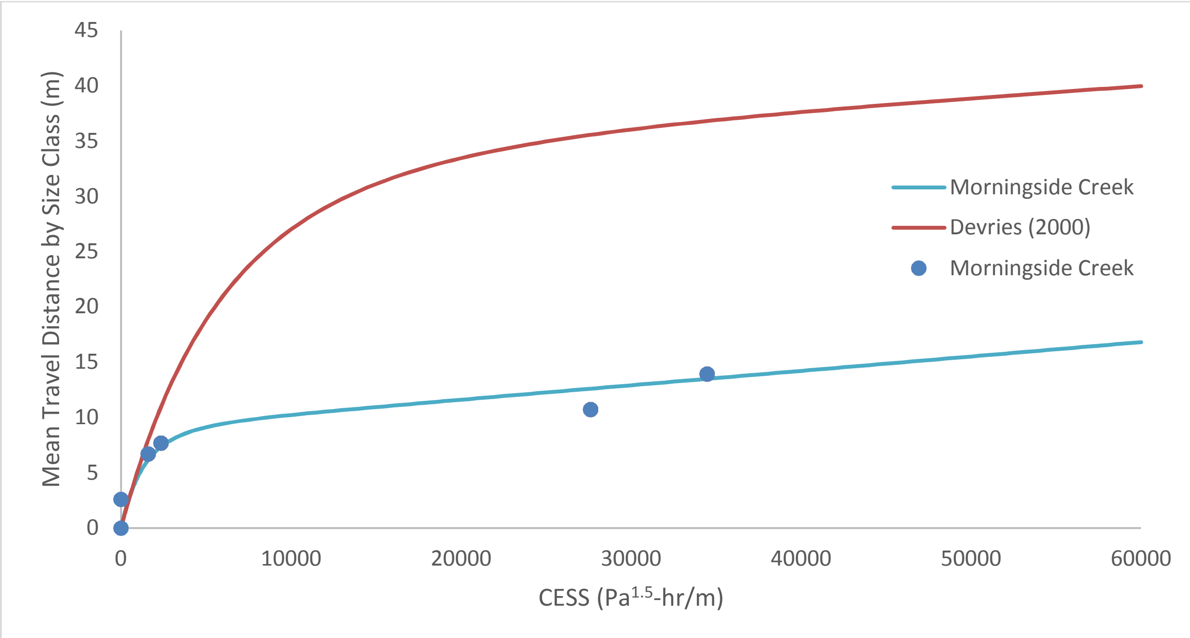


Figure 26: The relationship between average particle travel length and CESS. Field measured data is plotted against the function from the work of DeVries (2000). A new approximate relation was fit visually to the Morningside data.

The modified relationship proposed accounts for the non-mobile fraction by including the F_m term reducing the mean travel distances of each size class. As proposed by DeVries (2000) the F_m term was included in the equation to account for partial mobility. DeVries also suggested that a linear relationship would only be evident under fully mobile conditions. The non-linearity of the relationship is representative of partial mobility.

Chapter 5
Model Results

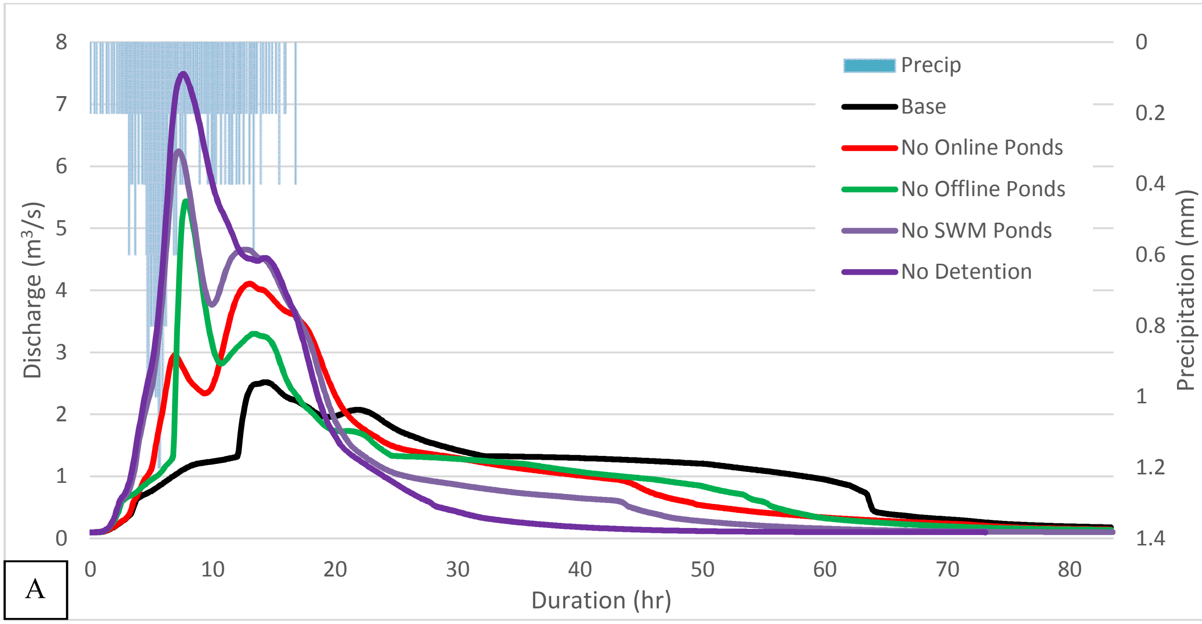
5.1 Modeled Scenarios

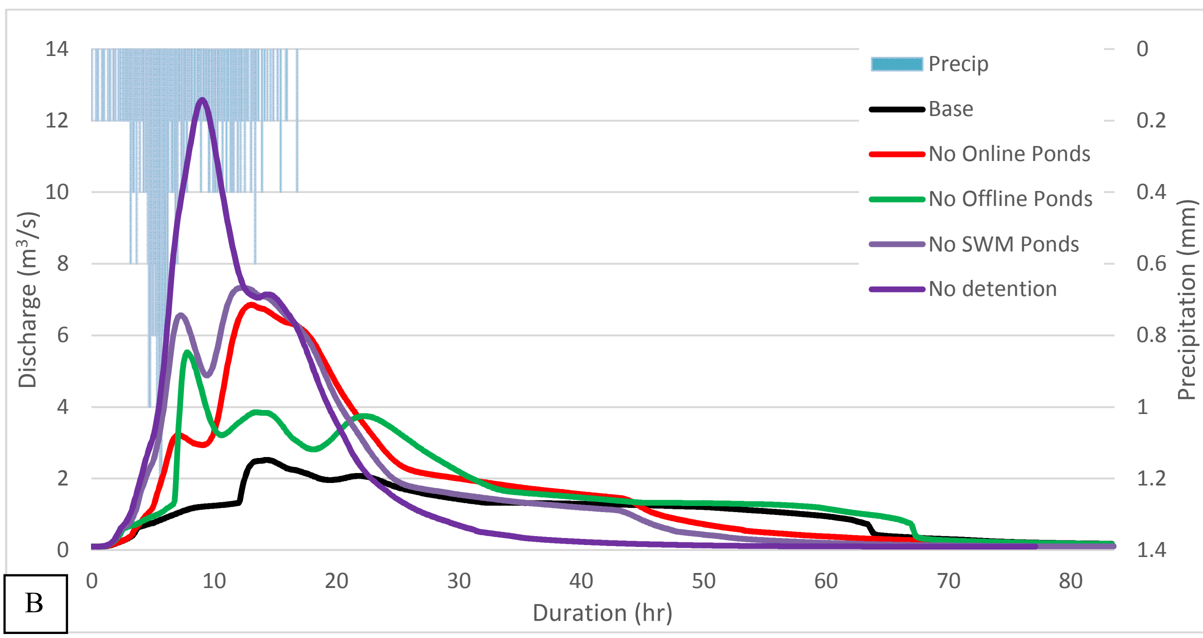
The calibrated hydrologic model of Morningside Creek was used as the base from which to run a number of scenarios. A number of scenarios were tested to establish the impacts of SWM features on the flow characteristics and bedload sediment transport in the study reach. Additional scenarios were run to determine the extent of impact due to urbanization and variable storm size.

5.1.1 Influence of SWM controls (Tests 1 and 2)

Tests 1 and 2 were run to establish the impact of the stormwater management facilities on the overall hydrology of the system. The scenarios varied the SWM controls present in the system by turning on and off the online ponds, offline ponds, natural detention and Tapscott diversion. In Test 1 the diversion structure is included as is and the following scenarios were tested: 1) the existing SWM strategy with all controls, 2) no online ponds, 3) no offline detention ponds, 4) no SWM ponds (offline or online), and 5) no natural detention or SWM ponds. In Test 2 the 5 scenarios were repeated while removing the diversion structure from the model. Each scenario was run with field measured precipitation from two unique storm events, October 28, 2015 and June 11, 2016. These two storms have two very different precipitation patterns, one with a long duration, low intensity storm (October 28) and the second with a short, high intensity event (June 11). The two storm events were used in the calibration of the hydrologic model and thus there is confidence in the model's output for these particular storms. The June 11, 2016 event, was also the threshold setting event, and allows for the assessment of the SWM features in near threshold events.

The results of the modelled scenarios in Test 1 and 2 are presented in Figure 27, with the dischrage hydrographs of each SWM scenario. In both storm events, the SWM controls reduce peak flows, as the elimination of each control results in an increase in peak flow up to as much as 3 to 5 times in the no detention, fully uncontrolled case. The controls do increase the flow duration, and in the October event, being the larger precipitation event, the time of exceedance is increased by the addition of the SWM ponds. From the results of Test 1, it is inconclusive whether the online or offline ponds provide greater peak flow reduction, as the two storms provide opposing results. In the October storm event the offline and online ponds reduce the peak flow by 54% and 39% respectively, while in the June storm event the reductions are 24% and 62%, respectively.


Precip
Base
No Online Ponds
No Offline Ponds
No SWM Ponds
No Detention
Discharge (m³/s)
Precipitation (mm)
Duration (hr)
A


Precip
Base
No Online Ponds
No Offline Ponds
No SWM Ponds
No detention
Discharge (m³/s)
Precipitation (mm)
Duration (hr)
B

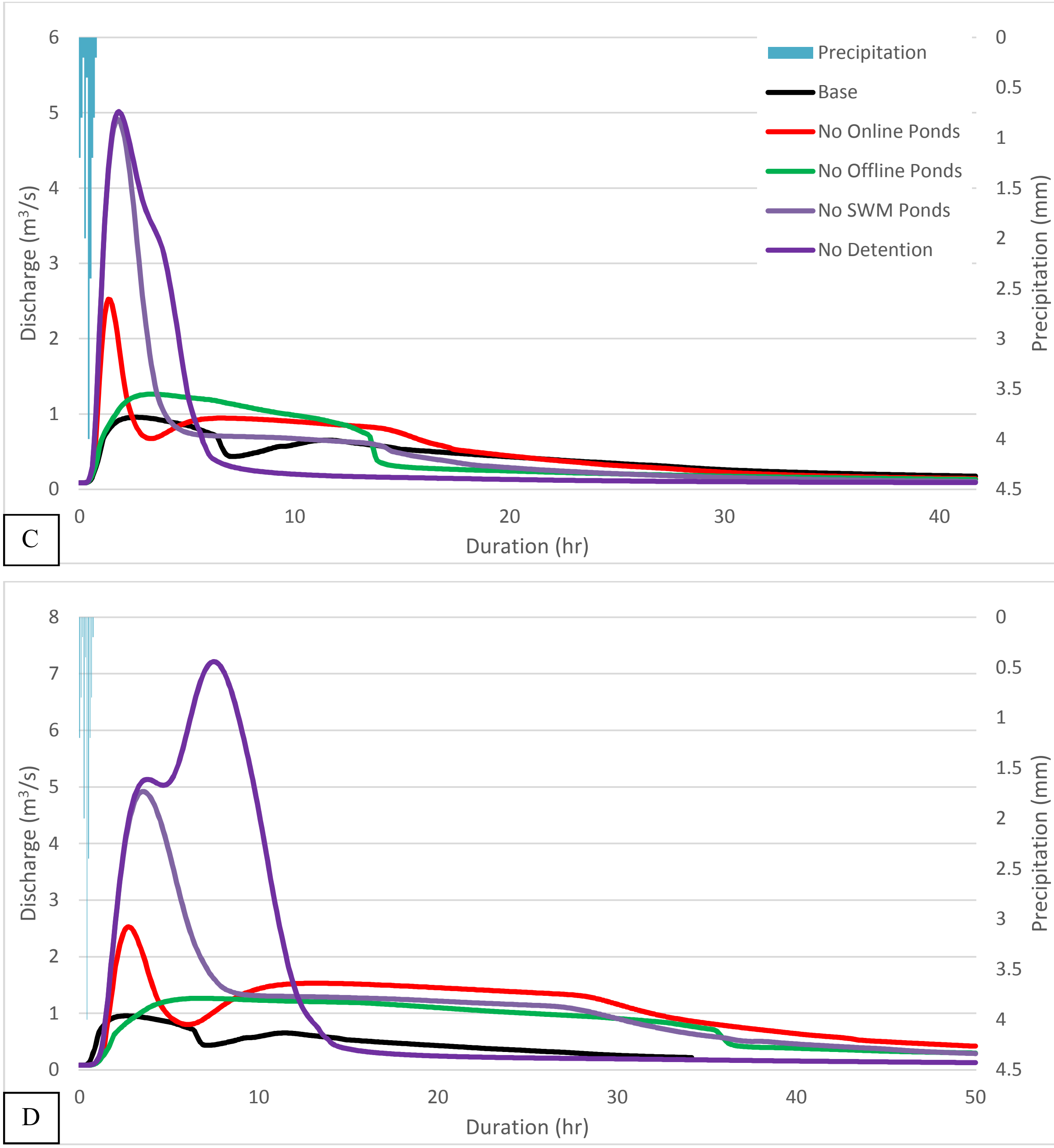


Figure 27: Modelled hydrograph results from the various SWM strategy scenarios in Test 1, with the diversion structure and Test 2, without the diversion: A) Test 1: October 28, 2015 B) Test 2: October 28, 2015 C) Test 1: June 11, 2016 and D) Test 2: June 11, 2016.

The percent reduction for each hydrometric was determined for all the SWM control cases in Tests 1 and 2 using the following equation:

$$\% \, Reduction = 1 - \frac{Result\ of\ scenario\ i}{Result\ of\ fully\ uncontrolled\ scenario}$$

where the fully uncontrolled scenario was for Test 2 (without the diversion structure) Scenario 5 (no SWM ponds or natural detention). In all scenarios, the results show the presence of the SWM features provide a positive reduction in Q_p, Wi, I_*, and CESS. In general, the presence of the SWM ponds have a greater impact on Q_p than the diversion. The percent reduction from the diversion is greater for the scenarios with the October storm, which has a larger channel flow compared to the June storm event. This is a result of the diversion structure diverting a larger percentage of the incoming flow as the flows in the channel increase. Figure 28 shows the percent reduction of each SWM scenario from Tests 1 and 2.

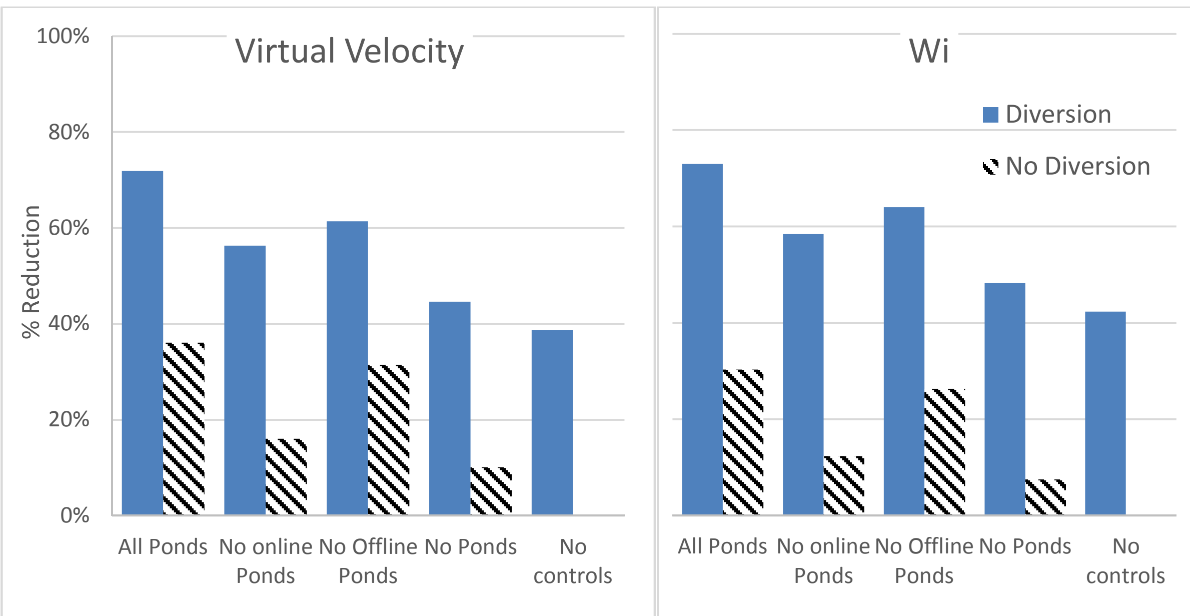


Figure 28: Results of percent reduction for virtual velocity and Wi from the modelled scenarios in Tests 1 and 2 for the October 28, 2015 storm event.

A closer look at four particular scenarios from Tests 1 and 2 provides a detailed breakdown of the effectiveness of specific facilities within the larger SWM strategy. Details of the SWM features present in each of the four scenarios is present in Table 13.

Table 13: Scenarios comparing the impacts of SWM detention features. The check implies the presence of the SWM feature in the particular scenario

Scenario	2-5	2-1	1-5	1-1
Presence of the Diversion			✓	✓
Presence of the Detention features		✓		✓

The first comparison of scenarios 2-5 and 2-1, quantifies the reduction from the SWM detention controls alone, without the influence of the diversion. The second comparison, 2-5 and 1-5, quantifies the reduction associated with the diversion structure alone, without the presence of any SWM controls. The third comparison, 2-5 and 1-1, quantifies the reduction from the current SWM strategy over a completely uncontrolled scenario. The fourth comparison, 2-1 and 1-1, quantifies the reduction associated with the diversion weir in the existing SWM strategy. In a similar manner, the fifth comparison, 1-5 and 1-1 quantifies the reduction associated with the SWM ponds in the existing SWM strategy. The results of the reductions of the different hydrometrics for the comparative analysis are presented in Table 14 and Figure 29.

Table 14: Comparing the reduction in hydrologic metrics associated with different SWM scenarios

	SWM Ponds only (2-5 and 2-1)	Diversion only (2-5 and 1-5)	SWM Ponds and the Diversion (existing case) (2-5 and 1-1)	Addition of Diversion when SWM controls present (2-1 and 1-1)	Addition of SWM Ponds when Diversion present (1-5 and 1-1)
October 28, 2015					
Q_p	67%	40%	80%	13%	40%
Wi	30%	42%	73%	43%	31%
I*	-6%	32%	51%	57%	19%
Runoff Ratio	-3%	38%	30%	33%	-8%
CESS	41%	44%	82%	41%	38%
Virtual Velocity	36%	39%	72%	36%	33%
June 11, 2016					
Q_p	87%	30%	87%	0%	56%
Wi	100%	46%	100%	0%	54%
I*	100%	38%	100%	0%	62%
Runoff Ratio	-15%	35%	7%	22%	-28%
CESS	100%	49%	100%	0%	51%
Virtual Velocity	100%	22%	100%	0%	78%

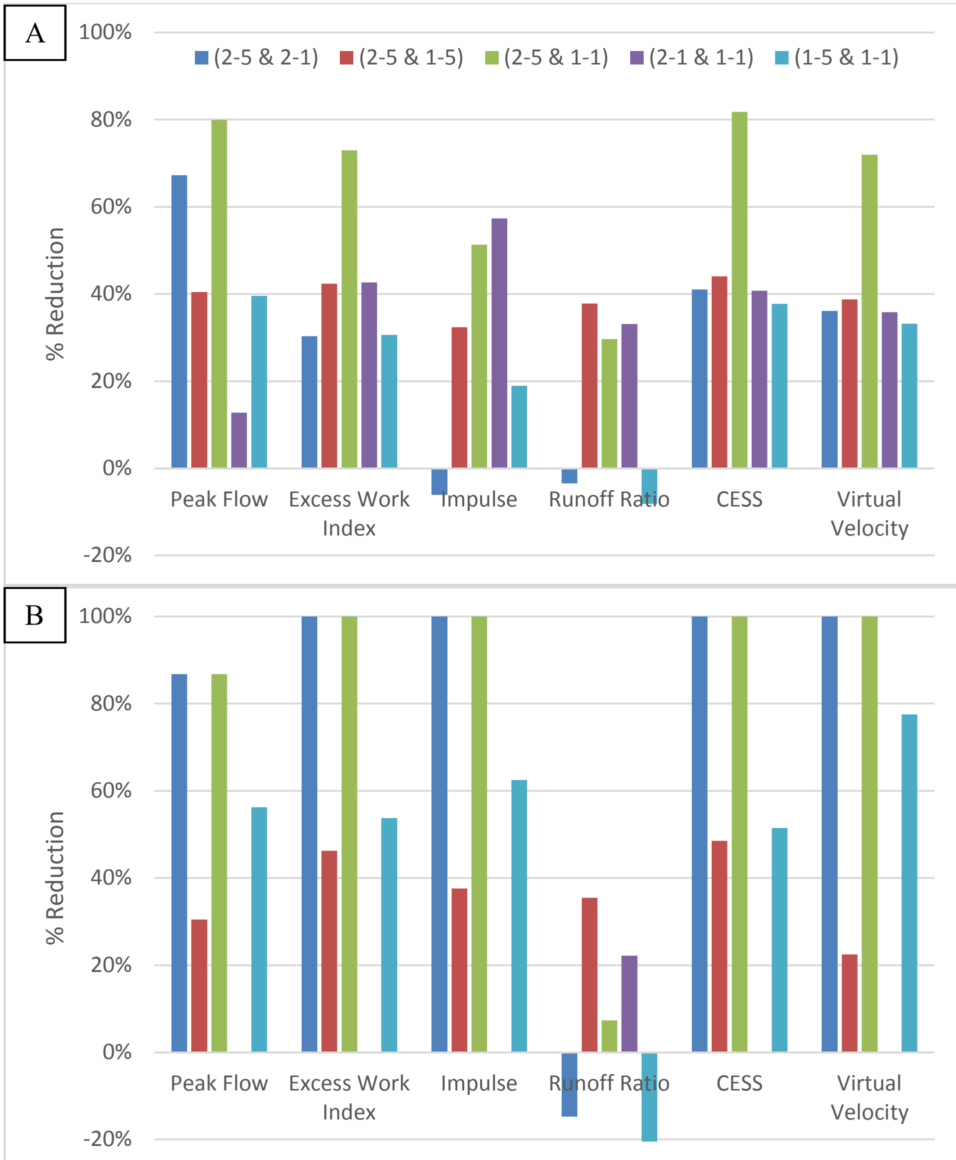


Figure 29: Relative reductions through the implementation of various SWM strategies for A) October 28, 2015 and B) June 11, 2016.

The SWM ponds provide a strong reduction in the Q_p, which is expected as it is the primary objective of the SWM pond feature. The addition of a volume reduction measure such as the diversion structure to a SWM strategy (2-1 and 1-1) had a greater impact on reducing the hydrometrics than did the addition of the SWM ponds (1-5 and 1-1). When the SWM ponds are present, the addition of the diversion still provides an additional reduction, in the event there is significant flow. The 0% reduction associated with the diversion seen in the June 11, 2016 event is due to the fact that the short, high intensity storm was sufficiently controlled by the SWM ponds, thus the flow was not significantly large enough to warrant the diversion structure. Interestingly, the impulse appears to increase slightly with the addition of the SWM ponds alone. This would be evidence in support of the findings of McCuen and Moglen (1988), whereby the addition of SWM detention increases the flow impulse. However, all the other hydrometrics still find a reduction in the strategy with the SWM ponds only.

5.1.2 Influence of urbanization on SWM effectiveness (Test 3)

Test 3 was designed to assess the robustness of the existing SWM strategy at Morningside Creek to increasing urbanization. The model mimicked an increase in urbanization by increasing the percent imperviousness of the currently developed sub-catchments. The NASH-HYD commands were uniformly adjusted to levels of imperviousness ranging from 20 – 90% for the varying model runs. The impervious percentages were adjusted simultaneously and homogenously, such that every subcatchment had the same value in each scenario. All the other features of the existing calibrated model, including all SWM features were left unaltered. The results of each hydrometric were normalized based on the modelled results from the existing scenario. The impervious factor was also a normalization of the scenarios impervious percentage with the existing scenario, which had a total imperviousness of 45%.

The results of the scenarios in Test 3 are shown in Figure 30. Due to the effects of the variable imperviousness in the subcatchments of the existing case compared to the homogeneous distribution in the Test 3 scenarios, the normalized results don't pass directly through 1 as would be expected.

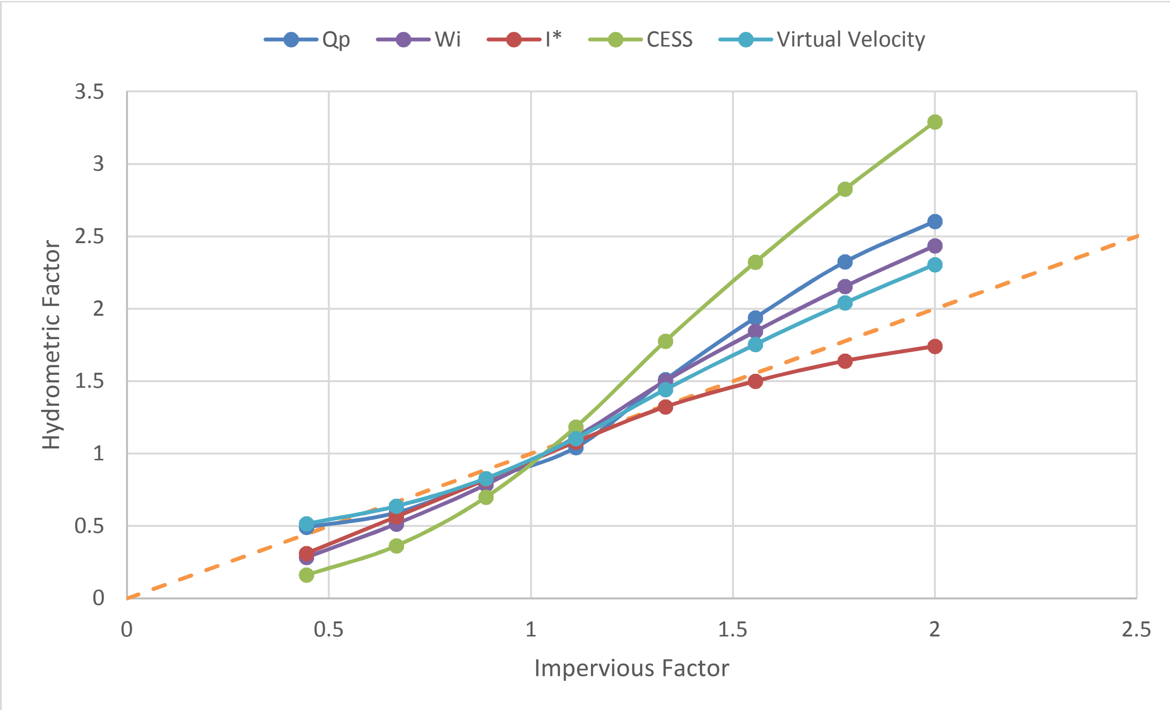


Figure 30: Results from the October 28 storm event in Test 3, showing the relationships between various hydrometrics with respect to increasing percent imperviousness as a factor of the existing case

In all cases, there is a positive correlation between percent imperviousness and the hydrometrics: Q_p, Wi, I_*, and CESS. As in the previous scenarios, virtual velocity was based on the excess shear equation and was an indicator of overall bulk sediment transport in the channel. The increasing trends of the hydrometrics above the 1:1 line, support the notion that SWM becomes less effective at higher levels of urban cover (Goff & Gentry, 2006). The only exception is the impulse hydrometric that plots below the 1:1 line for increasing levels of imperviousness. It should be noted that the SWM strategy developed in the Morningside catchment was for the existing level of urban development and so it should be expected that greater urbanization and hence greater runoff would lead to increases in the hydrometrics as the SWM features become overwhelmed and under designed.

5.1.3 Influence of Storm magnitude on SWM effectiveness (Test 4)

Test 4 was designed to model the response of the existing SWM system subject to larger storm events. The scenarios took two real storm events, of varying intensity and duration, and scaled either the intensity or duration in order to produce a series of rainfall events that would be a 2, 5, 10, and 25 year return storm (Figure 31). The two storms were selected to test different rainfall patterns; the October 28 storm is a long duration, low intensity event and the June 11 storm is a typical high intensity, low duration event. The model results for each created storm in Test 4 are presented in Table 15 and the hydrographs are plotted in Figure 32.

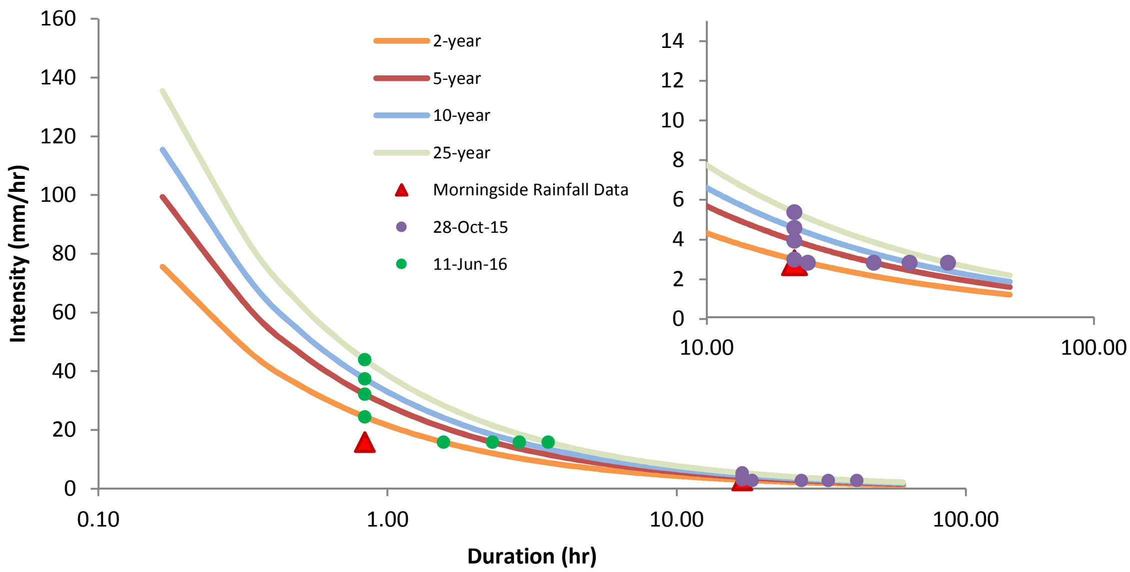


Figure 31: Extrapolation of duration and intensity for the October 28, 2015 and June 11, 2015 storm events to create scaled events of return period 2, 5, 10 and 25 years.

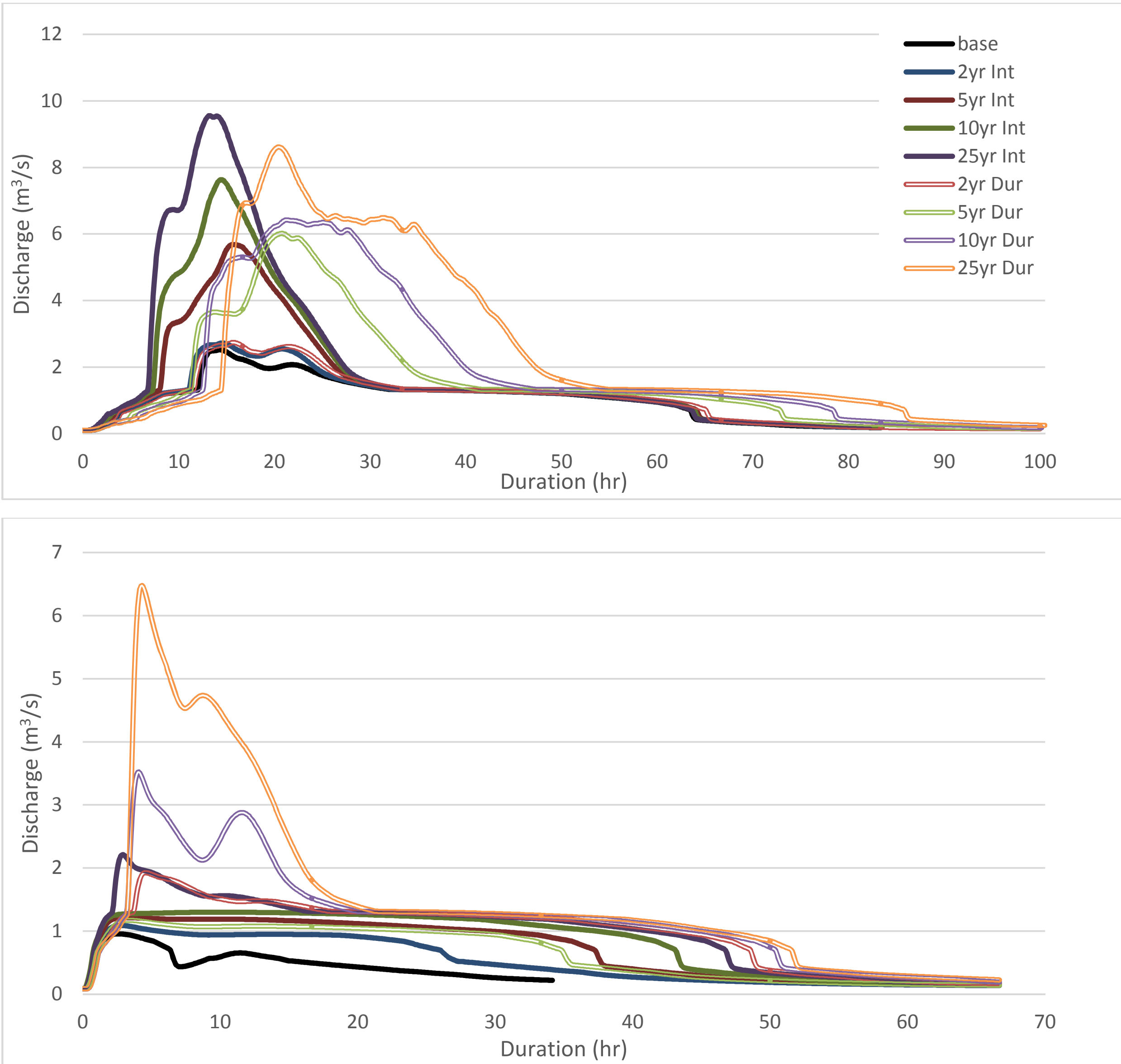


Figure 32: Hydrographs for scenarios in Test 4, for the A) October 28 and B) June 11 storms. The base model output is given in black, with the results of the extrapolated storms of the 2, 5, 10, 25 return year events with scaled intensities and durations.

Table 15: Model Results for Test 4

	Oct 28, 2015		June 11, 2016	
	Scaled Intensity	Scaled Duration	Scaled Intensity	Scaled Duration
2 year				
Storm Intensity (mm/hr)	3.00	2.84	24.54	15.84
Storm Duration (hr)	16.83	18.22	0.83	1.56
Q_p (m^3/s)	2.72	2.74	1.09	1.17
Wi (N/m)	1,194,853	86,910	11,979	86,910
I_*	27.98	29.16	0.26	2.11
CESS ($Pa^{1.5}$-hr/m)	69,392	73,794	175	1,639
Virtual Velocity (m/event)	10.94	11.38	1.12	6.22
5 year				
Storm Intensity (mm/hr)	3.95	2.84	32.26	15.84
Storm Duration (hr)	16.83	26.96	0.83	2.31
Q_p (m^3/s)	5.68	6.02	1.21	1.93
Wi (N/m)	2,489,955	3,153,620	167,598	542,614
I_*	45.42	55.77	4.45	14.18
CESS ($Pa^{1.5}$-hr/m)	177,165	228,814	4,612	23,798
Virtual Velocity (m/event)	21.72	26.88	8.95	12.09
10 year				
Storm Intensity (mm/hr)	4.59	2.84	37.49	15.84
Storm Duration (hr)	16.83	33.42	0.83	2.86
Q_p (m^3/s)	7.63	6.42	1.30	3.52
Wi (N/m)	3,193,494	4,403,884	328,879	1,040,480
I_*	52.82	71.49	8.92	23.51
CESS ($Pa^{1.5}$-hr/m)	236,871	331,743	11,531	62,532
Virtual Velocity (m/event)	27.69	37.17	10.44	17.13
25 year				
Storm Intensity (mm/hr)	5.38	2.84	43.96	15.84
Storm Duration (hr)	16.83	41.97	0.83	3.59
Q_p (m^3/s)	9.56	8.61	2.21	6.48
Wi (N/m)	3,934,956	5,983,729	580,658	1,888,652
I_*	59.41	90.32	14.93	34.21
CESS ($Pa^{1.5}$-hr/m)	299,720	461,184	26,909	133,658
Virtual Velocity (m/event)	33.97	50.12	16.76	26.38

The impact of increasing the duration or intensity seems to follow a power relation with CESS, (Figure 33). The IDF curve creates a power function for the relationship between intensity and duration, and it is important to note that in these graphs while one variable is being scaled for larger return periods, the other variable is also scaling as well. What can be derived from these plots is that the longer duration storms tend to create more effective work within the channel. The CESS increases with increasing storm duration, and consequently decreases with respect to increasing storm intensity.

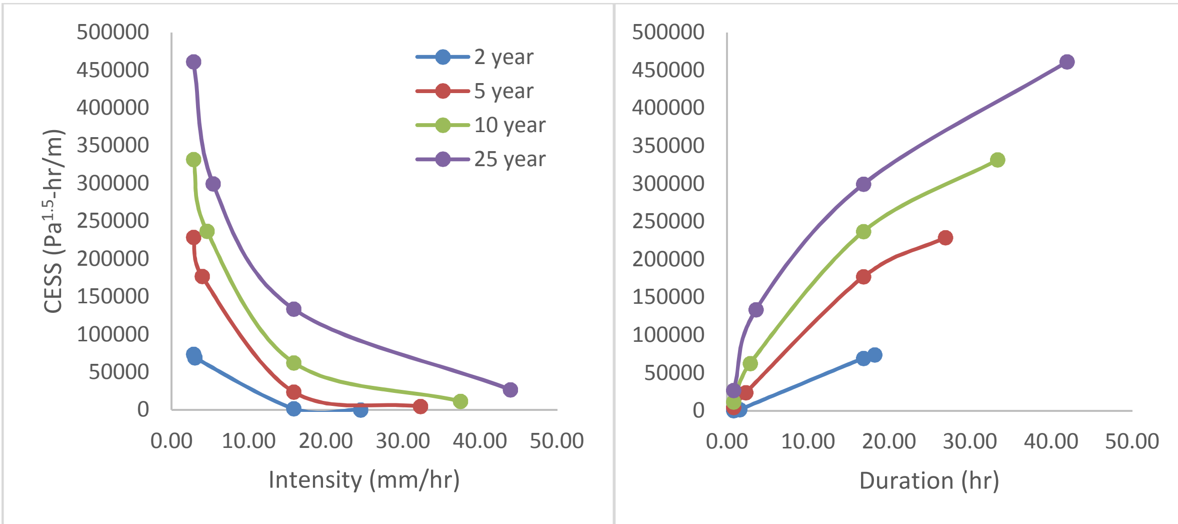


Figure 33: Results of Test 4, relationship of the CESS with storm intensity and storm duration, respectively.

Comparing the response of peak shear stress and CESS across the varying storm durations and intensities provides an interesting result (Figure 34). The peak flow for the Oct 28 events does not differ significantly during the intensity scaling or the duration scaling. The Oct 28 event has a greater peak shear stress in the scaled intensity storm of 8% and 4% in the 10 and 25 year storms, respectively. However, the opposite is true for CESS, where the scaled duration, is significantly greater than the scaled intensity storm, 54% greater in the 25 year event. In the June 11 storms, the duration causes significantly greater values in both peak and cumulative measures of shear stress. This is further evidence that the duration is more impactful in a SWM controlled system.

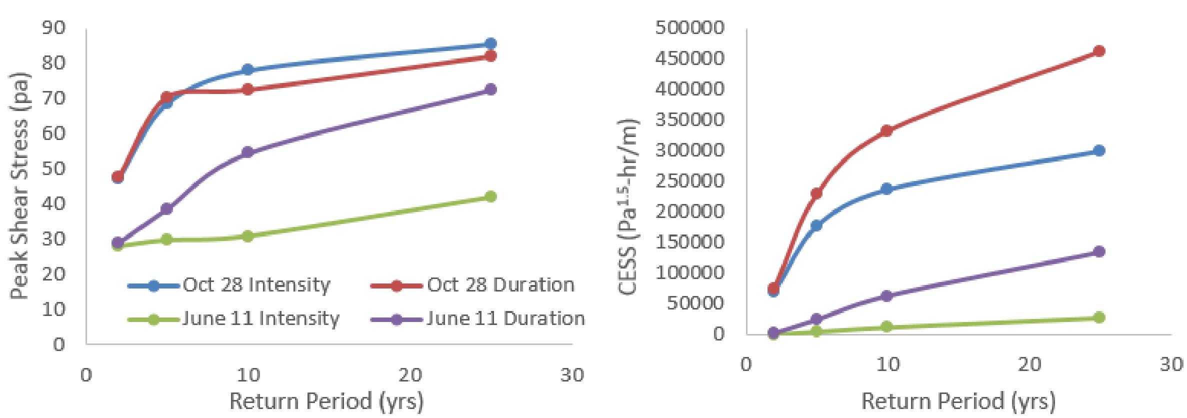


Figure 34: Results from Test 4, showing peak shear stress and CESS with respect to storm return period for variable intensities and durations.

Chapter 6
Discussion

6.1 Particle Dispersion

This study used RFID tracers to track event scale bedload sediment transport for the coarse grain fraction in Morningside Creek, an urban channel. Field measurements revealed a critical Shield's parameter of 0.043 derived from two events associated with minimal to no significant particle movement. This value fits well with the literature value for mixed sediments of a gravel-bed river of Buffington and Montgomery (1997) and Church (2006) of 0.045.

The Shields parameters for the larger D_{75} and D_{90} size classes were adjusted based on Egiazaroff's hiding function, and yielded 0.028 and 0.020, respectively. Whereas Shields' plot of critical shear stress yields much higher thresholds for particle movement, the hiding function of Egiazaroff matches the particle mobility measured in Morningside Creek.

In terms of particle travel distances, the relative travel distances of each of the three size classes plotted well against the Church and Hassan (1992) relationship for relative particle travel distance and relative grain size. Due to the low mobility rate, inclusion of the full particle tracer set would cause the geometric mean of all the particles to drop significantly. For this reason, the geometric mean travel length was determined based on the mobile particles only.

Studies have shown that the geomorphology of the channel influences particle travel distances (Pryce & Ashmore, 2003; Rice et al., 2009; Papangelakis & Hassan, 2016). From the results found in Morningside Creek for the two tracking periods with significant movement, the average geometric travel distance of the D_{50} size class was 14.8 and 6.4 m. The bankfull width of the channel within the study reach ranged between 4 m and 7 m and the spacing between pool-riffle sequences was roughly 30 m. This equates to a pool-riffle spacing of 4.5 to 7.5 times the channel width. Gregory et al. (1994) noted spacing between pool-riffle sequences is generally between 5 to 7 times the channel width, constant in both forested and urban catchments. Morningside Creek fits within this range, as well as having particle travel distances much smaller than the pool-riffle sequencing. It would appear from these results that the travel distance was controlled by the flow rather than the geomorphology. This suggests that travel distances are better defined by the channel geomorphology at larger flows, such that the geomorphology limits very large travel distances. Due to an absence of larger storm events during the study period, there is no data available to confirm this hypothesis. The results found here would indicate that the mobility rate is a factor of critical shear stress, which is a function of particle size and bed structure, while the particle travel distance is a function of CESS, with riffle-pool spacing providing a probabilistic upper threshold on lengths. It should be noted that Morningside Creek does not fit within the typical riffle-pool categorization of channels, owing to its unusually shallow pools, no deeper than 1 to 1.5 m at bankfull depth. Additionally, areas of exposed bedrock were noticed within the channel, likely providing grade control and preventing down cutting or deepening of pools. These areas of exposed bedrock would likely impact the sediment routing, providing significantly less resistance to bedload movement; however, these bedrock areas were short in length (4-8 m) and made up a small fraction of the channel bed in the study reach.

Long term continual monitoring of the tracer stones is suggested to determine if particle travel distance is limited by bar-to-bar spacing, as seen in flume experiments by Pryce and Ashmore (2003). Larger flood events, achieving channel-forming flows would provide greater information as to whether bar spacing is the dominant path length. Particle tracking will continue as part of the broader project study objectives of the research group.

6.2 Model of Bedload Sediment Transport

One of the main objectives of the study was to find a link between the channel hydrometrics and bedload sediment transport. It has been shown that SWM impacts the hydrologic response of the channel and that bedload sediment transport is subject to the flow regime. However, the large variety of hydrometrics available in the literature show the lack of a strong indicator for predicting bedload sediment transport.

Many researchers, when developing relationships for travel distance, have looked at peak values such as maximum shear stress (Phillips & Jerolmack, 2014) and maximum specific stream power (Houbrechts et al., 2015). Using a singular peak value to characterize very complex flow events, such as those in Morningside Creek, provides very little understanding of the mechanics of bedload transport. Time integrated parameters such as impulse (Phillips et al., 2013) and CESS (DeVries, 2000) account for the magnitude and duration of competent flow.

Field measured data from Morningside Creek was used to modify a model of particle travel length proposed by DeVries (2000), creating the following relationship:

$$\mathrm{L_{xi}} = [0.0001\mathrm{CESS_i} + 4(1 - (0.00015\mathrm{CESS_i} + 1)^{-5})]/F_m$$

The particle path length was used in calculating virtual velocity for the median grain size of the channel bed. Virtual velocity is one of the key parameters used in the spatial integration method for calculating bulk sediment transport. The above equation was used in determining the impact of SWM facilities on bedload sediment transport using the generated scenarios in the hydrologic model developed for Morningside Creek.

In comparison to DeVries (2000) work, the modified equation generates smaller travel distances at corresponding CESS. It should be noted that DeVries' work was done in larger channels, with greater widths and steeper bed slopes. It is possible that by normalizing the results by channel width, the data from these studies would fit into a single relationship.

The confidence for the developed model may be low due to a very limited amount of field measured data. One of the primary concerns was the absence of larger storm events in the data set. A much larger sample size should be incorporated to increase confidence in the empirical relationship.

6.3 Calibration of the Hydrologic Model

The importance of model calibration is made evident when determining hydrometrics based on time integrals such as impulse and CESS. Common practice, whereby models are calibrated to peak values, is insufficient and does not provide context for the complex storm response. In Morningside Creek, the channel response is particularly complex due to the cumulative impacts of the SWM features within the catchment. Calibrating a model based on the Nash-Sutcliffe efficiency coefficient also has its limitations. The coefficient is calculated based on the comparison of the modelled and measured values at each moment in time. However, the high temporal resolution of flow monitoring at Morningside Creek, coupled with the flashy response of the urbanized catchment, the ability to perfectly match the timing of response for each unique storm event is nearly impossible, considering the number of variables. As seen in the validation of the Morningside Creek hydrology model, the Nash-Sutcliffe coefficient will score very poorly if the timing is off even by just one timestep. Therefore, in terms of predicting bedload sediment transport from a time integrated metric like CESS, it is more useful to model the correct hydrograph shape and magnitude than the absolute timing of peak flow. Best practices would be wise in adopting a standard to ensure that models can match a hydrometric that correlates well to erosion, such as cumulative effective work or cumulative excess shear stress.

The Morningside hydrology model was calibrated to two very distinct storm events. The model is capable of characterizing the “double peak” unique to Morningside Creek, and simulates the falling limb very well for the smaller storm event. However, the model was not capable of accurately representing the falling limb in the larger event, predicting flow values higher than measured, before dropping off quite suddenly. Much of the difficulty in modelling the storm events was caused by using real storm data, which carries with it errors in measurement. Furthermore, applying the assumption of homogeneous precipitation across the entire catchment is likely a misrepresentation of the system, which in turn yields inaccurate results.

6.4 Impact of SWM features

The hydrologic model provided details on the impact of SWM features on the channel response in Morningside Creek. Scenarios 1 and 2 were designed to isolate each SWM feature so as to study its impact on the various hydrometrics. This section of the discussion will focus on findings from the October 28 storm event, which exceeded the threshold for particle entrainment.

One significant finding for the October 28 storm event was in the calculation of impulse. The results showed that the detention ponds in isolation actually increase the impulse as compared to an uncontrolled system. This finding is supported by the work of McCuen and Moglen (1988) who showed an increase in impulse intensity as a result of adding detention to a development. Based on the work of Phillips et al. (2013), increases in impulse should correspond to an increase in particle travel distances, thus providing greater estimates of bedload sediment transport. In contrast, when the ponds are part of the larger system, the reduction in impulse attributed to the SWM ponds is 19%.

It is interesting to note the ability of the ponds in isolation to provide a positive reduction for the values of the other hydrometrics. This leads to the opposite conclusion one might draw from the impulse, in that detention ponds are here seen to reduce the channel’s transport potential, which in turn highlights the importance of choosing an appropriate metric when modelling bedload sediment transport.

Similar to findings by Booth and Jackson (1997), the detention ponds proved very effective in controlling peak flow, providing a reduction of 67% when considered in isolation. On the other hand, the Tapscott diversion proved more effective than the SWM ponds in reducing the metrics of cumulative effective work, impulse, and CESS. Based on the existing SWM strategy, the reduction in virtual velocity associated with the diversion structure was slightly greater than the SWM ponds. This provides support in favour of volume reduction over detention features; however, this conclusion should not be generalized, as it is dependent on the specific SWM strategy of the Morningside Creek catchment. The SWM offline detention ponds in the Morningside Heights subcatchment are designed for erosion control, meaning that the ponds are capable of capturing and releasing runoff associated with a 33 mm storm event over 48 hours. This is equivalent to approximately a 2 year return period. Since the Tapscott diversion weir is activated in higher flows, the importance of volume reduction becomes apparent in larger storm events. Pomeroy et al. (2008) note that due to the increase in runoff volume seen in urbanization, unless volume reduction controls are implemented, the time of exceedance will increase regardless of the presence or absence of SWM detention features. Looking at the cumulative effects of the existing SWM strategy shows a reduction in virtual velocity by over 70% compared to the uncontrolled case. Goff and Gentry (2006) found that fully developed catchments with detention were not able to maintain pre-development flows at all points within the channel. While it may not be possible to define pre-development flows at Morningside Creek, the ability of the diversion to reduce the volumetric flow rate with increasing storm runoff makes it necessary in achieving lower peak flows.

Chapter 7
Conclusions

A RFID tracer study was performed to track bedload sediment transport at Morningside Creek, in Toronto, Ontario. The results were incorporated into a Visual Otthymo hydrologic model for the catchment area, which was calibrated to real storm events in order to replicate the hydrologic responses measured in the field. The hydrologic model was used to analyze the impacts of the local SWM features on the channel's storm response and capacity to transport bedload sediment. The following observations were made:

- Partial transport of the coarse tail of the grain size distribution was observed for the two major mobility events. The relative travel distances of each of the D_{50}, D_{75} and D_{90} half phi size classes fit the non-linear relationship proposed by Church and Hassan (1992) indicating partial mobility.

- The field determined Shield's parameter for the 40 mm D_{50} stone class was 0.0425 similar to accepted literature values. Egiazaroff's hiding function (1965) provided a very strong fit in determining mobility thresholds for the larger size classes.

- Particle path length was fit to a model based on cumulative excess shear stress based on modifying a model developed in the work of DeVries (2000).

  $$\mathrm{L_{xi}} = [0.0001\mathrm{CESS_i} + 4(1 - (0.00015\mathrm{CESS_i} + 1)^{-5})]/F_m$$

  The model was generated using a limited data set void of any larger storm events providing limited confidence in the model. Further validation should be undertaken to increase confidence in the model.

- Detention ponds in the Morningside Creek catchment were shown to produce a 33% reduction in bedload sediment transport based on the calculation of virtual velocity. The diversion infrastructure improves the reduction capability of the SWM strategy, with increasing potential during larger storm events.

- For Morningside Creek and the existing SWM system, increasing the impervious percentage of the contributing subcatchments results in a linear increase in bulk sediment transport based on the calculation of virtual velocity.

- In the Morningside Creek catchment, for storm events of fixed return period, longer storm durations generate greater bulk sediment transport compared to storms of higher intensity.

Future work is recommended in increasing the temporal scale of the study, adding tracking events to the dataset to refine the model of particle travel distance develop. The findings should be placed in comparison to other systems with varying levels of SWM control and urban land cover to provide better understanding of the limitations of SWM infrastructure such as detention ponds. Consideration should be made to channel size for application of the particle length model based on cumulative excess shear stress. Due to the complex interactions and cumulative impacts associated with stormwater management features, further research is needed to fully understand the nature of these best management practices on bedload sediment transport.

Bibliography

Allan, J. C., R. Hart, and J. V. Tranquili. 2006. "The use of Passive Integrated Transponder (PIT) Tags to Trace Cobble Transport in a Mixed Sand-and-Gravel Beach on the High-Energy Oregon Coast, USA." *Marine Geology* 232 (1-2): 63-86.

Annable, W. K., V. G. Lounder, and C. C. Watson. 2011. "Estimating channel-forming Discharge in Urban Watercourses." *River Research and Applications* 27 (6): 738-753.

Annable, W. K., C. C. Watson, and P. J. Thompson. 2012. "Quasi-equilibrium Conditions of Urban gravel-bed Stream Channels in Southern Ontario, Canada." *River Research and Applications* 28 (3): 302-325.

Ashiq, M. 1999. "Role of Active Bed Width in Bedload transport–Coarse Bed Rivers." *Jaywardena Et.Al.(Eds) River Sedimentation, Balkema, Rotterdam*: 951-955.

Ashworth, Philip J. and Robert I. Ferguson. 1989. "Size-Selective Entrainment of Bed Load in Gravel Bed Streams." *Water Resources Research* 25 (4): 627.

Bagnold, Ralph Alger. 1966. *An Approach to the Sediment Transport Problem from General Physics* US government printing office.

Baker, D. W., C. A. Pomeroy, W. K. Annable, J. G. MacBroom, J. S. Schwartz, and J. Gracie. 2008. "Evaluating the Effects of Urbanization on Stream Flow and Channel Stability - State of Practice."

Baker, David B., R. P. Richards, Timothy T. Loftus, and Jack W. Kramer. 2004. "A New Flashiness Index: Characteristics and Applications to Midwestern Rivers and Streams." *JAWRA Journal of the American Water Resources Association* 40 (2): 503-522.

Beck, Nicole G., Gary Conley, Lisa Kanner, and Margaret Mathias. 2017. "An Urban Runoff Model Designed to Inform Stormwater Management Decisions." *Journal of Environmental Management* 193: 257-269.

Bell, Colin D., Sara K. McMillan, Sandra M. Clinton, and Anne J. Jefferson. 2016. "Hydrologic Response to Stormwater Control Measures in Urban Watersheds." *Journal of Hydrology* 541: 1488-1500.

Bledsoe, Brian P. and Chester C. Watson. 2001. "Effects of Urbanization on Channel Instability." *JAWRA Journal of the American Water Resources Association* 37 (2): 255-270.

Bledsoe, B. P. 2002. "Stream Erosion Potential and Stormwater Management Strategies." *Journal of Water Resources Planning and Management* 128 (6): 451-455.

Booth, Derek B. 1990. "Stream-channel Incision Following drainage-basin Urbanization." *JAWRA Journal of the American Water Resources Association* 26 (3): 407-417.

Booth, Derek B. 1991. "Urbanization and the Natural Drainage System--Impacts, Solutions, and Prognoses." University of Washington Water Center.

Booth, Derek B. and C. Rhett Jackson. 1997. "Urbanization of Aquatic Systems: Degradation Thresholds, Stormwater Detection, and the Limits of mitigation1." *JAWRA Journal of the American Water Resources Association* 33 (5): 1077-1090.

Buffington, John M. and David R. Montgomery. 1997. "A Systematic Analysis of Eight Decades of Incipient Motion Studies, with Special Reference to gravel-bedded Rivers." *Water Resources Research* 33 (8): 1993-2029.

Chin, A. 2006. "Urban Transformation of River Landscapes in a Global Context." *Geomorphology* 79 (3-4): 460-487.

Church, Michael and Marwan A. Hassan. 1992. "Size and Distance of Travel of Unconstrained Clasts on a Streambed." *Water Resources Research* 28 (1): 299-303.

Church, M. and M. A. Hassan. 2002. "Mobility of Bed Material in Harris Creek." *Water Resources Research* 38 (11): 191-1912.

Church, Michael. 2006. "Bed Material Transport and the Morphology of Alluvial River Channels." *Annual Review of Earth and Planetary Sciences* 34 (1): 325-354.

City of Toronto. 2006. "Wet Weather Flow Management Guidelines."

Colin, B. Phillips, L. Martin Raleigh, and J. Jerolmack Douglas. 2013. "Impulse Framework for Unsteady Flows Reveals Superdiffusive Bed Load Transport." *Geophysical Research Letters* 40 (7): 1328.

DeVries, Paul. "Scour in Low Gradient Gravel Bed Streams: Patterns, Processes and Implications for the Survival of Salmonid Embryos." PhD thesis, University of Washington, 2000.

DeVries, Paul. 2002. "Bedload Layer Thickness and Disturbance Depth in Gravel Bed Streams." *Journal of Hydraulic Engineering* 128 (11): 983-991.

DeVries, Paul, Miguel Wong, Steve Burges, and Tim Brown. "Bedload Particle Travel Distance Increases Linearly With Cumulative Excess Shear Stress." Poster presented at the AGU, San Francisco, California, December 11-15, 2006.

Diplas, Panayiotis, Clint L. Dancey, Ahmet O. Celik, Manousos Valyrakis, Krista Greer, and Tanju Akar. 2008. "The Role of Impulse on the Initiation of Particle Movement Under Turbulent Flow Conditions." *Science* 322 (5902): 717-720.

Doyle, M. W., J. M. Harbor, C. F. Rich, and A. Spacie. 2000. "Examining the Effects of Urbanization on Streams using Indicators of Geomorphic Stability." *Physical Geography* 21 (2): 155-181.

Du Boys, MP. 1879. "The Rhone and Streams with Movable Beds." *Annals Des Pontes Et Chaussees* 18: 22.

Dunne, Thomas and Luna B. Leopold. 1978. *Water in Environmental Planning* Macmillan.

Einstein, Hans Albert. 1950. *The Bed-Load Function for Sediment Transportation in Open Channel Flows*. Vol. 1026 US Department of Agriculture Washington DC.

Environment Canada. 2006. “Climate Normals 1971 to 2000.” Accessed from http://www.statcan.gc.ca/tables-tableaux/sum-som/l01/cst01/phys08a-eng.htm.

Foster, GR, LD Meyer, and CA Onstad. 1977. "An Erosion Equation Derived from Basic Erosion Principles." *Transactions of the ASAE* 20 (4): 678-0682.

Fusillo, Thomas V., GH Nieswand, and TB Shelton. 1977. "Sediment Yields in a Small Watershed Under Suburban Development."

Galster, J. C., F. J. Pazzaglia, and D. Germanoski. 2008. "Measuring the Impact of Urbanization on Channel Widths using Historic Aerial Photographs and Modern Surveys." *Journal of the American Water Resources Association* 44 (4): 948-960.

Garcia, Marcelo. 2008. "Sedimentation Engineering: Processes, Measurements, Modeling, and Practice." American Society of Civil Engineers.

Goff, Karen M. and Randall W. Gentry. 2006. "The Influence of Watershed and Development Characteristics on the Cumulative Impacts of Stormwater Detention Ponds." *Water Resources Management* 20 (6): 829-860.

Gordon Wolman, M. and John P. Miller. 1960. "Magnitude and Frequency of Forces in Geomorphic Processes." *The Journal of Geology* 68 (1): 54-74.

Graf, William L. 1977. "Network Characteristics in Suburbanizing Streams." *Water Resources Research* 13 (2): 459-463.

Grant, Gordon E., James E. O'Connor, and M. Gordon Wolman. 2013. "A River Runs through it: Conceptual Models in Fluvial Geomorphology."

Gregory, K. J., A. M. Gurnell, C. T. Hill, and S. Tooth. 1994. "Stability of the Pool-Riffle Sequence in Changing River Channels." *Regulated Rivers: Research & Management* 9 (1): 35-43.

Haschenburger, Judith K. and Michael Church. 1998. "Bed Material Transport Estimated from the Virtual Velocity of Sediment." *Earth Surface Processes and Landforms* 23 (9): 791-808.

Haschenburger, J. K. 2013. "Tracing River Gravels: Insights into Dispersion from a Long-Term Field Experiment." *Geomorphology* 200: 121-131.

Hassan, Marwan A. 1990. "Scour, Fill, and Burial Depth of Coarse Material in Gravel Bed Streams." *Earth Surface Processes and Landforms* 15 (4): 341-356.

Hassan, Marwan A. and Peter Ergenzinger. 2003. "Use of Tracers in Fluvial Geomorphology." *Tools in Fluvial Geomorphology*: 397-423.

Hassan, M. A., M. Church, and A. P. Schick. 1991. "Distance of Movement of Coarse Particles in Gravel Bed Streams." *Water Resources Research* 27 (4): 503-511.

Hawley, Robert J., Katherine R. MacMannis, and Matthew S. Wooten. 2013. "Bed Coarsening, Riffle Shortening, and Channel Enlargement in Urbanizing Watersheds, Northern Kentucky, USA." *Geomorphology* 201: 111-126.

Hirsch, RM, JF Walker, JC Day, and R. Kallio. 1990. "The Influence of Man on Hydrologic Systems." *IN: Surface Water Hydrology.Geological Society of America, Boulder, Colorado.1990.p 329-359, 29 Fig, 7 Tab, 66 Ref.*

Hollis, GE. 1975. "The Effect of Urbanization on Floods of Different Recurrence Interval." *Water Resources Research* 11 (3): 431-435.

Houbrechts, Geoffrey, Yannick Levecq, Alexandre Peeters, Eric Hallot, Jean Van Campenhout, Anne-Cécile Denis, and François Petit. 2015. "Evaluation of Long-Term Bedload Virtual Velocity in Gravel-Bed Rivers (Ardenne, Belgium)." *Geomorphology* 251: 6-19.

Konrad, Christopher P., Derek B. Booth, and Stephen J. Burges. 2005. "Effects of Urban Development in the Puget Lowland, Washington, on Interannual Streamflow Patterns: Consequences for Channel Form and Streambed Disturbance." *Water Resources Research* 41 (7): W07009-n/a.

Ladson, Anthony R., Christopher J. Walsh, and Timothy D. Fletcher. 2006. "Improving Stream Health in Urban Areas by Reducing Runoff Frequency from Impervious Surfaces." *Australian Journal of Water Resources* 10 (1): 23-33.

Lamarre, Helene and Andre G. Roy. 2008. "The Role of Morphology on the Displacement of Particles in a step–pool River System." *Geomorphology* 99 (1): 270-279.

Lamarre, Helene, Bruce MacVicar, and Andre G. Roy. 2005. "Using Passive Integrated Transponder (PIT) Tags to Investigate Sediment Transport in Gravel-Bed Rivers." *Journal of Sedimentary Research* 75 (4): 736-741.

Lane, Emory Wilson. 1955. "Importance of Fluvial Morphology in Hydraulic Engineering." *Proceedings (American Society of Civil Engineers); v.81, Paper no.745*.

Laronne, JB, DN Outhet, PA Carling, and TJ McCabe. 1994. "Scour Chain Employment in Gravel Bed Rivers." *Catena* 22 (4): 299-306.

Legleiter, Carl J. and Phaedon C. Kyriakidis. 2006. "Forward and Inverse Transformations between Cartesian and Channel-Fitted Coordinate Systems for Meandering Rivers." *Mathematical Geology* 38 (8): 927-958.

Leopold, Luna B. 1968. "Hydrology for Urban Land Planning: A Guidebook on the Hydrologic Effects of Urban Land use." US Government Printing Office Washington, DC.

Leopold, Luna B. and M. Gordon Wolman. 81. "Miller] P. 1964. Fluvial Processes in Geomorphology." *San Francisco, California, Freeman* 522: 135-163.

MacRae, C. R. ‘‘A procedure for design of storage facilities for instream erosion control in urban streams.’’ PhD thesis, University of Ottawa, 1991.

MacRae, C. R. 1993. ‘‘An alternative design approach for the control of stream erosion potential in urbanizing watersheds.’’ 6th Int. Conf. on Urban Storm Drainage, Niagara Falls, Ontario.

MacRae, C. R. 1997. “Experience From Morphological Research on Canadian Streams Is Control of the Two Year Frequency Runoff Event the Best Basis for Stream Channel Protection.” Effects of watershed development and management of aquatic ecosystems, L. A. Roesner, ed., ASCE, Reston, Va., 144–162.

MacRae, CR and AC Rowney. 1992. "The Role of Moderate Flow Events and Bank Structure in the Determination of Channel Response to Urbanization."

MacVicar, Bruce, Margot Chapuis, Emma Buckrell, and André Roy. 2015. "Assessing the Performance of in-Stream Restoration Projects using Radio Frequency Identification (RFID) Transponders." *Water* 7 (10): 5566-5591.

Malcolm, H. Rooney and Charles Smallwood. 1977. "Sediment Prediction in the Eastern United States." *Journal of the Water Resources Planning and Management Division* 103 (2): 285-298.

McCuen, Richard H. 1974. "A Regional Approach to Urban Storm Water Detention." *Geophysical Research Letters* 1 (7): 321-322.

McCuen, R. H. 1979. "Downstream Effects of Stormwater Management Basins." *Journal of the Hydraulics Division, ASCE* 105 (HY11, Proc. Paper, 14977): 1343-1356.

McCuen, R. H. and G. E. Moglen. 1988. "Multicriterion Stormwater Management Methods." *Journal of Water Resources Planning and Management* 114 (4): 414-431.

Meyer-Peter, Eugen and R. Müller. 1948. "Formulas for Bed-Load Transport."IAHR.

Milan, David J. 2013. "Virtual Velocity of Tracers in a Gravel-Bed River using Size-Based Competence Duration." *Geomorphology* 198: 107-114.

Milan, David J. 2013. "Virtual Velocity of Tracers in a Gravel-Bed River using Size-Based Competence Duration." *Geomorphology* 198: 107-114.

Ministry of Natural Resources and Forestry. 2014. “Redside Dace.” Last modified December 9, 2015, https://www.ontario.ca/page/redside-dace.

Ministry of the Environment (MOE). 2003. “Stormwater management planning and design manual.” Ministry of the Environment, Toronto, Ontario.

Mobley, John T. and Teresa B. Culver. 2014. "Design of Outlet Control Structures for Ecological Detention Ponds." *Journal of Water Resources Planning and Management* 140 (2): 250-257.

Moscrip, Amy L. and David R. Montgomery. 1997. "Urbanization, Flood Frequency, and Salmon Abundance in Puget Lowland Streams." *JAWRA Journal of the American Water Resources Association* 33 (6): 1289-1297.

Nash, J. E. and J. V. Sutcliffe. 1970. "River Flow Forecasting through Conceptual Models Part I — A Discussion of Principles." *Journal of Hydrology* 10 (3): 282-290.

Nathan Bradley, D. and G. E. Tucker. 2012. "Measuring Gravel Transport and Dispersion in a Mountain River using Passive Radio Tracers." *Earth Surface Processes and Landforms* 37 (10): 1034-1045.

Nehrke, Seth M. and Larry A. Roesner. *Effect of Detention and BMPs on Flow Frequency of Runoff*.

Nelson, Erin J. and Derek B. Booth. 2002. "Sediment Sources in an Urbanizing, Mixed Land-use Watershed." *Journal of Hydrology* 264 (1): 51-68.

Nichols, MH. 2004. "A Radio Frequency Identification System for Monitoring Coarse Sediment Particle Displacement." *Applied Engineering in Agriculture* 20 (6): 783.

Nikora, Vladimir, Helmut Habersack, Thomas Huber, and Ian McEwan. 2002. "On Bed Particle Diffusion in Gravel Bed Flows Under Weak Bed Load Transport." *Water Resources Research* 38 (6): 1081.

Ontario Ministry of Natural Resources (MNR). 2011. "DRAFT Guidance for Development Activities in Redside Dace Protected Habitat." Ontario Ministry of Natural Resources, Peterborough, Ontario.

Ontario Ministry of Natural Resources and Toronto Region Conservation Authority. 2011. "Draft Rouge Fisheries Management Plan." 4-99 – 4-110

Papangelakis, Elli and Marwan A. Hassan. 2016. "The Role of Channel Morphology on the Mobility and Dispersion of Bed Sediment in a Small gravel-bed Stream." *Earth Surface Processes and Landforms* 41 (15): 2191-2206.

Parker, Gary. 2008. "Transport of Gravel and Sediment Mixtures." *Sedimentation Engineering: Processes, Measurements, Modeling, and Practice* 110: 165-252.

Paul, M. J. and J. L. Meyer. 2001. "Streams in the Urban Landscape." *Annual Review of Ecology and Systematics* 32: 333-365.

Phillips, C. B. and D. J. Jerolmack. 2014. "Dynamics and Mechanics of Tracer Particles." *Earth Surface Dynamics Discussions* 2 (1): 429-476.

Poff, N. LeRoy, J. David Allan, Mark B. Bain, James R. Karr, Karen L. Prestegaard, Brian D. Richter, Richard E. Sparks, and Julie C. Stromberg. 1997. "The Natural Flow Regime." *Bioscience* 47 (11): 769-784.

Poff, N. L., Brian P. Bledsoe, and Christopher O. Cuhaciyan. 2006. "Hydrologic Variation with Land use Across the Contiguous United States: Geomorphic and Ecological Consequences for Stream Ecosystems." *Geomorphology* 79 (3): 264-285.

Pomeroy, C. A., N. A. Postel, P. O'Neill A., and L. A. Roesner. 2008. "Development of Storm-Water Management Design Criteria to Maintain Geomorphic Stability in Kansas City Metropolitan Area Streams." *Journal of Irrigation and Drainage Engineering* 134 (5): 562-566.

Pyrce, Richard S. and Peter E. Ashmore. 2003. "The Relation between Particle Path Length Distributions and Channel Morphology in Gravel-Bed Streams: A Synthesis." *Geomorphology* 56 (1): 167-187.

Rice, Stephen P., Church, M., Wooldridge C.L., and Hickin, E.J. 2009. "Morphology and Evolution of Bars in a Wandering Gravel-Bed River; Lower Fraser River, British Columbia, Canada." *Sedimentology* 56 (3): 709-736.

Roesner, Larry A., Brian P. Bledsoe, and Robert W. Brashear. 2001. "Are Best-Management-Practice Criteria really Environmentally Friendly?" *Journal of Water Resources Planning and Management* 127 (3): 150-154.

Rosburg, Tyler T., Peter A. Nelson, and Brian P. Bledsoe. 2017. "Effects of Urbanization on Flow Duration and Stream Flashiness: A Case Study of Puget Sound Streams, Western Washington, USA." *JAWRA Journal of the American Water Resources Association* 53 (2): 493-507.

Rosgen, David L. 1996. *Applied River Morphology* Wildland Hydrology.

Schaeffers Consulting Engineers. 1999. "Master Environmental Servicing Plan, Appendix K Surface Water Resources."

Schaeffers Consulting Engineers. 2001. "Stormwater Management Pond & Skeleton Services Addendum, Morningside Heights."

Schaeffers Consulting Engineers. 2002. "Valley Design Report Morningside & Neilson Tributaries".

Schneider, J., R. Hegglin, S. Meier, JM Turowski, M. Nitsche, and D. Rickenmann. 2010. "Studying Sediment Transport in Mountain Rivers by Mobile and Stationary RFID Antennas."Bundesanstalt für Wasserbau: Karlsruhe, Germany.

Schneider, Johannes M., Jens M. Turowski, Dieter Rickenmann, Ramon Hegglin, Sabrina Arrigo, Luca Mao, and James W. Kirchner. 2014. "Scaling Relationships between Bed Load Volumes, Transport Distances, and Stream Power in Steep Mountain Channels." *Journal of Geophysical Research: Earth Surface* 119 (3): 533-549.

Schumm, Stanley A. 1969. "River Metamorphosis." *Journal of the Hydraulics Division* 95 (1): 255-274.

Sear, D.A, M.W.E. Lee, P.A. Carling, R.J. Oakey, and MB Collins. 2003. "An Assessment of the Accuracy of the Spatial Integration Method (SIM) for Estimating Coarse Bed Load Transport in Gravel-Bedded Streams using Tracers." *Iahs Publication*: 164-171.

Sear, D. A. 1996. "Sediment Transport Processes in pool–riffle Sequences." *Earth Surface Processes and Landforms* 21 (3): 241-262.

Sharpe, DR. 1997. *Surficial Geology of the Greater Toronto and Oak Ridges Moraine Area, Southern Ontario* Geological Survey of Canada.

Shields, Albert. 1936. "Anwendung Der Aehnlichkeitsmechanik Und Der Turbulenzforschung Auf Die Geschiebebewegung." *PhD Thesis Technical University Berlin*.

Smith, B. K., J. A. Smith, M. L. Baeck, G. Villarini, and D. B. Wright. 2013. "Spectrum of Storm Event Hydrologic Response in Urban Watersheds." *Water Resources Research* 49 (5): 2649-2663.

Townsend, Colin R., Mike R. Scarsbrook, and Sylvain Dolédec. 1997. "The Intermediate Disturbance Hypothesis, Refugia, and Biodiversity in Streams." *Limnology and Oceanography* 42 (5): 938-949.

Trimble, S. W. 1997. "Contribution of Stream Channel Erosion to Sediment Yield from an Urbanizing Watershed." *Science* 278 (5342): 1442-1444.

Trudeau, M. P. and Murray Richardson. 2015. "Change in Event-Scale Hydrologic Response in Two Urbanizing Watersheds of the Great Lakes St Lawrence Basin 1969–2010." *Journal of Hydrology* 527: 1174-1188.

Trudeau, M. P. and Murray Richardson. 2016. "Empirical Assessment of Effects of Urbanization on Event Flow Hydrology in Watersheds of Canada's Great Lakes-St Lawrence Basin." *Journal of Hydrology* 541: 1456-1474.

United Nations, Department of Economic and Social Affairs, Population Division. 2015. "World Urbanization Prospects: The 2014 Revision." http://esa.un.org/unpd/wup/FinalReport/WUP2014-Report.pdf.

Vázquez-Tarrío, Daniel and Rosana Menéndez-Duarte. 2014. "Bedload Transport Rates for Coarse-Bed Streams in an Atlantic Region (Narcea River, NW Iberian Peninsula)." *Geomorphology* 217: 1-14.

Venditti, Jeremy G., Peter A. Nelson, Ryan W. Bradley, Dan Haught, and Alessandro B. Gitto. 2017. "Bedforms, Structures, Patches, and Sediment Supply in Gravel-Bed Rivers." *Gravel-Bed Rivers: Process and Disasters*: 439.

Walsh, C. J., A. H. Roy, J. W. Feminella, P. D. Cottingham, P. M. Groffman, and R. P. Morgan II. 2005. "The Urban Stream Syndrome: Current Knowledge and the Search for a Cure." *Journal of the North American Benthological Society* 24 (3): 706-723.

Whitaker, Gene A., Richard H. McCuen, and John Brush. 1979. "Channel Modification and Macroinvertebrate Community Diversity in Small Streams." *JAWRA Journal of the American Water Resources Association* 15 (3): 874-879.

Wilcock, Peter R. and Brian W. McArdell. 1997. "Partial Transport of a sand/gravel Sediment." *Water Resources Research* 33 (1): 235-245.

Wilcock, Peter R. and Brian W. McArdell. 1993. "Surface-Based Fractional Transport Rates: Mobilization Thresholds and Partial Transport of a Sand-Gravel Sediment." *Water Resources Research* 29 (4): 1297-1312.

Wolman, M. Gordon. 1967. "A Cycle of Sedimentation and Erosion in Urban River Channels." *Geografiska Annaler.Series A.Physical Geography*: 385-395.

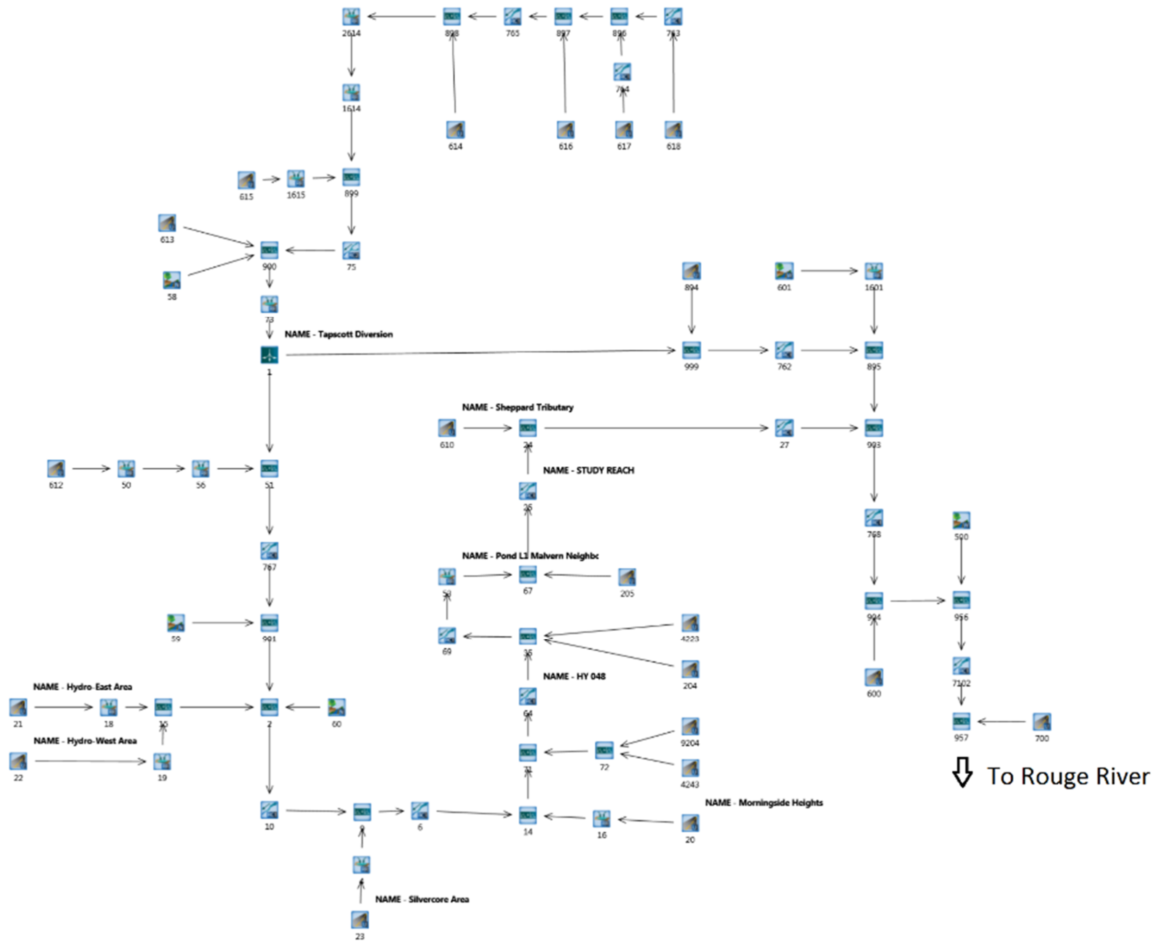
Wolman, M. Gordon. 1954. "A Method of Sampling Coarse river-bed Material." *EOS, Transactions American Geophysical Union* 35 (6): 951-956.

Wong, Miguel, Gary Parker, Paul DeVries, Timothy M. Brown, and Stephen J. Burges. 2007. "Experiments on Dispersion of Tracer Stones Under Lower-Regime Plane-Bed Equilibrium Bed Load Transport." *Water Resources Research* 43 (3): W03440.

Yager, EM and HE Schott. 2013. "9.7 the Initiation of Sediment Motion and Formation of Armor Layers." .

Appendix A

Visual Otthymo Output


NAME - Tapscott Diversion
NAME - Sheppard Tributary
NAME - STUDY REACH
NAME - Pond L1 Malvern Neighbc
NAME - HY 048
NAME - Hydro-East Area
NAME - Hydro-West Area
NAME - Morningside Heights
NAME - Silvercore Area
To Rouge River

```
==========================================================================================
=================

       V    V   I   SSSSS  U   U    A    L
       V    V   I   SS     U   U   A A   L
        V  V    I    SS    U   U  AAAAA  L
        V  V    I     SS   U   U  A   A  L
         VV     I   SSSSS  UUUUU  A   A  LLLLL

        OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO     TM
       O   O    T      T    H   H   Y Y   MM MM  O   O
       O   O    T      T    H   H    Y    M   M  O   O    Company

        OOO     T      T    H   H    Y    M   M   OOO                 Serial

Developed and Distributed by Clarifica Inc.
Copyright 1996, 2007 Clarifica Inc.
All rights reserved.

                  *****  D E T A I L E D   O U T P U T *****

  Input   filename: C:\Program Files (x86)\Visual Otthymo 2.4\VO2\voin.dat

  Output  filename:
C:\Users\RasoT\AppData\Local\Temp\31c99f01-c837-48de-ad6a-fc8b76397bcf\Scenario.out

  Summary filename:
C:\Users\RasoT\AppData\Local\Temp\31c99f01-c837-48de-ad6a-fc8b76397bcf\Scenario.sum


DATE: 07/31/2017                          TIME: 01:21:51

USER:


COMMENTS: ____________________________________________________________

------------------------------------------------------------------------------------------
-----------------
  ****************************
  ** SIMULATION NUMBER:   9 **
  ****************************


--------------------
|    READ STORM     |    Filename: C:\Users\RasoT\AppD
|                   |              ata\Local\Temp\
|                   |              31c99f01-c837-48de-ad6a-fc8b76397bcf\0f1ff51c
| Ptotal= 13.20 mm  |    Comments: Jun11/2016
--------------------
                 TIME    RAIN |  TIME    RAIN |' TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'  hrs   mm/hr |   hrs   mm/hr
                 0.08    0.00 |  0.33    2.40 |  0.58   48.00 |  0.83    9.60
                 0.17   14.40 |  0.42   24.00 |  0.67   28.80 |  0.92    2.40
                 0.25    9.60 |  0.50    4.80 |  0.75   14.40 |

-------------------------------------------------------------------------------

--------------------
| CALIB              |
| STANDHYD  (0700)   |   Area    (ha)= 307.23
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  24.70   Dir. Conn.(%)=  24.70
--------------------
                          IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     75.89       231.35
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.01         0.01
```

```
     Length            (m)=   1431.16      1431.00
     Mannings n          =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     21.60         0.79
               over (min)      30.00      2150.00
     Storage Coeff.  (min)=     92.68 (ii) 2146.55 (ii)
     Unit Hyd. Tpeak (min)=     30.00      2150.00
     Unit Hyd. peak  (cms)=      0.02         0.00
                                                        *TOTALS*
     PEAK FLOW       (cms)=      1.34         0.01        1.345 (iii)
     TIME TO PEAK    (hrs)=      1.08        36.42        1.08
     RUNOFF VOLUME    (mm)=     13.10         0.79        3.83
     TOTAL RAINFALL   (mm)=     13.20        13.20       13.20
     RUNOFF COEFFICIENT   =      0.99         0.06        0.29

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  76.7    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0600)   |   Area    (ha)= 822.24
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  20.90   Dir. Conn.(%)=  20.90
--------------------
                          IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=    171.85       650.39
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.44         0.44
     Length            (m)=   2341.28      2341.00
     Mannings n          =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     21.60         0.47
               over (min)      30.00      1135.00
     Storage Coeff.  (min)=     40.01 (ii) 1134.18 (ii)
     Unit Hyd. Tpeak (min)=     30.00      1135.00
     Unit Hyd. peak  (cms)=      0.03         0.00
                                                        *TOTALS*
     PEAK FLOW       (cms)=      5.40         0.03        5.404 (iii)
     TIME TO PEAK    (hrs)=      1.00        19.50        1.00
     RUNOFF VOLUME    (mm)=     13.10         0.47        3.11
     TOTAL RAINFALL   (mm)=     13.20        13.20       13.20
     RUNOFF COEFFICIENT   =      0.99         0.04        0.24

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  65.1    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| WILHYD    (0601)   |   Area    (ha)= 552.24   Curve Number   (CN)= 72.7
|ID= 1 DT= 5.0 min   |   Ia       (mm)=   5.00   Recession const.(K)= 1.55
--------------------     U.H. Tp(hrs)=   1.83

     U.H. peak       (cms)=  12.09

     PEAK FLOW       (cms)=   0.31 (i)
     TIME TO PEAK    (hrs)=   2.50
     RUNOFF VOLUME    (mm)=   0.64
     TOTAL RAINFALL   (mm)=  13.20
     RUNOFF COEFFICIENT   =   0.05

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
-------------------------------------------------------------------------------
--------------------
```

```
| RESERVOIR (1601) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |      OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000    |    2.2000     1.7880
                          0.0500     0.0017    |    3.4800     1.8760
                          0.1880     0.0265    |    9.1900     2.1400
                          0.5300     0.2125    |   12.0600     2.1410
                          0.9700     0.7171    |   12.0600     2.1500
                          1.5000     1.7000    |    0.0000     0.0000

                               AREA      QPEAK      TPEAK       R.V.
                               (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0601)   552.245     0.307       2.50        0.64
     OUTFLOW: ID= 1 (1601)   552.245     0.241       3.25        0.64

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 78.40
                   TIME SHIFT OF PEAK FLOW        (min)= 45.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0554

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0613)   |   Area    (ha)=  40.00
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  41.60   Dir. Conn.(%)=  41.60
--------------------
                          IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     16.64        23.36
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.42         0.42
     Length            (m)=    516.40       986.00
     Mannings n          =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     30.40         0.79
               over (min)      15.00       550.00
     Storage Coeff.  (min)=     14.29 (ii)  549.52 (ii)
     Unit Hyd. Tpeak (min)=     15.00       550.00
     Unit Hyd. peak  (cms)=      0.08         0.00
                                                        *TOTALS*
     PEAK FLOW       (cms)=      0.98         0.00        0.978 (iii)
     TIME TO PEAK    (hrs)=      0.75         9.75        0.75
     RUNOFF VOLUME    (mm)=     13.10         0.79        5.90
     TOTAL RAINFALL   (mm)=     13.20        13.20       13.20
     RUNOFF COEFFICIENT   =      0.99         0.06        0.45

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  76.7    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0058)   |   Area    (ha)=  59.90   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min   |   Ia       (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------     U.H. Tp(hrs)=   0.50

     Unit Hyd Qpeak  (cms)=   4.576

     PEAK FLOW       (cms)=   0.054 (i)
     TIME TO PEAK    (hrs)=   1.167
     RUNOFF VOLUME    (mm)=   0.311
     TOTAL RAINFALL   (mm)=  13.200
     RUNOFF COEFFICIENT   =   0.024

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
```

```
| STANDHYD  (0616)   |   Area    (ha)= 101.39
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  44.00   Dir. Conn.(%)=  44.00
--------------------
                          IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     44.61        56.78
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.62         0.62
     Length            (m)=    822.14       822.00
     Mannings n          =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     26.40         0.79
               over (min)      20.00       445.00
     Storage Coeff.  (min)=     17.78 (ii)  444.75 (ii)
     Unit Hyd. Tpeak (min)=     20.00       445.00
     Unit Hyd. peak  (cms)=      0.06         0.00
                                                        *TOTALS*
     PEAK FLOW       (cms)=      2.31         0.01        2.308 (iii)
     TIME TO PEAK    (hrs)=      0.83         8.00        0.83
     RUNOFF VOLUME    (mm)=     13.10         0.79        6.20
     TOTAL RAINFALL   (mm)=     13.20        13.20       13.20
     RUNOFF COEFFICIENT   =      0.99         0.06        0.47

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  76.7    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0617)   |   Area    (ha)=  39.29
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  40.30   Dir. Conn.(%)=  40.30
--------------------
                          IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     15.83        23.45
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.76         0.76
     Length            (m)=    511.78       512.00
     Mannings n          =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     38.40         0.79
               over (min)      10.00       315.00
     Storage Coeff.  (min)=     10.83 (ii)  313.18 (ii)
     Unit Hyd. Tpeak (min)=     10.00       315.00
     Unit Hyd. peak  (cms)=      0.11         0.00
                                                        *TOTALS*
     PEAK FLOW       (cms)=      1.08         0.01        1.076 (iii)
     TIME TO PEAK    (hrs)=      0.67         5.83        0.67
     RUNOFF VOLUME    (mm)=     13.10         0.79        5.74
     TOTAL RAINFALL   (mm)=     13.20        13.20       13.20
     RUNOFF COEFFICIENT   =      0.99         0.06        0.44

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  76.7    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0764)   |
| IN= 2---> OUT= 1   |     Routing time step (min)'=  5.00
--------------------
                  <------ DATA FOR SECTION (  64.0) ------>
                  Distance      Elevation        Manning
                    100.00        160.00         0.1200
                    104.00        158.27         0.1200
                    108.00        156.96         0.1200
                    109.00        156.54     0.1200 /0.0500  Main Channel
```

```
      110.50          156.30          0.0500      Main Channel
      112.00          156.43          0.0500      Main Channel
      112.50          156.90      0.0500 /0.1200  Main Channel
      120.00          157.13          0.1200
      130.00          158.90          0.1200
      150.00          160.00          0.1200

 <--------------------- TRAVEL TIME TABLE --------------------->
  DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
   (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
  0.12    156.42    .168E+03         0.0          0.32        67.73
  0.24    156.54    .593E+03         0.3          0.57        38.26
  0.45    156.75    .151E+04         1.1          0.93        23.24
  0.65    156.95    .267E+04         2.4          1.16        18.66
  0.85    157.15    .524E+04         4.4          1.10        19.67
  1.06    157.36    .884E+04         7.5          1.11        19.53
  1.26    157.56    .129E+05        11.6          1.17        18.58
  1.46    157.76    .174E+05        16.5          1.23        17.58
  1.67    157.97    .224E+05        22.4          1.30        16.68
  1.87    158.17    .279E+05        29.3          1.36        15.89
  2.07    158.37    .338E+05        37.1          1.43        15.19
  2.28    158.58    .402E+05        46.0          1.49        14.57
  2.48    158.78    .470E+05        55.9          1.55        14.01
  2.68    158.98    .543E+05        66.0          1.58        13.71
  2.89    159.19    .625E+05        76.4          1.59        13.65
  3.09    159.39    .719E+05        88.5          1.60        13.54
  3.29    159.59    .824E+05       102.4          1.62        13.41
  3.50    159.80    .939E+05       118.0          1.63        13.26
  3.70    160.00    .107E+06       135.6          1.65        13.10

                                   <---- hydrograph ---->   <-pipe / channel->
                             AREA    QPEAK    TPEAK   R.V.   MAX DEPTH   MAX VEL
                             (ha)    (cms)    (hrs)   (mm)      (m)       (m/s)
 INFLOW : ID= 2 (0617)      39.29     1.08     0.67   5.74      0.45       0.93
 OUTFLOW: ID= 1 (0764)      39.29     0.55     0.92   5.74      0.32       0.66

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0618)   |   Area    (ha)= 111.19
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  45.00   Dir. Conn.(%)=  45.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      50.04         61.16
     Dep. Storage     (mm)=       0.10          5.00
     Average Slope     (%)=       0.49          0.49
     Length            (m)=     860.99        861.00
     Mannings n          =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      26.40          0.79
                over (min)       20.00        495.00
     Storage Coeff.  (min)=      19.62 (ii)   490.74 (ii)
     Unit Hyd. Tpeak (min)=      20.00        495.00
     Unit Hyd. peak  (cms)=       0.06          0.00
                                                             *TOTALS*
     PEAK FLOW       (cms)=       2.47          0.01          2.470 (iii)
     TIME TO PEAK    (hrs)=       0.83          8.83          0.83
     RUNOFF VOLUME    (mm)=      13.10          0.79          6.33
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =       0.99          0.06          0.48

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0763) |
```

```
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  63.0) ------>
                 Distance      Elevation        Manning
                  100.00        160.00          0.1200
                  104.00        158.27          0.1200
                  108.00        156.96          0.1200
                  109.00        156.54      0.1200 /0.0500  Main Channel
                  110.50        156.30          0.0500      Main Channel
                  112.00        156.43          0.0500      Main Channel
                  112.50        156.90      0.0500 /0.1200  Main Channel
                  120.00        157.13          0.1200
                  130.00        158.90          0.1200
                  150.00        160.00          0.1200

 <--------------------- TRAVEL TIME TABLE --------------------->
  DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
   (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
  0.12    156.42    .158E+03         0.0          0.32        62.69
  0.24    156.54    .557E+03         0.3          0.57        35.41
  0.45    156.75    .142E+04         1.1          0.95        21.51
  0.65    156.95    .250E+04         2.4          1.18        17.27
  0.85    157.15    .492E+04         4.5          1.12        18.20
  1.06    157.36    .830E+04         7.6          1.12        18.08
  1.26    157.56    .121E+05        11.7          1.18        17.20
  1.46    157.76    .164E+05        16.8          1.25        16.27
  1.67    157.97    .211E+05        22.7          1.32        15.44
  1.87    158.17    .262E+05        29.7          1.38        14.71
  2.07    158.37    .318E+05        37.6          1.45        14.06
  2.28    158.58    .377E+05        46.6          1.51        13.48
  2.48    158.78    .441E+05        56.7          1.57        12.97
  2.68    158.98    .509E+05        66.9          1.60        12.69
  2.89    159.19    .587E+05        77.4          1.61        12.63
  3.09    159.39    .675E+05        89.7          1.62        12.53
  3.29    159.59    .773E+05       103.8          1.64        12.41
  3.50    159.80    .881E+05       119.7          1.66        12.27
  3.70    160.00    .100E+06       137.5          1.68        12.12

                                   <---- hydrograph ---->   <-pipe / channel->
                             AREA    QPEAK    TPEAK   R.V.   MAX DEPTH   MAX VEL
                             (ha)    (cms)    (hrs)   (mm)      (m)       (m/s)
 INFLOW : ID= 2 (0618)     111.19     2.47     0.83   6.33      0.66       1.18
 OUTFLOW: ID= 1 (0763)     111.19     1.83     1.00   6.33      0.56       1.06

-------------------------------------------------------------------------------
--------------------
| ADD HYD   (0896) |
|  1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0763):    111.19     1.831     1.00       6.33
        + ID2= 2 (0764):     39.29     0.553     0.92       5.74
          ==================================================
          ID = 3 (0896):    150.48     2.349     1.00       6.17

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| ADD HYD   (0897) |
|  1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0616):    101.39     2.308     0.83       6.20
        + ID2= 2 (0896):    150.48     2.349     1.00       6.17
          ==================================================
          ID = 3 (0897):    251.87     4.461     0.92       6.19

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
```

```
--------------------
| ROUTE CHN (0765) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  65.0) ------>
                 Distance      Elevation        Manning
                  100.00        160.00          0.1200
                  104.00        158.27          0.1200
                  108.00        156.96          0.1200
                  109.00        156.54      0.1200 /0.0500  Main Channel
                  110.50        156.30          0.0500      Main Channel
                  112.00        156.43          0.0500      Main Channel
                  112.50        156.90      0.0500 /0.1200  Main Channel
                  120.00        157.13          0.1200
                  130.00        158.90          0.1200
                  150.00        160.00          0.1200

 <--------------------- TRAVEL TIME TABLE --------------------->
  DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
   (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
  0.12    156.42    .417E+03         0.1          0.45       120.01
  0.24    156.54    .147E+04         0.4          0.79        67.79
  0.45    156.75    .375E+04         1.5          1.31        41.18
  0.65    156.95    .662E+04         3.3          1.63        33.07
  0.85    157.15    .130E+05         6.2          1.54        34.85
  1.06    157.36    .219E+05        10.6          1.55        34.61
  1.26    157.56    .320E+05        16.2          1.63        32.92
  1.46    157.76    .433E+05        23.2          1.73        31.15
  1.67    157.97    .557E+05        31.4          1.82        29.56
  1.87    158.17    .693E+05        41.0          1.91        28.16
  2.07    158.37    .840E+05        52.0          2.00        26.92
  2.28    158.58    .998E+05        64.4          2.08        25.81
  2.48    158.78    .117E+06        78.3          2.17        24.82
  2.68    158.98    .135E+06        92.4          2.21        24.29
  2.89    159.19    .155E+06       107.0          2.22        24.18
  3.09    159.39    .178E+06       124.0          2.24        23.99
  3.29    159.59    .204E+06       143.4          2.26        23.76
  3.50    159.80    .233E+06       165.4          2.29        23.50
  3.70    160.00    .265E+06       190.0          2.32        23.21

                                   <---- hydrograph ---->   <-pipe / channel->
                             AREA    QPEAK    TPEAK   R.V.   MAX DEPTH   MAX VEL
                             (ha)    (cms)    (hrs)   (mm)      (m)       (m/s)
 INFLOW : ID= 2 (0897)     251.87     4.46     0.92   6.19      0.73       1.59
 OUTFLOW: ID= 1 (0765)     251.87     2.63     1.25   6.18      0.57       1.49

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0614)   |   Area    (ha)= 450.63
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  40.40   Dir. Conn.(%)=  40.40
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     182.06        268.58
     Dep. Storage     (mm)=       0.10          5.00
     Average Slope     (%)=       0.65          0.65
     Length            (m)=    1733.27       1730.00
     Mannings n          =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      21.60          0.79
                over (min)       30.00        690.00
     Storage Coeff.  (min)=      29.72 (ii)   687.59 (ii)
     Unit Hyd. Tpeak (min)=      30.00        690.00
     Unit Hyd. peak  (cms)=       0.04          0.00
                                                             *TOTALS*
     PEAK FLOW       (cms)=       6.80          0.03          6.801 (iii)
     TIME TO PEAK    (hrs)=       1.00         12.08          1.00
     RUNOFF VOLUME    (mm)=      13.10          0.79          5.76
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =       0.99          0.06          0.44
```

```
     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| ADD HYD   (0898) |
|  1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0614):    450.63     6.801     1.00       5.76
        + ID2= 2 (0765):    251.87     2.634     1.25       6.18
          ==================================================
          ID = 3 (0898):    702.50     9.021     1.08       5.91

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (2614) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |       OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------        (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |    5.4330     8.9450
                           0.0550     2.6410    |    9.6740     9.4540
                           0.2450     3.4540    |   15.3290     9.9630
                           0.6180     4.7960    |   22.1250    10.4710
                           1.1140     5.5860    |   38.6530    11.4890
                           1.9930     7.9280    |   48.2480    11.9980

                               AREA      QPEAK      TPEAK        R.V.
                               (ha)      (cms)      (hrs)        (mm)
     INFLOW : ID= 2 (0898)    702.503      9.021       1.08        5.91
     OUTFLOW: ID= 1 (2614)    702.503      0.254       4.08        4.55

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  2.81
                  TIME SHIFT OF PEAK FLOW         (min)=180.00
                  MAXIMUM  STORAGE   USED        (ha.m.)=  3.4855

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (1614) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |       OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------        (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |    4.8340     4.8070
                           0.7000     0.6320    |   10.5600     5.7180
                           1.5360     1.2640    |   19.5800     6.8560
                           2.0480     1.5790    |   24.8600     7.4250
                           2.5600     1.9520    |   29.1700     7.9930
                           3.4560     2.6960    |   32.5000    10.1310
                           4.0940     3.4400    |    0.0000     0.0000

                               AREA      QPEAK      TPEAK        R.V.
                               (ha)      (cms)      (hrs)        (mm)
     INFLOW : ID= 2 (2614)    702.503      0.254       4.08        4.55
     OUTFLOW: ID= 1 (1614)    702.503      0.213       8.17        4.52

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)= 84.10
                  TIME SHIFT OF PEAK FLOW         (min)=245.00
                  MAXIMUM  STORAGE   USED        (ha.m.)=  0.1927

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0615)   |   Area    (ha)= 344.12
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  47.20   Dir. Conn.(%)=  47.20
--------------------
                              IMPERVIOUS    PERVIOUS (i)
```

```
   Surface Area     (ha)=     162.43       181.70
   Dep. Storage     (mm)=       0.10         5.00
   Average Slope     (%)=       0.79         0.79
   Length            (m)=    1514.64      1515.00
   Mannings n           =      0.013        0.250

   Max.Eff.Inten.(mm/hr)=      24.00         0.79
              over (min)       25.00       600.00
   Storage Coeff.  (min)=      24.78 (ii)  597.78 (ii)
   Unit Hyd. Tpeak (min)=      25.00       600.00
   Unit Hyd. peak  (cms)=       0.05         0.00
                                                          *TOTALS*
   PEAK FLOW       (cms)=       6.90         0.03           6.905 (iii)
   TIME TO PEAK    (hrs)=       0.92        10.58            0.92
   RUNOFF VOLUME    (mm)=      13.10         0.79            6.60
   TOTAL RAINFALL   (mm)=      13.20        13.20           13.20
   RUNOFF COEFFICIENT   =       0.99         0.06            0.50

    (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
         CN* =  76.7    Ia = Dep. Storage  (Above)
   (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
        THAN THE STORAGE COEFFICIENT.
  (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (1615) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    4.7400     3.1500
                          1.8400     0.1500   |    6.6000     3.8100
                          2.2800     0.3230   |    8.9000     5.1500
                          2.6400     0.7000   |   13.4000     5.8230
                          3.0000     2.0000   |   17.0000     6.1000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (0615)     344.121     6.905      0.92       6.60
   OUTFLOW: ID= 1 (1615)     344.121     2.699      1.42       6.60

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 39.08
               TIME SHIFT OF PEAK FLOW         (min)= 30.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  0.9145

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0899) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (1614):   702.50   0.213     8.17      4.52
      + ID2= 2 (1615):   344.12   2.699     1.42      6.60
        ===============================================
        ID = 3 (0899):  1046.62   2.706     1.50      5.20

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
 --------------------
 | ROUTE MC (0075)  |
 | IN=2---> OUT=1   |
 --------------------

 ROUTING TIME STEP = 5.0 min.

 <-----------------DATA FOR SECTION (1.1)----------------->
 Distance (m)     Elevation (m)     Mannings 'n'
   100.0              160.00            0.12
   104.0              158.27            0.12
   108.0              156.96            0.12
```

```
   109.0             156.55          0.12/0.05    Main Channel
   110.5             156.30          0.05         Main Channel
   112.0             156.44          0.05         Main Channel
   112.5             156.90          0.05/0.12    Main Channel
   120.0             157.13          0.12
   130.0             158.90          0.12
   150.0             160.00          0.12
                                AREA      QPEAK   TPEAK      R.V.    QBASE
                                (ha)      (cms)   (hrs)      (mm)    (cms)
  INFLOW:  ID=2  (0899)      1046.62      2.71     1.50      5.20    0.0
  OUTFLOW: ID=1  (0075)      1046.62      1.60     8.42      5.20    0.0

 ------------------------------------------------------------------------------

--------------------
| ADD HYD   (0900) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (0058):    59.90   0.054     1.17      0.31
      + ID2= 2 (0075):  1046.62   1.600     8.42      5.20
        ===============================================
        ID = 3 (0900):  1106.52   1.600     8.42      4.90

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0900) |
|  3 +  2 =  1     |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 3 (0900):  1106.52   1.600     8.42      4.90
      + ID2= 2 (0613):    40.00   0.978     0.75      5.90
        ===============================================
        ID = 1 (0900):  1146.52   1.603     8.42      4.93

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0073) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    1.5000     8.0000
                          0.2000     0.5000   |    3.0000     9.0000
                          0.6000     1.5000   |    5.0000    10.0000
                          1.0000     2.0000   |   10.0000    10.0050
                          1.2000     5.0000   |    0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (0900)    1146.524     1.603      8.42       4.93
   OUTFLOW: ID= 1 (0073)    1146.524     0.692     10.00       4.88

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 43.20
               TIME SHIFT OF PEAK FLOW         (min)= 95.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  1.6159

   **** WARNING : HYDROGRAPH WAS CUT. CHECK VOLUME.
------------------------------------------------------------------------------
--------------------
| DIVERT HYD (0001)|
| IN= 1   # OUT= 5 |
--------------------
   Outflow / Inflow  Relationships

    Flow 1 + Flow 2 + Flow 3 + Flow 4 + Flow 5 = Total
    (cms)    (cms)    (cms)    (cms)    (cms)    (cms)
     0.00     0.00     0.00     0.00     0.00     0.00
     0.70     0.70     0.00     0.00     0.00     1.40
```

```
      2.00     1.00     0.00     0.00     0.00     3.00
      3.10     2.60     0.00     0.00     0.00     5.70
      9.60     3.60     0.00     0.00     0.00    13.20
     14.30     4.60     0.00     0.00     0.00    18.90
     20.10     5.80     0.00     0.00     0.00    25.90
     25.40     6.50     0.00     0.00     0.00    31.90
     30.50     7.50     0.00     0.00     0.00    38.00
     95.80    19.20     0.00     0.00     0.00   115.00

                            AREA     QPEAK     TPEAK    R.V.
                            (ha)     (cms)     (hrs)    (mm)
   TOTAL HYD.(ID= 1):1146.52         0.69     10.00    4.88
   ======================================================
   ID= 2  (    2)    : 573.26        0.35     10.00    4.88
   ID= 3  (    2)    : 573.26        0.35     10.00    4.88
   ID= 4  (    2)    :   0.00        0.00      0.00    0.00
   ID= 5  (    2)    :   0.00        0.00      0.00    0.00
   ID= 6  (    2)    :   0.00        0.00      0.00    0.00
------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0894) |   Area    (ha)=  10.00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  35.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
   Surface Area     (ha)=       5.00         5.00
   Dep. Storage     (mm)=       1.00         1.50
   Average Slope     (%)=       1.00         2.00
   Length            (m)=     258.20        40.00
   Mannings n           =      0.013        0.250

   Max.Eff.Inten.(mm/hr)=      48.00         3.51
              over (min)        5.00        35.00
   Storage Coeff.  (min)=       6.05 (ii)   33.01 (ii)
   Unit Hyd. Tpeak (min)=       5.00        35.00
   Unit Hyd. peak  (cms)=       0.19         0.03
                                                          *TOTALS*
   PEAK FLOW       (cms)=       0.30         0.03           0.305 (iii)
   TIME TO PEAK    (hrs)=       0.58         1.08            0.58
   RUNOFF VOLUME    (mm)=      12.20         1.35            5.15
   TOTAL RAINFALL   (mm)=      13.20        13.20           13.20
   RUNOFF COEFFICIENT   =       0.92         0.10            0.39

    (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
         Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
         Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=  0.00
   (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
        THAN THE STORAGE COEFFICIENT.
  (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0999) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (0001):   573.26   0.346    10.00      4.88
      + ID2= 2 (0894):    10.00   0.305     0.58      5.15
        ===============================================
        ID = 3 (0999):   583.26   0.346    10.00      4.89

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0762) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
              <------ DATA FOR SECTION (  62.0) ------>
              Distance      Elevation       Manning
                 0.00         145.00        0.1800
```

```
             4.00          140.00         0.1800
            15.00          135.00         0.1800
            20.00          133.00         0.1800
            25.00          132.00         0.1800
            45.00          131.00         0.1800
            93.00          132.00         0.1800
           114.00          132.00         0.1800
           210.00          131.00         0.1800
           238.00          130.00         0.1800
           239.00          129.00     0.1800 /0.0700  Main Channel
           244.00          128.00         0.0700      Main Channel
           262.00          128.00         0.0700      Main Channel
           264.00          129.00     0.0700 /0.1950  Main Channel
           306.00          129.00         0.1950
           318.00          130.00         0.1950
           335.00          129.00         0.1950
           345.00          130.00         0.1950
           354.00          135.00         0.1950
           365.00          145.00         0.1950

   <-------------------- TRAVEL TIME TABLE --------------------->
    DEPTH    ELEV      VOLUME    FLOW RATE    VELOCITY   TRAV.TIME
     (m)      (m)     (cu.m.)      (cms)        (m/s)      (min)
     0.50   128.50   .788E+05        5.9         0.60      222.28
     1.00   129.00   .172E+06       19.5         0.91      146.86
     1.94   129.94   .816E+06       83.4         0.82      163.10
     2.88   130.88   .171E+07      204.0         0.95      139.80
     3.82   131.82   .319E+07      387.6         0.97      137.01
     4.76   132.76   .558E+07      698.4         1.00      133.20
     5.71   133.71   .807E+07     1127.5         1.11      119.29
     6.65   134.65   .106E+08     1653.2         1.25      106.75
     7.59   135.59   .131E+08     2268.5         1.38       96.52
     8.53   136.53   .157E+08     2966.8         1.51       88.25
     9.47   137.47   .183E+08     3742.6         1.63       81.51
    10.41   138.41   .209E+08     4592.4         1.75       75.93
    11.35   139.35   .236E+08     5513.5         1.87       71.23
    12.29   140.29   .262E+08     6506.2         1.98       67.19
    13.24   141.24   .289E+08     7572.0         2.09       63.63
    14.18   142.18   .316E+08     8702.5         2.20       60.53
    15.12   143.12   .343E+08     9895.6         2.30       57.79
    16.06   144.06   .370E+08    11150.0         2.40       55.36
    17.00   145.00   .398E+08    12463.9         2.50       53.18

                                   <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK    R.V.   MAX DEPTH   MAX VEL
                            (ha)    (cms)   (hrs)    (mm)      (m)       (m/s)
   INFLOW : ID= 2 (0999)  583.26     0.35   10.00    4.89      0.03       0.60
   OUTFLOW: ID= 1 (0762)  583.26     0.24   13.75    4.86      0.02       0.60

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0895) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (0762):   583.26   0.237    13.75      4.86
      + ID2= 2 (1601):   552.24   0.241     3.25      0.64
        ===============================================
        ID = 3 (0895):  1135.51   0.279     3.25      2.81

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0205) |   Area    (ha)=   0.00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  35.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
   Surface Area     (ha)=       0.00         0.00
   Dep. Storage     (mm)=       1.00         1.50
```

```
     Average Slope     (%)=       1.00          2.00
     Length            (m)=       0.00         40.00
     Mannings n           =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      48.00          3.51
                over (min)        5.00         30.00
     Storage Coeff.  (min)=       0.00 (ii)    26.95 (ii)
     Unit Hyd. Tpeak (min)=       5.00         30.00
     Unit Hyd. peak  (cms)=       0.34          0.08
                                                            *TOTALS*
     PEAK FLOW       (cms)=       0.00          0.00          0.000 (iii)
     TIME TO PEAK    (hrs)=       0.00          0.00          0.00
     RUNOFF VOLUME    (mm)=     NaN           NaN           NaN
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =     NaN           NaN           NaN

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
            Fo   (mm/hr)= 50.00          K  (1/hr)=  2.00
            Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=  0.00
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
-------------------
| CALIB             |
| STANDHYD  (0023) |   Area    (ha)=  19.20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
-------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=      10.06          9.14
     Dep. Storage      (mm)=       0.10          5.00
     Average Slope      (%)=       0.99          0.99
     Length             (m)=     357.77        358.00
     Mannings n            =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      38.40          0.42
                over (min)       10.00        300.00
     Storage Coeff.  (min)=       8.07 (ii)   299.17 (ii)
     Unit Hyd. Tpeak (min)=      10.00        300.00
     Unit Hyd. peak  (cms)=       0.13          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=       0.77          0.00          0.773 (iii)
     TIME TO PEAK    (hrs)=       0.67          5.58          0.67
     RUNOFF VOLUME    (mm)=      13.10          0.42          7.05
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =       0.99          0.03          0.53

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  62.3    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
-------------------
| RESERVOIR (0004) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
-------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0280     0.2320
                          0.0130     0.0290   |   0.0290     0.2610
                          0.0160     0.0580   |   0.0320     0.2900
                          0.0190     0.0870   |   0.0340     0.3480
                          0.0210     0.1160   |   0.0360     0.4610
                          0.0230     0.1450   |   0.3056     0.5278
                          0.0250     0.1740   |   0.7985     0.5947
                          0.0270     0.2030   |   1.4368     0.6615
```

```
                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0023)    19.200     0.773      0.67        7.05
     OUTFLOW: ID= 1 (0004)    19.200     0.022      1.33        7.02

                 PEAK   FLOW   REDUCTION [Qout/Qin](%)=  2.79
                 TIME SHIFT OF PEAK FLOW         (min)= 40.00
                 MAXIMUM  STORAGE   USED       (ha.m.)=  0.1246

--------------------------------------------------------------------------------
-------------------
| CALIB             |
| STANDHYD  (0021) |   Area    (ha)=  55.00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  41.60   Dir. Conn.(%)=  41.60
-------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=      22.88         32.12
     Dep. Storage      (mm)=       0.10          5.00
     Average Slope      (%)=       0.42          0.42
     Length             (m)=     605.53        605.00
     Mannings n            =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      30.40          0.79
                over (min)       15.00        415.00
     Storage Coeff.  (min)=      15.72 (ii)   414.99 (ii)
     Unit Hyd. Tpeak (min)=      15.00        415.00
     Unit Hyd. peak  (cms)=       0.07          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=       1.29          0.01          1.286 (iii)
     TIME TO PEAK    (hrs)=       0.75          7.50          0.75
     RUNOFF VOLUME    (mm)=      13.10          0.79          5.91
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =       0.99          0.06          0.45

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
-------------------
| RESERVOIR (0018) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
-------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0728     0.9731
                          0.0130     0.1216   |   0.0778     1.0947
                          0.0275     0.2433   |   0.0825     1.2164
                          0.0389     0.3649   |   0.0870     1.3380
                          0.0476     0.4865   |   1.2731     1.4845
                          0.0550     0.6082   |   3.4420     1.6310
                          0.0615     0.7298   |   6.2505     1.7775
                          0.0674     0.8515   |   0.0000     0.0000

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0021)    55.000     1.286      0.75        5.91
     OUTFLOW: ID= 1 (0018)    55.000     0.031      1.92        5.86

                 PEAK   FLOW   REDUCTION [Qout/Qin](%)=  2.44
                 TIME SHIFT OF PEAK FLOW         (min)= 70.00
                 MAXIMUM  STORAGE   USED       (ha.m.)=  0.2852

--------------------------------------------------------------------------------
-------------------
| CALIB             |
| STANDHYD  (0022) |   Area    (ha)=  45.60
|ID= 1 DT= 5.0 min |   Total Imp(%)=  63.10   Dir. Conn.(%)=  63.10
-------------------
                             IMPERVIOUS    PERVIOUS (i)
```

```
     Surface Area      (ha)=      28.77         16.83
     Dep. Storage      (mm)=       0.10          5.00
     Average Slope      (%)=       0.97          0.97
     Length             (m)=     551.36        551.00
     Mannings n            =       0.013         0.250

     Max.Eff.Inten.(mm/hr)=      38.40          0.79
                over (min)       10.00        305.00
     Storage Coeff.  (min)=      10.53 (ii)   304.19 (ii)
     Unit Hyd. Tpeak (min)=      10.00        305.00
     Unit Hyd. peak  (cms)=       0.11          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=       1.98          0.00          1.980 (iii)
     TIME TO PEAK    (hrs)=       0.67          5.67          0.67
     RUNOFF VOLUME    (mm)=      13.10          0.79          8.55
     TOTAL RAINFALL   (mm)=      13.20         13.20         13.20
     RUNOFF COEFFICIENT   =       0.99          0.06          0.65

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
-------------------
| RESERVOIR (0019) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
-------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0704     0.8335
                          0.0056     0.0926   |   0.0747     0.9262
                          0.0254     0.1852   |   0.0787     1.0188
                          0.0355     0.2778   |   0.0825     1.1114
                          0.0433     0.3705   |   0.0844     1.2040
                          0.0500     0.4631   |   1.4323     1.3120
                          0.0558     0.5557   |   3.8969     1.4200
                          0.0611     0.6483   |   7.0884     1.5280
                          0.0659     0.7409   |   0.0000     0.0000

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0022)    45.600     1.980      0.67        8.55
     OUTFLOW: ID= 1 (0019)    45.600     0.043      1.50        8.43

                 PEAK   FLOW   REDUCTION [Qout/Qin](%)=  2.15
                 TIME SHIFT OF PEAK FLOW         (min)= 50.00
                 MAXIMUM  STORAGE   USED       (ha.m.)=  0.3622

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0015) |
|   1 +  2 =  3    |         AREA     QPEAK     TPEAK      R.V.
-------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0018):     55.00    0.031      1.92      5.86
        + ID2= 2 (0019):     45.60    0.043      1.50      8.43
          ===============================================
          ID = 3 (0015):    100.60    0.074      1.75      7.03

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
-------------------
| CALIB             |
| STANDHYD  (0612) |   Area    (ha)=  97.00
|ID= 1 DT=15.0 min |   Total Imp(%)=  63.10   Dir. Conn.(%)=  63.10
-------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=      61.21         35.79
     Dep. Storage      (mm)=       0.10          5.00
```

```
     Average Slope     (%)=       0.97          0.97
     Length            (m)=     804.16       1143.00
     Mannings n           =       0.013         0.250

         NOTE:  RAINFALL WAS TRANSFORMED TO  15.0 MIN. TIME STEP.

                              ---- TRANSFORMED HYETOGRAPH ----
              TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
               hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
             0.250    8.00 | 0.500   10.40 | 0.750   30.40 |  1.00    4.80

     Max.Eff.Inten.(mm/hr)=      30.40          0.82
                over (min)       15.00        465.00
     Storage Coeff.  (min)=      14.50 (ii)   461.20 (ii)
     Unit Hyd. Tpeak (min)=      15.00        465.00
     Unit Hyd. peak  (cms)=       0.07          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=       3.84          0.01          3.842 (iii)
     TIME TO PEAK    (hrs)=       0.75          8.25          0.75
     RUNOFF VOLUME    (mm)=      13.30          0.82          8.69
     TOTAL RAINFALL   (mm)=      13.40         13.40         13.40
     RUNOFF COEFFICIENT   =       0.99          0.06          0.65

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
-------------------
| RESERVOIR (0050) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
-------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   4.8880     1.6100
                          0.1710     1.2880   |  15.0000    18.5000

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0612)    97.000     3.842      0.75        8.69
     OUTFLOW: ID= 1 (0050)    97.000     0.102      1.75        8.67

                 PEAK   FLOW   REDUCTION [Qout/Qin](%)=  2.66
                 TIME SHIFT OF PEAK FLOW         (min)= 60.00
                 MAXIMUM  STORAGE   USED       (ha.m.)=  0.7700

--------------------------------------------------------------------------------
-------------------
| RESERVOIR (0056) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
-------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   5.6570     2.7400
                          2.5000     2.1920   |  15.0000     2.9500

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0050)    97.000     0.102      1.75        8.67
     OUTFLOW: ID= 1 (0056)    97.000     0.082      6.83        8.67

                 PEAK   FLOW   REDUCTION [Qout/Qin](%)= 79.96
                 TIME SHIFT OF PEAK FLOW         (min)=305.00
                 MAXIMUM  STORAGE   USED       (ha.m.)=  0.0716

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0051) |
```

```
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK     R.V.
--------------------          (ha)     (cms)     (hrs)     (mm)
         ID1= 1 (0001):     573.26    0.346     10.00      4.88
       + ID2= 2 (0056):      97.00    0.082      6.83      8.67
         ==================================================
         ID = 3 (0051):     670.26    0.423      9.92      5.43

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0767) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  67.0) ------>
                 Distance      Elevation        Manning
                   99.00         152.00          0.1200
                  100.00         150.00          0.1200
                  125.00         149.12          0.1200
                  130.00         148.99      0.1200 /0.0500  Main Channel
                  131.00         148.23          0.0500      Main Channel
                  131.75         148.12          0.0500      Main Channel
                  132.50         148.19          0.0500      Main Channel
                  133.00         148.66      0.0500 /0.1200  Main Channel
                  138.00         149.10          0.1200
                  149.00         150.00          0.1200
                  150.00         152.00          0.1200

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
         (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
        0.18    148.30     .651E+02        0.1          0.28       17.58
        0.36    148.49     .169E+03        0.3          0.45       11.12
        0.54    148.66     .297E+03        0.6          0.57        8.82
        0.75    148.87     .542E+03        1.1          0.63        7.98
        0.96    149.08     .100E+04        2.0          0.61        8.19
        1.17    149.29     .199E+04        3.6          0.54        9.32
        1.37    149.50     .351E+04        5.9          0.50        9.91
        1.58    149.71     .557E+04        9.3          0.50        9.99
        1.79    149.92     .815E+04       13.9          0.51        9.80
        2.00    150.12     .112E+05       20.4          0.55        9.14
        2.21    150.33     .142E+05       28.6          0.60        8.30
        2.42    150.54     .173E+05       38.0          0.66        7.60
        2.62    150.75     .204E+05       48.5          0.71        7.02
        2.83    150.96     .236E+05       60.1          0.76        6.54
        3.04    151.17     .267E+05       72.6          0.82        6.13
        3.25    151.37     .298E+05       86.0          0.86        5.78
        3.46    151.58     .330E+05      100.3          0.91        5.48
        3.67    151.79     .362E+05      115.5          0.96        5.22
        3.87    152.00     .393E+05      131.5          1.00        4.98

                                      <---- hydrograph ---->   <-pipe / channel->
                              AREA     QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                              (ha)     (cms)   (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0051)  670.26      0.42    9.92   5.43      0.46       0.51
     OUTFLOW: ID= 1 (0767)  670.26      0.42   10.08   5.43      0.46       0.51

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0059) |   Area    (ha)=  12.10   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

        NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                              ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
               0.083    0.00 | 0.333    2.40 | 0.583   48.00 |  0.83    9.60
```

```
              0.167   14.40 | 0.417   24.00 | 0.667   28.80 |  0.92    2.40
              0.250    9.60 | 0.500    4.80 | 0.750   14.40 |

     Unit Hyd Qpeak  (cms)=   2.311

     PEAK FLOW       (cms)=   0.023 (i)
     TIME TO PEAK    (hrs)=   0.833
     RUNOFF VOLUME    (mm)=   0.311
     TOTAL RAINFALL   (mm)=  13.200
     RUNOFF COEFFICIENT   =   0.024

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0901) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK     R.V.
--------------------          (ha)     (cms)     (hrs)     (mm)
         ID1= 1 (0059):      12.10    0.023      0.83      0.31
       + ID2= 2 (0767):     670.26    0.422     10.08      5.43
         ==================================================
         ID = 3 (0901):     682.36    0.422     10.08      5.34

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| NASHYD    (0060) |   Area    (ha)=  18.10   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     Unit Hyd Qpeak  (cms)=   3.457

     PEAK FLOW       (cms)=   0.034 (i)
     TIME TO PEAK    (hrs)=   0.833
     RUNOFF VOLUME    (mm)=   0.311
     TOTAL RAINFALL   (mm)=  13.200
     RUNOFF COEFFICIENT   =   0.024

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0002) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK     R.V.
--------------------          (ha)     (cms)     (hrs)     (mm)
         ID1= 1 (0015):     100.60    0.074      1.75      7.03
       + ID2= 2 (0060):      18.10    0.034      0.83      0.31
         ==================================================
         ID = 3 (0002):     118.70    0.090      0.92      6.00

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0002) |
|   3 +  2 =  1    |          AREA     QPEAK     TPEAK     R.V.
--------------------          (ha)     (cms)     (hrs)     (mm)
         ID1= 3 (0002):     118.70    0.090      0.92      6.00
       + ID2= 2 (0901):     682.36    0.422     10.08      5.34
         ==================================================
         ID = 1 (0002):     801.06    0.480     10.08      5.44

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0010) |
```

```
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (   1.1) ------>
                 Distance      Elevation        Manning
                    0.00         152.00          0.1200
                    1.00         150.00          0.1200
                   26.00         149.12          0.1200
                   31.00         148.99      0.1200 /0.0500  Main Channel
                   32.00         148.23          0.0500      Main Channel
                   32.75         148.12          0.0500      Main Channel
                   33.50         148.19          0.0500      Main Channel
                   34.00         148.66      0.0500 /0.1200  Main Channel
                   39.00         149.10          0.1200
                   50.00         150.00          0.1200
                   51.00         152.00          0.1200

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
         (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
        0.18    148.30     .108E+03        0.1          0.28       29.30
        0.36    148.49     .282E+03        0.3          0.45       18.54
        0.54    148.66     .495E+03        0.6          0.57       14.70
        0.75    148.87     .903E+03        1.1          0.63       13.29
        0.96    149.08     .167E+04        2.0          0.61       13.64
        1.17    149.29     .332E+04        3.6          0.54       15.53
        1.37    149.50     .585E+04        5.9          0.50       16.51
        1.58    149.71     .928E+04        9.3          0.50       16.65
        1.79    149.92     .136E+05       13.9          0.51       16.33
        2.00    150.12     .186E+05       20.4          0.55       15.23
        2.21    150.33     .237E+05       28.6          0.60       13.84
        2.42    150.54     .289E+05       38.0          0.66       12.67
        2.62    150.75     .341E+05       48.5          0.71       11.70
        2.83    150.96     .393E+05       60.1          0.76       10.90
        3.04    151.17     .445E+05       72.6          0.82       10.22
        3.25    151.37     .497E+05       86.0          0.86        9.64
        3.46    151.58     .550E+05      100.3          0.91        9.13
        3.67    151.79     .603E+05      115.5          0.96        8.69
        3.87    152.00     .656E+05      131.5          1.00        8.31

                                      <---- hydrograph ---->   <-pipe / channel->
                              AREA     QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                              (ha)     (cms)   (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0002)  801.06      0.48   10.08   5.44      0.49       0.53
     OUTFLOW: ID= 1 (0010)  801.06      0.48   10.25   5.43      0.49       0.53

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0009) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK     R.V.
--------------------          (ha)     (cms)     (hrs)     (mm)
         ID1= 1 (0004):      19.20    0.022      1.33      7.02
       + ID2= 2 (0010):     801.06    0.476     10.25      5.43
         ==================================================
         ID = 3 (0009):     820.26    0.493     10.25      5.47

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0006) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (   1.1) ------>
                 Distance      Elevation        Manning
                    0.00         152.00          0.1200
                    1.00         150.00          0.1200
                   26.00         149.12          0.1200
                   31.00         148.99      0.1200 /0.0500  Main Channel
                   32.00         148.23          0.0500      Main Channel
                   32.75         148.12          0.0500      Main Channel
```

```
                   33.50         148.19          0.0500      Main Channel
                   34.00         148.66      0.0500 /0.1200  Main Channel
                   39.00         149.10          0.1200
                   50.00         150.00          0.1200
                   51.00         152.00          0.1200

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
         (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
        0.18    148.30     .130E+03        0.1          0.28       35.16
        0.36    148.49     .339E+03        0.3          0.45       22.24
        0.54    148.66     .594E+03        0.6          0.57       17.64
        0.75    148.87     .108E+04        1.1          0.63       15.95
        0.96    149.08     .200E+04        2.0          0.61       16.37
        1.17    149.29     .398E+04        3.6          0.54       18.63
        1.37    149.50     .702E+04        5.9          0.50       19.81
        1.58    149.71     .111E+05        9.3          0.50       19.99
        1.79    149.92     .163E+05       13.9          0.51       19.60
        2.00    150.12     .223E+05       20.4          0.55       18.27
        2.21    150.33     .285E+05       28.6          0.60       16.60
        2.42    150.54     .347E+05       38.0          0.66       15.21
        2.62    150.75     .409E+05       48.5          0.71       14.05
        2.83    150.96     .471E+05       60.1          0.76       13.08
        3.04    151.17     .534E+05       72.6          0.82       12.26
        3.25    151.37     .597E+05       86.0          0.86       11.56
        3.46    151.58     .660E+05      100.3          0.91       10.96
        3.67    151.79     .723E+05      115.5          0.96       10.43
        3.87    152.00     .787E+05      131.5          1.00        9.97

                                      <---- hydrograph ---->   <-pipe / channel->
                              AREA     QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                              (ha)     (cms)   (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0009)  820.26      0.49   10.25   5.47      0.50       0.54
     OUTFLOW: ID= 1 (0006)  820.26      0.49   10.42   5.47      0.50       0.53

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0020) |   Area    (ha)=  97.70
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     51.19        46.51
     Dep. Storage     (mm)=      0.10         5.00
     Average Slope     (%)=      0.99         0.99
     Length            (m)=    807.05       807.00
     Mannings n           =     0.013        0.250

     Max.Eff.Inten.(mm/hr)=     30.40         0.42
               over (min)      15.00       490.00
     Storage Coeff.  (min)=     14.44 (ii)  488.49 (ii)
     Unit Hyd. Tpeak (min)=     15.00       490.00
     Unit Hyd. peak  (cms)=      0.08         0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=      2.99         0.00         2.994 (iii)
     TIME TO PEAK    (hrs)=      0.75         8.75          0.75
     RUNOFF VOLUME    (mm)=     13.10         0.42          7.06
     TOTAL RAINFALL   (mm)=     13.20        13.20         13.20
     RUNOFF COEFFICIENT   =      0.99         0.03          0.53

      (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  62.3    Ia = Dep. Storage  (Above)
     (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0016) |
| IN= 2---> OUT= 1 |
```

```
| DT=  5.0 min     |      OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000    |  0.1430     1.2448
                          0.0380     0.0189    |  0.1520     1.4335
                          0.0530     0.1132    |  0.1560     1.5278
                          0.0640     0.2075    |  0.1650     1.7164
                          0.0830     0.3961    |  0.1770     2.0936
                          0.1040     0.5847    |  0.1800     2.2985
                          0.1100     0.6790    |  2.6063     2.4710
                          0.1220     0.8676    |  7.0425     2.6435
                          0.1280     0.9619    | 12.7872     2.8160
                          0.1380     1.1505    | 19.5901     2.9885

                               AREA      QPEAK      TPEAK        R.V.
                               (ha)      (cms)      (hrs)        (mm)
     INFLOW : ID= 2 (0020)    97.700      2.994       0.75        7.06
     OUTFLOW: ID= 1 (0016)    97.700      0.106       1.75        7.06

               PEAK   FLOW   REDUCTION [Qout/Qin](%)=  3.55
               TIME SHIFT OF PEAK FLOW         (min)= 60.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  0.6191

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0014) |
|   1 +  2 =  3    |           AREA    QPEAK    TPEAK     R.V.
--------------------           (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0006):     820.26    0.489    10.42     5.47
        + ID2= 2 (0016):      97.70    0.106     1.75     7.06
          ===============================================
          ID = 3 (0014):     917.96    0.567    10.42     5.64

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| CALIB            |
| STANDHYD  (9204) |   Area    (ha)=  18.90
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  50.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      9.45          9.45
     Dep. Storage     (mm)=      0.10          5.00
     Average Slope     (%)=      0.50          0.97
     Length            (m)=    354.96       1500.00
     Mannings n          =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=     38.40          0.79
                over (min)      10.00        550.00
     Storage Coeff.  (min)=      9.86 (ii)   545.43 (ii)
     Unit Hyd. Tpeak (min)=     10.00        550.00
     Unit Hyd. peak  (cms)=      0.11          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=      0.67          0.00           0.669 (iii)
     TIME TO PEAK    (hrs)=      0.67          9.75           0.67
     RUNOFF VOLUME    (mm)=     13.10          0.79           6.93
     TOTAL RAINFALL   (mm)=     13.20         13.20          13.20
     RUNOFF COEFFICIENT   =      0.99          0.06           0.52

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

--------------------
| CALIB            |
| STANDHYD  (4243) |   Area    (ha)=   6.10
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
```

```
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      3.20          2.90
     Dep. Storage     (mm)=      0.10          5.00
     Average Slope     (%)=      0.50          0.97
     Length            (m)=    201.66       1500.00
     Mannings n          =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=     48.00          0.79
                over (min)       5.00        545.00
     Storage Coeff.  (min)=      6.43 (ii)   541.99 (ii)
     Unit Hyd. Tpeak (min)=      5.00        545.00
     Unit Hyd. peak  (cms)=      0.18          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=      0.27          0.00           0.270 (iii)
     TIME TO PEAK    (hrs)=      0.58          9.67           0.58
     RUNOFF VOLUME    (mm)=     13.10          0.79           7.19
     TOTAL RAINFALL   (mm)=     13.20         13.20          13.20
     RUNOFF COEFFICIENT   =      0.99          0.06           0.54

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0072) |
|   1 +  2 =  3    |           AREA    QPEAK    TPEAK     R.V.
--------------------           (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (4243):       6.10    0.270     0.58     7.19
        + ID2= 2 (9204):      18.90    0.669     0.67     6.93
          ===============================================
          ID = 3 (0072):      25.00    0.931     0.67     6.99

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0071) |
|   1 +  2 =  3    |           AREA    QPEAK    TPEAK     R.V.
--------------------           (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0014):     917.96    0.567    10.42     5.64
        + ID2= 2 (0072):      25.00    0.931     0.67     6.99
          ===============================================
          ID = 3 (0071):     942.96    0.998     0.67     5.67

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0064) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance      Elevation       Manning
                  0.00         152.00         0.1200
                  1.00         150.00         0.1200
                 26.00         149.12         0.1200
                 31.00         148.99     0.1200 /0.0500  Main Channel
                 32.00         148.23         0.0500      Main Channel
                 32.75         148.12         0.0500      Main Channel
                 33.50         148.19         0.0500      Main Channel
                 34.00         148.66     0.0500 /0.1200  Main Channel
                 39.00         149.10         0.1200
                 50.00         150.00         0.1200
                 51.00         152.00         0.1200
```

```
      <--------------------- TRAVEL TIME TABLE --------------------->
       DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
        (m)       (m)     (cu.m.)      (cms)         (m/s)       (min)
       0.18    148.30    .141E+03       0.1           0.28       38.09
       0.36    148.49    .367E+03       0.3           0.45       24.10
       0.54    148.66    .643E+03       0.6           0.57       19.11
       0.75    148.87    .117E+04       1.1           0.63       17.28
       0.96    149.08    .217E+04       2.0           0.61       17.73
       1.17    149.29    .431E+04       3.6           0.54       20.19
       1.37    149.50    .761E+04       5.9           0.50       21.47
       1.58    149.71    .121E+05       9.3           0.50       21.65
       1.79    149.92    .177E+05      13.9           0.51       21.23
       2.00    150.12    .242E+05      20.4           0.55       19.79
       2.21    150.33    .309E+05      28.6           0.60       17.99
       2.42    150.54    .376E+05      38.0           0.66       16.47
       2.62    150.75    .443E+05      48.5           0.71       15.22
       2.83    150.96    .511E+05      60.1           0.76       14.17
       3.04    151.17    .578E+05      72.6           0.82       13.28
       3.25    151.37    .646E+05      86.0           0.86       12.53
       3.46    151.58    .715E+05     100.3           0.91       11.87
       3.67    151.79    .784E+05     115.5           0.96       11.30
       3.87    152.00    .852E+05     131.5           1.00       10.80

                                    <---- hydrograph ---->    <-pipe / channel->
                             AREA     QPEAK   TPEAK   R.V.    MAX DEPTH  MAX VEL
                             (ha)     (cms)   (hrs)   (mm)       (m)      (m/s)
     INFLOW : ID= 2 (0071)  942.96     1.00    0.67   5.67      0.70      0.61
     OUTFLOW: ID= 1 (0064)  942.96     0.65    0.83   5.67      0.57      0.58

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (4223) |   Area    (ha)=  47.80
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  50.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     23.90         23.90
     Dep. Storage     (mm)=      0.10          5.00
     Average Slope     (%)=      0.50          0.97
     Length            (m)=    564.51       1143.00
     Mannings n          =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=     30.40          0.79
                over (min)      15.00        470.00
     Storage Coeff.  (min)=     14.31 (ii)   469.28 (ii)
     Unit Hyd. Tpeak (min)=     15.00        470.00
     Unit Hyd. peak  (cms)=      0.08          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=      1.40          0.00           1.404 (iii)
     TIME TO PEAK    (hrs)=      0.75          8.42           0.75
     RUNOFF VOLUME    (mm)=     13.10          0.79           6.94
     TOTAL RAINFALL   (mm)=     13.20         13.20          13.20
     RUNOFF COEFFICIENT   =      0.99          0.06           0.53

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0204) |   Area    (ha)= 142.20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  40.00   Dir. Conn.(%)=  40.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     56.88         85.32
     Dep. Storage     (mm)=      0.10          5.00
     Average Slope     (%)=      0.50          0.97
```

```
     Length            (m)=    973.65       1143.00
     Mannings n          =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=     26.40          0.79
                over (min)      20.00        480.00
     Storage Coeff.  (min)=     20.99 (ii)   475.97 (ii)
     Unit Hyd. Tpeak (min)=     20.00        480.00
     Unit Hyd. peak  (cms)=      0.05          0.00
                                                            *TOTALS*
     PEAK FLOW       (cms)=      2.71          0.02           2.714 (iii)
     TIME TO PEAK    (hrs)=      0.83          8.58           0.83
     RUNOFF VOLUME    (mm)=     13.10          0.79           5.71
     TOTAL RAINFALL   (mm)=     13.20         13.20          13.20
     RUNOFF COEFFICIENT   =      0.99          0.06           0.43

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0035) |
|   1 +  2 =  3    |           AREA    QPEAK    TPEAK     R.V.
--------------------           (ha)    (cms)    (hrs)     (mm)
          ID1= 1 (0064):     942.96    0.654     0.83     5.67
        + ID2= 2 (0204):     142.20    2.714     0.83     5.71
          ===============================================
          ID = 3 (0035):    1085.16    3.367     0.83     5.68

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0035) |
|   3 +  2 =  1    |           AREA    QPEAK    TPEAK     R.V.
--------------------           (ha)    (cms)    (hrs)     (mm)
          ID1= 3 (0035):    1085.16    3.367     0.83     5.68
        + ID2= 2 (4223):      47.80    1.404     0.75     6.94
          ===============================================
          ID = 1 (0035):    1132.96    4.730     0.83     5.73

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0069) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance      Elevation       Manning
                  0.00         152.00         0.1200
                  1.00         150.00         0.1200
                 26.00         149.12         0.1200
                 31.00         148.99     0.1200 /0.0500  Main Channel
                 32.00         148.23         0.0500      Main Channel
                 32.75         148.12         0.0500      Main Channel
                 33.50         148.19         0.0500      Main Channel
                 34.00         148.66     0.0500 /0.1200  Main Channel
                 39.00         149.10         0.1200
                 50.00         150.00         0.1200
                 51.00         152.00         0.1200

      <--------------------- TRAVEL TIME TABLE --------------------->
       DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
        (m)       (m)     (cu.m.)      (cms)         (m/s)       (min)
       0.18    148.30    .217E+03       0.1           0.28       58.60
       0.36    148.49    .565E+03       0.3           0.45       37.07
```

```
    0.54   148.66    .989E+03      0.6          0.57      29.39
    0.75   148.87    .181E+04      1.1          0.63      26.59
    0.96   149.08    .334E+04      2.0          0.61      27.28
    1.17   149.29    .663E+04      3.6          0.54      31.05
    1.37   149.50    .117E+05      5.9          0.50      33.02
    1.58   149.71    .186E+05      9.3          0.50      33.31
    1.79   149.92    .272E+05     13.9          0.51      32.67
    2.00   150.12    .372E+05     20.4          0.55      30.45
    2.21   150.33    .475E+05     28.6          0.60      27.67
    2.42   150.54    .578E+05     38.0          0.66      25.34
    2.62   150.75    .682E+05     48.5          0.71      23.41
    2.83   150.96    .785E+05     60.1          0.76      21.80
    3.04   151.17    .890E+05     72.6          0.82      20.43
    3.25   151.37    .995E+05     86.0          0.86      19.27
    3.46   151.58    .110E+06    100.3          0.91      18.27
    3.67   151.79    .121E+06    115.5          0.96      17.39
    3.87   152.00    .131E+06    131.5          1.00      16.62

                               <---- hydrograph ---->  <-pipe / channel->
                        AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                        (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
   INFLOW : ID= 2 (0035) 1132.96   4.73    0.83   5.73     1.27      0.52
   OUTFLOW: ID= 1 (0069) 1132.96   2.77    1.17   5.73     1.05      0.57

-------------------------------------------------------------------------------
--------------------
| RESERVOIR (0053) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |    OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------     (cms)     (ha.m.)   |    (cms)     (ha.m.)
                         0.0000     0.0000   |    1.1000     1.6000
                         0.3000     0.0050   |    1.1800     2.3000
                         0.6000     0.0250   |    1.2270     2.7000
                         0.7500     0.2000   |    5.2800     3.0500
                         0.8500     0.5000   |   27.4400     3.9440
                         0.9500     1.0000   |   59.0000     4.8000

                           AREA      QPEAK      TPEAK       R.V.
                           (ha)      (cms)      (hrs)       (mm)
   INFLOW : ID= 2 (0069)   1132.962     2.773       1.17        5.73
   OUTFLOW: ID= 1 (0053)   1132.962     0.878       2.17        5.73

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 31.66
              TIME SHIFT OF PEAK FLOW          (min)= 60.00
              MAXIMUM  STORAGE   USED        (ha.m.)=  0.6399

-------------------------------------------------------------------------------
--------------------
| ADD HYD   (0067) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
*** W A R N I N G :  HYDROGRAPH 0205 <ID= 2> IS DRY.
*** W A R N I N G :  HYDROGRAPH 0003 = HYDROGRAPH 0001
*** W A R N I N G :  HYDROGRAPH 0003 = HYDROGRAPH 0001
          ID1= 1 (0053):  1132.96   0.878      2.17      5.73
        + ID2= 2 (0205):     0.00   0.000      0.00    NaN
          ===============================================
          ID = 3 (0067):  1132.96   0.878      2.17      5.73

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0025) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
             <------ DATA FOR SECTION (   1.1) ------>
             Distance       Elevation       Manning
                 0.00          137.50        0.1200
                 1.00          135.50        0.1200
```

```
                26.00          134.52        0.1200
                29.80          134.54        0.1200
                30.00          134.09        0.1200
                30.50          134.03        0.1200
                31.00          134.01      0.1200 /0.0500  Main Channel
                31.50          134.01        0.0500        Main Channel
                32.00          133.96        0.0500        Main Channel
                32.50          133.95        0.0500        Main Channel
                33.00          133.99        0.0500        Main Channel
                33.50          134.06        0.0500        Main Channel
                34.00          134.07      0.0500 /0.1200  Main Channel
                35.00          134.06        0.1200
                35.50          134.07        0.1200
                36.00          134.30        0.1200
                36.50          134.41        0.1200
                37.00          134.44        0.1200
                50.00          135.50        0.1200
                51.00          137.50        0.1200

   <-------------------- TRAVEL TIME TABLE --------------------->
    DEPTH    ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
     (m)      (m)     (cu.m.)      (cms)         (m/s)       (min)
    0.06   134.01    .475E+02      0.0          0.19      65.59
    0.24   134.20    .717E+03      0.5          0.49      25.30
    0.43   134.38    .156E+04      1.5          0.73      17.21
    0.61   134.56    .270E+04      2.9          0.80      15.70
    0.79   134.75    .503E+04      5.2          0.78      15.99
    0.98   134.93    .833E+04      8.8          0.79      15.83
    1.16   135.11    .126E+05     13.6          0.81      15.38
    1.34   135.30    .178E+05     20.0          0.84      14.80
    1.53   135.48    .240E+05     28.2          0.88      14.18
    1.71   135.66    .307E+05     39.5          0.97      12.95
    1.90   135.85    .375E+05     52.8          1.06      11.84
    2.08   136.03    .443E+05     67.6          1.14      10.93
    2.26   136.22    .511E+05     83.8          1.23      10.16
    2.45   136.40    .580E+05    101.5          1.31       9.52
    2.63   136.58    .649E+05    120.5          1.39       8.97
    2.81   136.77    .718E+05    140.7          1.47       8.50
    3.00   136.95    .787E+05    162.2          1.55       8.09
    3.18   137.13    .857E+05    184.9          1.62       7.72
    3.36   137.32    .926E+05    208.7          1.69       7.40

                               <---- hydrograph ---->  <-pipe / channel->
                        AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                        (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
   INFLOW : ID= 2 (0067) 1132.96   0.88    2.17   5.73     0.32      0.56
   OUTFLOW: ID= 1 (0025) 1132.96   0.87    2.58   5.72     0.31      0.56

-------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0610) |   Area    (ha)= 431.20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
--------------------
                           IMPERVIOUS    PERVIOUS (i)
   Surface Area      (ha)=    225.95       205.25
   Dep. Storage      (mm)=      0.10         5.00
   Average Slope      (%)=      0.99         0.99
   Length             (m)=   1695.48      2298.00
   Mannings n            =     0.013        0.250

   Max.Eff.Inten.(mm/hr)=     24.00         0.42
             over (min)       25.00       915.00
   Storage Coeff.  (min)=     24.78 (ii)  912.98 (ii)
   Unit Hyd. Tpeak (min)=     25.00       915.00
   Unit Hyd. peak  (cms)=      0.05         0.00
                                                        *TOTALS*
   PEAK FLOW       (cms)=      9.60         0.01         9.605 (iii)
   TIME TO PEAK    (hrs)=      0.92        15.83          0.92
   RUNOFF VOLUME    (mm)=     13.10         0.42          7.06
   TOTAL RAINFALL   (mm)=     13.20        13.20         13.20
```

```
   RUNOFF COEFFICIENT   =      0.99         0.03          0.53

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
         CN*  =  62.3    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------------
--------------------
| ADD HYD   (0024) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0025):  1132.96   0.871      2.58      5.72
        + ID2= 2 (0610):   431.20   9.605      0.92      7.06
          ===============================================
          ID = 3 (0024):  1564.16   9.978      0.92      6.09

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0027) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
             <------ DATA FOR SECTION (   1.1) ------>
             Distance       Elevation       Manning
                 0.00          152.00        0.1200
                 1.00          150.00        0.1200
                26.00          149.12        0.1200
                31.00          148.99      0.1200 /0.0500  Main Channel
                32.00          148.23        0.0500        Main Channel
                32.75          148.12        0.0500        Main Channel
                33.50          148.19        0.0500        Main Channel
                34.00          148.66      0.0500 /0.1200  Main Channel
                39.00          149.10        0.1200
                50.00          150.00        0.1200
                51.00          152.00        0.1200

   <-------------------- TRAVEL TIME TABLE --------------------->
    DEPTH    ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
     (m)      (m)     (cu.m.)      (cms)         (m/s)       (min)
    0.18   148.30    .362E+03      0.1          0.28      97.68
    0.36   148.49    .942E+03      0.3          0.45      61.80
    0.54   148.66    .165E+04      0.6          0.57      49.00
    0.75   148.87    .301E+04      1.1          0.63      44.32
    0.96   149.08    .556E+04      2.0          0.61      45.48
    1.17   149.29    .111E+05      3.6          0.54      51.77
    1.37   149.50    .195E+05      5.9          0.50      55.05
    1.58   149.71    .309E+05      9.3          0.50      55.53
    1.79   149.92    .453E+05     13.9          0.51      54.45
    2.00   150.12    .621E+05     20.4          0.55      50.77
    2.21   150.33    .792E+05     28.6          0.60      46.13
    2.42   150.54    .964E+05     38.0          0.66      42.25
    2.62   150.75    .114E+06     48.5          0.71      39.02
    2.83   150.96    .131E+06     60.1          0.76      36.33
    3.04   151.17    .148E+06     72.6          0.82      34.06
    3.25   151.37    .166E+06     86.0          0.86      32.12
    3.46   151.58    .183E+06    100.3          0.91      30.45
    3.67   151.79    .201E+06    115.5          0.96      28.99
    3.87   152.00    .219E+06    131.5          1.00      27.70

                               <---- hydrograph ---->  <-pipe / channel->
                        AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                        (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
   INFLOW : ID= 2 (0024) 1564.16   9.98    0.92   6.09     1.61      0.50
   OUTFLOW: ID= 1 (0027) 1564.16   4.71    1.42   6.09     1.26      0.52

-------------------------------------------------------------------------------
```

```
--------------------
| ADD HYD   (0903) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0027):  1564.16   4.708      1.42      6.09
        + ID2= 2 (0895):  1135.51   0.279      3.25      2.81
          ===============================================
          ID = 3 (0903):  2699.67   4.811      1.42      4.71

   NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

-------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0768) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
             <------ DATA FOR SECTION (  68.0) ------>
             Distance       Elevation       Manning
                 0.00          105.00        0.2100
                 8.00          100.00        0.2100
                14.00           97.00        0.2100
                32.00           96.00        0.2100
                75.00           96.00        0.2100
               120.00           95.50        0.2100
               134.00           95.50        0.2100
               149.00           93.00        0.2100
               189.00           92.00        0.2100
               190.00           91.00        0.2100
               201.00           90.00      0.2100 /0.0700  Main Channel
               203.00           89.60        0.0700        Main Channel
               215.00           89.60        0.0700        Main Channel
               222.00           91.00      0.0700 /0.1950  Main Channel
               230.00           90.00        0.1950
               232.00           92.10        0.1950
               234.00           92.00        0.1950
               244.00           92.00        0.1950
               252.00           95.00        0.1950
               284.00          100.00        0.1950

   <-------------------- TRAVEL TIME TABLE --------------------->
    DEPTH    ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
     (m)      (m)     (cu.m.)      (cms)         (m/s)       (min)
    0.40    90.00    .390E+05      2.5          0.45     259.68
    0.93    90.53    .122E+06     11.7          0.67     174.20
    1.45    91.05    .253E+06     27.8          0.77     151.17
    1.98    91.58    .405E+06     53.6          0.92     125.83
    2.51    92.11    .571E+06     83.1          1.01     114.49
    3.03    92.63    .830E+06    122.5          1.03     112.94
    3.56    93.16    .117E+07    174.4          1.04     111.72
    4.08    93.68    .154E+07    239.1          1.08     107.41
    4.61    94.21    .193E+07    313.6          1.13     102.53
    5.14    94.74    .233E+07    397.6          1.19      97.84
    5.66    95.26    .276E+07    489.8          1.24      93.81
    6.19    95.79    .326E+07    560.9          1.20      96.73
    6.72    96.32    .402E+07    655.8          1.14     102.07
    7.24    96.84    .490E+07    803.3          1.14     101.64
    7.77    97.37    .582E+07    973.7          1.17      99.63
    8.29    97.89    .676E+07   1164.2          1.20      96.78
    8.82    98.42    .772E+07   1372.3          1.24      93.71
    9.35    98.95    .869E+07   1597.5          1.28      90.63
    9.87    99.47    .968E+07   1839.4          1.32      87.66

                               <---- hydrograph ---->  <-pipe / channel->
                        AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                        (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
   INFLOW : ID= 2 (0903) 2699.67   4.81    1.42   4.71     0.53      0.49
   OUTFLOW: ID= 1 (0768) 2699.67   1.58    3.75   4.69     0.25      0.45

-------------------------------------------------------------------------------
```

```
 --------------------
 | ADD HYD    (0904) |
 |  1 +  2 =  3      |          AREA     QPEAK     TPEAK      R.V.
 --------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0600):      822.24    5.404      1.00       3.11
        + ID2= 2 (0768):     2699.67    1.581      3.75       4.69
          ==================================================
          ID = 3 (0904):     3521.91    5.661      1.08       4.32

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ------------------------------------------------------------------------------
 --------------------
 | CALIB              |
 | WILHYD     (0500) |   Area     (ha)= 261.11   Curve Number   (CN)= 76.5
 |ID= 1 DT= 5.0 min  |   Ia       (mm)=   5.00   Recession const.(K)= 1.06
 --------------------    U.H. Tp(hrs)=    0.89

     U.H. peak        (cms)=   8.97

     PEAK FLOW        (cms)=   0.27 (i)
     TIME TO PEAK     (hrs)=   1.50
     RUNOFF VOLUME     (mm)=   0.77
     TOTAL RAINFALL    (mm)=  13.20
     RUNOFF COEFFICIENT    =   0.06

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
 ------------------------------------------------------------------------------

 --------------------
 | ADD HYD    (0956) |
 |  1 +  2 =  3      |          AREA     QPEAK     TPEAK      R.V.
 --------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0500):      261.11    0.272      1.50       0.77
        + ID2= 2 (0904):     3521.91    5.661      1.08       4.32
          ==================================================
          ID = 3 (0956):     3783.02    5.849      1.08       4.08

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ------------------------------------------------------------------------------
 --------------------
 | ROUTE CHN (7102) |
 | IN= 2---> OUT= 1 |      Routing time step (min)'=  5.00
 --------------------
                <------ DATA FOR SECTION ( 102.0) ------>
                Distance        Elevation          Manning
                   0.00           87.00            0.1800
                   2.00           86.00            0.1800
                   8.00           85.00            0.1800
                  19.00           80.00        0.1800 /0.0600   Main Channel
                  30.00           75.30            0.0600       Main Channel
                 150.00           75.30            0.0600       Main Channel
                 200.00           75.60            0.0600       Main Channel
                 286.00           75.30            0.0600       Main Channel
                 312.00           75.30            0.0600       Main Channel
                 318.00           76.00            0.0600       Main Channel
                 342.00           77.00        0.0600 /0.1800   Main Channel
                 350.00           80.00            0.1800
                 367.00           85.00            0.1800

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
         (m)       (m)     (cu.m.)       (cms)         (m/s)       (min)
        0.43     75.73    .259E+06        9.1          0.09       473.60
        0.85     76.15    .576E+06       34.0          0.15       282.46
        1.28     76.58    .904E+06       70.2          0.20       214.51
        1.70     77.00    .124E+07      116.7          0.24       177.62
        2.23     77.53    .168E+07      191.9          0.29       145.91
        2.77     78.07    .212E+07      281.4          0.34       125.55
        3.30     78.60    .256E+07      384.3          0.39       111.19
        3.83     79.13    .301E+07      499.6          0.43       100.44
        4.37     79.67    .346E+07      627.0          0.47        92.03
        4.90     80.20    .392E+07      766.5          0.50        85.16
        5.43     80.73    .438E+07      918.5          0.54        79.39
        5.97     81.27    .484E+07     1081.5          0.58        74.56
        6.50     81.80    .531E+07     1255.1          0.61        70.44
        7.03     82.33    .578E+07     1439.1          0.64        66.89
        7.57     82.87    .625E+07     1633.3          0.67        63.79
        8.10     83.40    .673E+07     1837.3          0.70        61.05
        8.63     83.93    .721E+07     2051.1          0.73        58.62
        9.17     84.47    .770E+07     2274.4          0.76        56.43
        9.70     85.00    .819E+07     2507.1          0.79        54.46

                                  <---- hydrograph ---->   <-pipe / channel->
                           AREA     QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                           (ha)     (cms)   (hrs)   (mm)      (m)       (m/s)
 INFLOW : ID= 2 (0956) 3783.02      5.85    1.08    4.08      0.27       0.09
 OUTFLOW: ID= 1 (7102) 3783.02      1.14    7.25    4.06      0.05       0.09

 ------------------------------------------------------------------------------

 --------------------
 | ADD HYD    (0957) |
 |  1 +  2 =  3      |          AREA     QPEAK     TPEAK      R.V.
 --------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0700):      307.23    1.345      1.08       3.83
        + ID2= 2 (7102):     3783.02    1.144      7.25       4.06
          ==================================================
          ID = 3 (0957):     4090.25    1.819      1.25       4.04

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ------------------------------------------------------------------------------
   ****************************
   ** SIMULATION NUMBER:  10 **
   ****************************

 --------------------
 |    READ STORM     |    Filename: C:\Users\RasoT\AppD
 |                   |              ata\Local\Temp\
 |                   |              31c99f01-c837-48de-ad6a-fc8b76397bcf\3654f719
 | Ptotal= 47.80 mm  |    Comments: Oct28/2015
 --------------------
              TIME    RAIN |   TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
               hrs   mm/hr |    hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
              0.08    0.00 |   4.33    2.40 |   8.58    2.40 | 12.83    0.00
              0.17    2.40 |   4.42    4.80 |   8.67    0.00 | 12.92    2.40
              0.25    0.00 |   4.50    2.40 |   8.75    2.40 | 13.00    0.00
              0.33    0.00 |   4.58    4.80 |   8.83    2.40 | 13.08    0.00
              0.42    0.00 |   4.67    4.80 |   8.92    2.40 | 13.17    4.80
              0.50    2.40 |   4.75    7.20 |   9.00    0.00 | 13.25    2.40
              0.58    0.00 |   4.83   12.00 |   9.08    4.80 | 13.33    2.40
              0.67    2.40 |   4.92   12.00 |   9.17    0.00 | 13.42    2.40
              0.75    0.00 |   5.00    7.20 |   9.25    2.40 | 13.50    7.20
              0.83    0.00 |   5.08    9.60 |   9.33    0.00 | 13.58    2.40
              0.92    0.00 |   5.17    9.60 |   9.42    2.40 | 13.67    0.00
              1.00    2.40 |   5.25    7.20 |   9.50    2.40 | 13.75    2.40
              1.08    0.00 |   5.33    9.60 |   9.58    2.40 | 13.83    0.00
              1.17    2.40 |   5.42    4.80 |   9.67    2.40 | 13.92    0.00
              1.25    0.00 |   5.50   12.00 |   9.75    4.80 | 14.00    0.00
              1.33    0.00 |   5.58    9.60 |   9.83    0.00 | 14.08    4.80
              1.42    0.00 |   5.67   12.00 |   9.92    2.40 | 14.17    0.00
              1.50    2.40 |   5.75   14.40 |  10.00    0.00 | 14.25    2.40
              1.58    0.00 |   5.83   14.40 |  10.08    4.80 | 14.33    0.00
              1.67    2.40 |   5.92    9.60 |  10.17    4.80 | 14.42    0.00
              1.75    0.00 |   6.00    7.20 |  10.25    4.80 | 14.50    2.40
              1.83    0.00 |   6.08   12.00 |  10.33    0.00 | 14.58    0.00
              1.92    2.40 |   6.17    9.60 |  10.42    4.80 | 14.67    0.00
              2.00    0.00 |   6.25    4.80 |  10.50    0.00 | 14.75    2.40
              2.08    2.40 |   6.33    9.60 |  10.58    2.40 | 14.83    0.00
              2.17    0.00 |   6.42    4.80 |  10.67    2.40 | 14.92    0.00
              2.25    0.00 |   6.50    2.40 |  10.75    2.40 | 15.00    2.40
              2.33    2.40 |   6.58    2.40 |  10.83    4.80 | 15.08    0.00
              2.42    0.00 |   6.67    4.80 |  10.92    2.40 | 15.17    0.00
              2.50    2.40 |   6.75    4.80 |  11.00    2.40 | 15.25    0.00
              2.58    0.00 |   6.83    2.40 |  11.08    0.00 | 15.33    2.40
              2.67    2.40 |   6.92    4.80 |  11.17    4.80 | 15.42    0.00
              2.75    2.40 |   7.00    7.20 |  11.25    2.40 | 15.50    0.00
              2.83    2.40 |   7.08    4.80 |  11.33    2.40 | 15.58    4.80
              2.92    2.40 |   7.17    7.20 |  11.42    0.00 | 15.67    0.00
              3.00    2.40 |   7.25    2.40 |  11.50    4.80 | 15.75    0.00
              3.08    2.40 |   7.33    2.40 |  11.58    0.00 | 15.83    0.00
              3.17    2.40 |   7.42    0.00 |  11.67    4.80 | 15.92    0.00
              3.25    2.40 |   7.50    4.80 |  11.75    4.80 | 16.00    2.40
              3.33    7.20 |   7.58    2.40 |  11.83    2.40 | 16.08    2.40
              3.42    2.40 |   7.67    4.80 |  11.92    2.40 | 16.17    0.00
              3.50    4.80 |   7.75    2.40 |  12.00    0.00 | 16.25    0.00
              3.58    4.80 |   7.83    2.40 |  12.08    4.80 | 16.33    0.00
              3.67    2.40 |   7.92    4.80 |  12.17    2.40 | 16.42    0.00
              3.75    2.40 |   8.00    2.40 |  12.25    2.40 | 16.50    0.00
              3.83    7.20 |   8.08    2.40 |  12.33    4.80 | 16.58    0.00
              3.92    2.40 |   8.17    2.40 |  12.42    2.40 | 16.67    0.00
              4.00    2.40 |   8.25    2.40 |  12.50    0.00 | 16.75    0.00
              4.08    4.80 |   8.33    2.40 |  12.58    2.40 | 16.83    0.00
              4.17    2.40 |   8.42    0.00 |  12.67    4.80 | 16.92    4.80
              4.25    2.40 |   8.50    2.40 |  12.75    2.40 |

 ------------------------------------------------------------------------------

 --------------------
 | CALIB              |
 | STANDHYD  (0700)  |   Area     (ha)= 307.23
 |ID= 1 DT= 5.0 min  |   Total Imp(%)=  24.70   Dir. Conn.(%)=  24.70
 --------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      75.89        231.35
     Dep. Storage     (mm)=       0.10          5.00
     Average Slope     (%)=       0.01          0.01
     Length            (m)=    1431.16       1431.00
     Mannings n            =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=      12.00          3.09
                over (min)       30.00       1310.00
     Storage Coeff.  (min)=     117.24 (ii) 1306.06 (ii)
     Unit Hyd. Tpeak (min)=      30.00       1310.00
     Unit Hyd. peak  (cms)=       0.01          0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=       1.38          0.26        1.379 (iii)
     TIME TO PEAK    (hrs)=       6.50         31.33        6.50
     RUNOFF VOLUME    (mm)=      47.70         15.26       23.28
     TOTAL RAINFALL   (mm)=      47.80         47.80       47.80
     RUNOFF COEFFICIENT    =      1.00          0.32        0.49

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

 ------------------------------------------------------------------------------
 --------------------
 | CALIB              |
 | STANDHYD  (0600)  |   Area     (ha)= 822.24
 |ID= 1 DT= 5.0 min  |   Total Imp(%)=  20.90   Dir. Conn.(%)=  20.90
 --------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=     171.85        650.39
     Dep. Storage     (mm)=       0.10          5.00
     Average Slope     (%)=       0.44          0.44
     Length            (m)=    2341.28       2341.00
     Mannings n            =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=      12.00          1.96
                over (min)       30.00        670.00
     Storage Coeff.  (min)=      50.62 (ii)   666.10 (ii)
     Unit Hyd. Tpeak (min)=      30.00        670.00
     Unit Hyd. peak  (cms)=       0.03          0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=       4.25          0.84        4.265 (iii)
     TIME TO PEAK    (hrs)=       6.25         21.25        6.33
     RUNOFF VOLUME    (mm)=      47.70         10.23       18.07
     TOTAL RAINFALL   (mm)=      47.80         47.80       47.80
     RUNOFF COEFFICIENT    =      1.00          0.21        0.38

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN*  =  65.1    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

 ------------------------------------------------------------------------------
 --------------------
 | CALIB              |
 | WILHYD     (0601) |   Area     (ha)= 552.24   Curve Number   (CN)= 72.7
 |ID= 1 DT= 5.0 min  |   Ia       (mm)=   5.00   Recession const.(K)= 1.55
 --------------------    U.H. Tp(hrs)=    1.83

     U.H. peak        (cms)=  12.09

     PEAK FLOW        (cms)=   1.99 (i)
     TIME TO PEAK     (hrs)=   8.33
     RUNOFF VOLUME     (mm)=  13.21
     TOTAL RAINFALL    (mm)=  47.80
     RUNOFF COEFFICIENT    =   0.28

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
 ------------------------------------------------------------------------------
 --------------------
 | RESERVOIR (1601) |
 | IN= 2---> OUT= 1 |
 | DT=  5.0 min      |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
 --------------------       (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |    2.2000     1.7880
                           0.0500     0.0017    |    3.4800     1.8760
                           0.1880     0.0265    |    9.1900     2.1400
                           0.5300     0.2125    |   12.0600     2.1410
                           0.9700     0.7171    |   12.0600     2.1500
                           1.5000     1.7000    |    0.0000     0.0000

                                 AREA       QPEAK      TPEAK       R.V.
                                 (ha)       (cms)      (hrs)       (mm)
     INFLOW : ID= 2 (0601)     552.245      1.991       8.33       13.21
     OUTFLOW: ID= 1 (1601)     552.245      1.451      15.50       13.21

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 72.87
               TIME SHIFT OF PEAK FLOW        (min)=430.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  1.6093

 ------------------------------------------------------------------------------
 --------------------
 | CALIB              |
 | STANDHYD  (0613)  |   Area     (ha)=  40.00
 |ID= 1 DT= 5.0 min  |   Total Imp(%)=  41.60   Dir. Conn.(%)=  41.60
 --------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      16.64         23.36
     Dep. Storage     (mm)=       0.10          5.00
     Average Slope     (%)=       0.42          0.42
     Length            (m)=     516.40        986.00
     Mannings n            =      0.013         0.250

     Max.Eff.Inten.(mm/hr)=      12.60          3.09
                over (min)       20.00        335.00
```

```
     Storage Coeff.  (min)=      20.32 (ii)  330.12 (ii)
     Unit Hyd. Tpeak (min)=      20.00       335.00
     Unit Hyd. peak  (cms)=       0.06         0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=       0.50         0.07        0.505 (iii)
     TIME TO PEAK    (hrs)=       6.08        16.25         6.08
     RUNOFF VOLUME    (mm)=      47.70        15.27        28.76
     TOTAL RAINFALL   (mm)=      47.80        47.80        47.80
     RUNOFF COEFFICIENT   =       1.00         0.32         0.60

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0058)   |   Area    (ha)=  59.90   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min   |   Ia      (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------    U.H. Tp(hrs)=   0.50

     Unit Hyd Qpeak  (cms)=   4.576

     PEAK FLOW       (cms)=   0.206 (i)
     TIME TO PEAK    (hrs)=   6.500
     RUNOFF VOLUME    (mm)=   7.309
     TOTAL RAINFALL   (mm)=  47.800
     RUNOFF COEFFICIENT   =   0.153

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0616)   |   Area    (ha)= 101.39
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  44.00   Dir. Conn.(%)=  44.00
--------------------
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      44.61        56.78
     Dep. Storage     (mm)=       0.10         5.00
     Average Slope     (%)=       0.62         0.62
     Length            (m)=     822.14       822.00
     Mannings n          =       0.013        0.250

     Max.Eff.Inten.(mm/hr)=      12.48         3.09
                over (min)       25.00       275.00
     Storage Coeff.  (min)=      23.99 (ii)  271.13 (ii)
     Unit Hyd. Tpeak (min)=      25.00       275.00
     Unit Hyd. peak  (cms)=       0.05         0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=       1.31         0.18        1.320 (iii)
     TIME TO PEAK    (hrs)=       6.17        15.33         6.17
     RUNOFF VOLUME    (mm)=      47.70        15.27        29.54
     TOTAL RAINFALL   (mm)=      47.80        47.80        47.80
     RUNOFF COEFFICIENT   =       1.00         0.32         0.62

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0617)   |   Area    (ha)=  39.29
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  40.30   Dir. Conn.(%)=  40.30
--------------------
```

```
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      15.83        23.45
     Dep. Storage     (mm)=       0.10         5.00
     Average Slope     (%)=       0.76         0.76
     Length            (m)=     511.78       512.00
     Mannings n          =       0.013        0.250

     Max.Eff.Inten.(mm/hr)=      13.60         3.09
                over (min)       15.00       195.00
     Storage Coeff.  (min)=      16.41 (ii)  191.42 (ii)
     Unit Hyd. Tpeak (min)=      15.00       195.00
     Unit Hyd. peak  (cms)=       0.07         0.01
                                                          *TOTALS*
     PEAK FLOW       (cms)=       0.50         0.08        0.508 (iii)
     TIME TO PEAK    (hrs)=       5.92        14.33         5.92
     RUNOFF VOLUME    (mm)=      47.70        15.27        28.34
     TOTAL RAINFALL   (mm)=      47.80        47.80        47.80
     RUNOFF COEFFICIENT   =       1.00         0.32         0.59

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0764) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  64.0) ------>
                 Distance      Elevation       Manning
                   100.00        160.00        0.1200
                   104.00        158.27        0.1200
                   108.00        156.96        0.1200
                   109.00        156.54     0.1200 /0.0500  Main Channel
                   110.50        156.30        0.0500       Main Channel
                   112.00        156.43        0.0500       Main Channel
                   112.50        156.90     0.0500 /0.1200  Main Channel
                   120.00        157.13        0.1200
                   130.00        158.90        0.1200
                   150.00        160.00        0.1200

        <-------------------- TRAVEL TIME TABLE --------------------->
         DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
          (m)       (m)     (cu.m.)       (cms)         (m/s)      (min)
          0.12    156.42    .168E+03       0.0          0.32       67.73
          0.24    156.54    .593E+03       0.3          0.57       38.26
          0.45    156.75    .151E+04       1.1          0.93       23.24
          0.65    156.95    .267E+04       2.4          1.16       18.66
          0.85    157.15    .524E+04       4.4          1.10       19.67
          1.06    157.36    .884E+04       7.5          1.11       19.53
          1.26    157.56    .129E+05      11.6          1.17       18.58
          1.46    157.76    .174E+05      16.5          1.23       17.58
          1.67    157.97    .224E+05      22.4          1.30       16.68
          1.87    158.17    .279E+05      29.3          1.36       15.89
          2.07    158.37    .338E+05      37.1          1.43       15.19
          2.28    158.58    .402E+05      46.0          1.49       14.57
          2.48    158.78    .470E+05      55.9          1.55       14.01
          2.68    158.98    .543E+05      66.0          1.58       13.71
          2.89    159.19    .625E+05      76.4          1.59       13.65
          3.09    159.39    .719E+05      88.5          1.60       13.54
          3.29    159.59    .824E+05     102.4          1.62       13.41
          3.50    159.80    .939E+05     118.0          1.63       13.26
          3.70    160.00    .107E+06     135.6          1.65       13.10

                                    <---- hydrograph ---->   <-pipe / channel->
                              AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                              (ha)    (cms)   (hrs)   (mm)     (m)      (m/s)
     INFLOW : ID= 2 (0617)   39.29     0.51    5.92  28.34     0.31      0.64
     OUTFLOW: ID= 1 (0764)   39.29     0.43    6.33  28.34     0.29      0.62
```

```
--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0618)   |   Area    (ha)= 111.19
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  45.00   Dir. Conn.(%)=  45.00
--------------------
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=      50.04        61.16
     Dep. Storage     (mm)=       0.10         5.00
     Average Slope     (%)=       0.49         0.49
     Length            (m)=     860.99       861.00
     Mannings n          =       0.013        0.250

     Max.Eff.Inten.(mm/hr)=      12.48         3.09
                over (min)       25.00       300.00
     Storage Coeff.  (min)=      26.47 (ii)  299.17 (ii)
     Unit Hyd. Tpeak (min)=      25.00       300.00
     Unit Hyd. peak  (cms)=       0.04         0.00
                                                          *TOTALS*
     PEAK FLOW       (cms)=       1.44         0.18        1.455 (iii)
     TIME TO PEAK    (hrs)=       6.17        15.75         6.17
     RUNOFF VOLUME    (mm)=      47.70        15.27        29.86
     TOTAL RAINFALL   (mm)=      47.80        47.80        47.80
     RUNOFF COEFFICIENT   =       1.00         0.32         0.62

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0763) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  63.0) ------>
                 Distance      Elevation       Manning
                   100.00        160.00        0.1200
                   104.00        158.27        0.1200
                   108.00        156.96        0.1200
                   109.00        156.54     0.1200 /0.0500  Main Channel
                   110.50        156.30        0.0500       Main Channel
                   112.00        156.43        0.0500       Main Channel
                   112.50        156.90     0.0500 /0.1200  Main Channel
                   120.00        157.13        0.1200
                   130.00        158.90        0.1200
                   150.00        160.00        0.1200

        <-------------------- TRAVEL TIME TABLE --------------------->
         DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
          (m)       (m)     (cu.m.)       (cms)         (m/s)      (min)
          0.12    156.42    .158E+03       0.0          0.32       62.69
          0.24    156.54    .557E+03       0.3          0.57       35.41
          0.45    156.75    .142E+04       1.1          0.95       21.51
          0.65    156.95    .250E+04       2.4          1.18       17.27
          0.85    157.15    .492E+04       4.5          1.12       18.20
          1.06    157.36    .830E+04       7.6          1.12       18.08
          1.26    157.56    .121E+05      11.7          1.18       17.20
          1.46    157.76    .164E+05      16.8          1.25       16.27
          1.67    157.97    .211E+05      22.7          1.32       15.44
          1.87    158.17    .262E+05      29.7          1.38       14.71
          2.07    158.37    .318E+05      37.6          1.45       14.06
          2.28    158.58    .377E+05      46.6          1.51       13.48
          2.48    158.78    .441E+05      56.7          1.57       12.97
          2.68    158.98    .509E+05      66.9          1.60       12.69
          2.89    159.19    .587E+05      77.4          1.61       12.63
          3.09    159.39    .675E+05      89.7          1.62       12.53
          3.29    159.59    .773E+05     103.8          1.64       12.41
```

```
          3.50    159.80    .881E+05     119.7          1.66       12.27
          3.70    160.00    .100E+06     137.5          1.68       12.12

                                    <---- hydrograph ---->   <-pipe / channel->
                              AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                              (ha)    (cms)   (hrs)   (mm)     (m)      (m/s)
     INFLOW : ID= 2 (0618)  111.19     1.46    6.17  29.86     0.50      1.00
     OUTFLOW: ID= 1 (0763)  111.19     1.36    6.33  29.86     0.49      0.98

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0896) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0763):    111.19    1.360      6.33      29.86
        + ID2= 2 (0764):     39.29    0.430      6.33      28.34
          ==================================================
          ID = 3 (0896):    150.48    1.791      6.33      29.46

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------

--------------------
| ADD HYD   (0897) |
|   1 +  2 =  3    |          AREA     QPEAK     TPEAK      R.V.
--------------------          (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0616):    101.39    1.320      6.17      29.54
        + ID2= 2 (0896):    150.48    1.791      6.33      29.46
          ==================================================
          ID = 3 (0897):    251.87    3.070      6.17      29.49

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0765) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                 <------ DATA FOR SECTION (  65.0) ------>
                 Distance      Elevation       Manning
                   100.00        160.00        0.1200
                   104.00        158.27        0.1200
                   108.00        156.96        0.1200
                   109.00        156.54     0.1200 /0.0500  Main Channel
                   110.50        156.30        0.0500       Main Channel
                   112.00        156.43        0.0500       Main Channel
                   112.50        156.90     0.0500 /0.1200  Main Channel
                   120.00        157.13        0.1200
                   130.00        158.90        0.1200
                   150.00        160.00        0.1200

        <-------------------- TRAVEL TIME TABLE --------------------->
         DEPTH     ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
          (m)       (m)     (cu.m.)       (cms)         (m/s)      (min)
          0.12    156.42    .417E+03       0.1          0.45      120.01
          0.24    156.54    .147E+04       0.4          0.79       67.79
          0.45    156.75    .375E+04       1.5          1.31       41.18
          0.65    156.95    .662E+04       3.3          1.63       33.07
          0.85    157.15    .130E+05       6.2          1.54       34.85
          1.06    157.36    .219E+05      10.6          1.55       34.61
          1.26    157.56    .320E+05      16.2          1.63       32.92
          1.46    157.76    .433E+05      23.2          1.73       31.15
          1.67    157.97    .557E+05      31.4          1.82       29.56
          1.87    158.17    .693E+05      41.0          1.91       28.16
          2.07    158.37    .840E+05      52.0          2.00       26.92
          2.28    158.58    .998E+05      64.4          2.08       25.81
          2.48    158.78    .117E+06      78.3          2.17       24.82
          2.68    158.98    .135E+06      92.4          2.21       24.29
          2.89    159.19    .155E+06     107.0          2.22       24.18
```

```
  3.09   159.39    .178E+06     124.0           2.24        23.99
  3.29   159.59    .204E+06     143.4           2.26        23.76
  3.50   159.80    .233E+06     165.4           2.29        23.50
  3.70   160.00    .265E+06     190.0           2.32        23.21

                             <---- hydrograph ---->   <-pipe / channel->
                      AREA     QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                      (ha)     (cms)   (hrs)   (mm)      (m)      (m/s)
INFLOW : ID= 2 (0897) 251.87    3.07    6.17  29.49     0.62      1.57
OUTFLOW: ID= 1 (0765) 251.87    2.75    6.50  29.49     0.59      1.51

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0614)   |   Area    (ha)= 450.63
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  40.40   Dir. Conn.(%)=  40.40
--------------------
                            IMPERVIOUS     PERVIOUS (i)
    Surface Area      (ha)=    182.06        268.58
    Dep. Storage      (mm)=      0.10          5.00
    Average Slope      (%)=      0.65          0.65
    Length             (m)=   1733.27       1730.00
    Mannings n           =      0.013         0.250

    Max.Eff.Inten.(mm/hr)=     12.00          3.09
               over (min)      30.00        420.00
    Storage Coeff.  (min)=     37.59 (ii)   418.38 (ii)
    Unit Hyd. Tpeak (min)=     30.00        420.00
    Unit Hyd. peak  (cms)=      0.03          0.00
                                                         *TOTALS*
    PEAK FLOW       (cms)=      4.86          0.69         4.889 (iii)
    TIME TO PEAK    (hrs)=      6.25         17.33          6.25
    RUNOFF VOLUME    (mm)=     47.70         15.27         28.37
    TOTAL RAINFALL   (mm)=     47.80         47.80         47.80
    RUNOFF COEFFICIENT   =      1.00          0.32          0.59

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN* =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0898) |
|   1 +  2 =  3    |        AREA     QPEAK    TPEAK     R.V.
--------------------        (ha)     (cms)    (hrs)     (mm)
          ID1= 1 (0614):   450.63    4.889     6.25    28.37
        + ID2= 2 (0765):   251.87    2.755     6.50    29.49
          ===============================================
          ID = 3 (0898):   702.50    7.525     6.33    28.77

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (2614) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    5.4330     8.9450
                          0.0550     2.6410   |    9.6740     9.4540
                          0.2450     3.4540   |   15.3290     9.9630
                          0.6180     4.7960   |   22.1250    10.4710
                          1.1140     5.5860   |   38.6530    11.4890
                          1.9930     7.9280   |   48.2480    11.9980

                             AREA     QPEAK      TPEAK       R.V.
                             (ha)     (cms)      (hrs)       (mm)
```

```
  INFLOW : ID= 2 (0898)    702.503      7.525      6.33      28.77
  OUTFLOW: ID= 1 (2614)    702.503      2.965     13.08      27.06

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 39.40
               TIME SHIFT OF PEAK FLOW        (min)=405.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  8.2158

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (1614) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    4.8340     4.8070
                          0.7000     0.6320   |   10.5600     5.7180
                          1.5360     1.2640   |   19.5800     6.8560
                          2.0480     1.5790   |   24.8600     7.4250
                          2.5600     1.9520   |   29.1700     7.9930
                          3.4560     2.6960   |   32.5000    10.1310
                          4.0940     3.4400   |    0.0000     0.0000

                              AREA     QPEAK      TPEAK       R.V.
                              (ha)     (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (2614)    702.503      2.965     13.08      27.06
  OUTFLOW: ID= 1 (1614)    702.503      2.656     14.92      27.03

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 89.57
               TIME SHIFT OF PEAK FLOW        (min)=110.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  2.0317

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0615)   |   Area    (ha)= 344.12
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  47.20   Dir. Conn.(%)=  47.20
--------------------
                            IMPERVIOUS     PERVIOUS (i)
    Surface Area      (ha)=    162.43        181.70
    Dep. Storage      (mm)=      0.10          5.00
    Average Slope      (%)=      0.79          0.79
    Length             (m)=   1514.64       1515.00
    Mannings n           =      0.013         0.250

    Max.Eff.Inten.(mm/hr)=     12.00          3.09
               over (min)      30.00        365.00
    Storage Coeff.  (min)=     32.70 (ii)   364.36 (ii)
    Unit Hyd. Tpeak (min)=     30.00        365.00
    Unit Hyd. peak  (cms)=      0.04          0.00
                                                         *TOTALS*
    PEAK FLOW       (cms)=      4.46          0.50         4.489 (iii)
    TIME TO PEAK    (hrs)=      6.25         16.67          6.25
    RUNOFF VOLUME    (mm)=     47.70         15.27         30.58
    TOTAL RAINFALL   (mm)=     47.80         47.80         47.80
    RUNOFF COEFFICIENT   =      1.00          0.32          0.64

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN* =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (1615) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    4.7400     3.1500
                          1.8400     0.1500   |    6.6000     3.8100
                          2.2800     0.3230   |    8.9000     5.1500
                          2.6400     0.7000   |   13.4000     5.8230
```

```
                          3.0000     2.0000   |   17.0000     6.1000

                              AREA     QPEAK      TPEAK       R.V.
                              (ha)     (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (0615)    344.121      4.489      6.25      30.58
  OUTFLOW: ID= 1 (1615)    344.121      2.724      7.17      30.58

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 60.69
               TIME SHIFT OF PEAK FLOW        (min)= 55.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  1.0055

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0899) |
|   1 +  2 =  3    |        AREA     QPEAK    TPEAK     R.V.
--------------------        (ha)     (cms)    (hrs)     (mm)
          ID1= 1 (1614):   702.50    2.656    14.92    27.03
        + ID2= 2 (1615):   344.12    2.724     7.17    30.58
          ===============================================
          ID = 3 (0899):  1046.62    4.134    14.00    28.20

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
  --------------------
  | ROUTE MC (0075)  |
  | IN=2---> OUT=1   |
  --------------------

  ROUTING TIME STEP = 5.0 min.

  <------------------DATA FOR SECTION (1.1)------------------>
  Distance (m)     Elevation (m)     Mannings 'n'
    100.0             160.00            0.12
    104.0             158.27            0.12
    108.0             156.96            0.12
    109.0             156.55            0.12/0.05    Main Channel
    110.5             156.30            0.05         Main Channel
    112.0             156.44            0.05         Main Channel
    112.5             156.90            0.05/0.12    Main Channel
    120.0             157.13            0.12
    130.0             158.90            0.12
    150.0             160.00            0.12

                           AREA      QPEAK    TPEAK     R.V.    QBASE
                           (ha)      (cms)    (hrs)     (mm)    (cms)
   INFLOW:  ID=2  (0899)  1046.62     4.13    14.00    28.20     0.0
   OUTFLOW: ID=1  (0075)  1046.62     3.93    17.42    28.20     0.0

  ---------------------------------------------------------------------------------
--------------------
| ADD HYD   (0900) |
|   1 +  2 =  3    |        AREA     QPEAK    TPEAK     R.V.
--------------------        (ha)     (cms)    (hrs)     (mm)
          ID1= 1 (0058):    59.90    0.206     6.50     7.31
        + ID2= 2 (0075):  1046.62    3.931    17.42    28.20
          ===============================================
          ID = 3 (0900):  1106.52    3.958    17.42    27.03

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0900) |
|   3 +  2 =  1    |        AREA     QPEAK    TPEAK     R.V.
--------------------        (ha)     (cms)    (hrs)     (mm)
          ID1= 3 (0900):  1106.52    3.958    17.42    27.03
        + ID2= 2 (0613):    40.00    0.505     6.08    28.76
          ===============================================
```

```
          ID = 1 (0900):  1146.52    4.049    17.25    27.09

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0073) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |    (cms)     (ha.m.)
                          0.0000     0.0000   |    1.5000     8.0000
                          0.2000     0.5000   |    3.0000     9.0000
                          0.6000     1.5000   |    5.0000    10.0000
                          1.0000     2.0000   |   10.0000    10.0050
                          1.2000     5.0000   |    0.0000     0.0000

                              AREA     QPEAK      TPEAK       R.V.
                              (ha)     (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (0900)   1146.524      4.049     17.25      27.09
  OUTFLOW: ID= 1 (0073)   1146.524      3.367     20.33      27.03

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 83.15
               TIME SHIFT OF PEAK FLOW        (min)=185.00
               MAXIMUM  STORAGE   USED       (ha.m.)=  9.1835

  **** WARNING : HYDROGRAPH WAS CUT. CHECK VOLUME.
--------------------------------------------------------------------------------
--------------------
| DIVERT HYD (0001)|
| IN= 1   # OUT= 5 |
--------------------
  Outflow / Inflow  Relationships

   Flow 1 + Flow 2 + Flow 3 + Flow 4 + Flow 5 = Total
   (cms)    (cms)    (cms)    (cms)    (cms)    (cms)
    0.00     0.00     0.00     0.00     0.00     0.00
    0.70     0.70     0.00     0.00     0.00     1.40
    2.00     1.00     0.00     0.00     0.00     3.00
    3.10     2.60     0.00     0.00     0.00     5.70
    9.60     3.60     0.00     0.00     0.00    13.20
   14.30     4.60     0.00     0.00     0.00    18.90
   20.10     5.80     0.00     0.00     0.00    25.90
   25.40     6.50     0.00     0.00     0.00    31.90
   30.50     7.50     0.00     0.00     0.00    38.00
   95.80    19.20     0.00     0.00     0.00   115.00

                           AREA      QPEAK      TPEAK     R.V.
                           (ha)      (cms)      (hrs)     (mm)
  TOTAL HYD.(ID= 1):1146.52          3.37      20.33    27.03
  =========================================================
  ID= 2  (    2)     : 636.80         2.15      20.33    27.03
  ID= 3  (    2)     : 509.73         1.22      20.33    27.03
  ID= 4  (    2)     :   0.00         0.00       0.00     0.00
  ID= 5  (    2)     :   0.00         0.00       0.00     0.00
  ID= 6  (    2)     :   0.00         0.00       0.00     0.00
--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD  (0894)   |   Area    (ha)=  10.00
|ID= 1 DT= 5.0 min   |   Total Imp(%)=  50.00   Dir. Conn.(%)=  35.00
--------------------
                            IMPERVIOUS     PERVIOUS (i)
    Surface Area      (ha)=      5.00          5.00
    Dep. Storage      (mm)=      1.00          1.50
    Average Slope      (%)=      1.00          2.00
    Length             (m)=    258.20         40.00
    Mannings n           =      0.013         0.250

    Max.Eff.Inten.(mm/hr)=     14.40          8.85
               over (min)      10.00         30.00
    Storage Coeff.  (min)=      9.80 (ii)    28.42 (ii)
    Unit Hyd. Tpeak (min)=     10.00         30.00
```

```
 Unit Hyd. peak  (cms)=       0.11          0.04
                                                        *TOTALS*
 PEAK FLOW       (cms)=       0.12          0.08         0.179 (iii)
 TIME TO PEAK    (hrs)=       5.92          6.25          6.17
 RUNOFF VOLUME    (mm)=      46.80          5.33         19.84
 TOTAL RAINFALL   (mm)=      47.80         47.80         47.80
 RUNOFF COEFFICIENT   =       0.98          0.11          0.42

   (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
        Fo   (mm/hr)= 50.00           K  (1/hr)=  2.00
        Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=  0.00
  (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
       THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0999) |
|   1 +  2 =  3    |          AREA    QPEAK     TPEAK      R.V.
--------------------          (ha)    (cms)     (hrs)      (mm)
          ID1= 1 (0001):    636.80   2.149     20.33     27.03
        + ID2= 2 (0894):     10.00   0.179      6.17     19.84
          ==================================================
          ID = 3 (0999):    646.80   2.149     20.33     26.92

 NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0762) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
                <------ DATA FOR SECTION (  62.0) ------>
                Distance       Elevation        Manning
                    0.00          145.00         0.1800
                    4.00          140.00         0.1800
                   15.00          135.00         0.1800
                   20.00          133.00         0.1800
                   25.00          132.00         0.1800
                   45.00          131.00         0.1800
                   93.00          132.00         0.1800
                  114.00          132.00         0.1800
                  210.00          131.00         0.1800
                  238.00          130.00         0.1800
                  239.00          129.00      0.1800 /0.0700  Main Channel
                  244.00          128.00         0.0700       Main Channel
                  262.00          128.00         0.0700       Main Channel
                  264.00          129.00      0.0700 /0.1950  Main Channel
                  306.00          129.00         0.1950
                  318.00          130.00         0.1950
                  335.00          129.00         0.1950
                  345.00          130.00         0.1950
                  354.00          135.00         0.1950
                  365.00          145.00         0.1950

      <-------------------- TRAVEL TIME TABLE --------------------->
       DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
        (m)       (m)      (cu.m.)       (cms)        (m/s)       (min)
       0.50    128.50     .788E+05        5.9          0.60      222.28
       1.00    129.00     .172E+06       19.5          0.91      146.86
       1.94    129.94     .816E+06       83.4          0.82      163.10
       2.88    130.88     .171E+07      204.0          0.95      139.80
       3.82    131.82     .319E+07      387.6          0.97      137.01
       4.76    132.76     .558E+07      698.4          1.00      133.20
       5.71    133.71     .807E+07     1127.5          1.11      119.29
       6.65    134.65     .106E+08     1653.2          1.25      106.75
       7.59    135.59     .131E+08     2268.5          1.38       96.52
       8.53    136.53     .157E+08     2966.8          1.51       88.25
       9.47    137.47     .183E+08     3742.6          1.63       81.51
      10.41    138.41     .209E+08     4592.4          1.75       75.93
```

```
      11.35    139.35     .236E+08     5513.5          1.87       71.23
      12.29    140.29     .262E+08     6506.2          1.98       67.19
      13.24    141.24     .289E+08     7572.0          2.09       63.63
      14.18    142.18     .316E+08     8702.5          2.20       60.53
      15.12    143.12     .343E+08     9895.6          2.30       57.79
      16.06    144.06     .370E+08    11150.0          2.40       55.36
      17.00    145.00     .398E+08    12463.9          2.50       53.18

                                    <---- hydrograph ---->   <-pipe / channel->
                              AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                              (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
 INFLOW : ID= 2 (0999)      646.80     2.15   20.33  26.92      0.18      0.60
 OUTFLOW: ID= 1 (0762)      646.80     1.75   24.33  26.89      0.15      0.60

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0895) |
|   1 +  2 =  3    |          AREA    QPEAK     TPEAK      R.V.
--------------------          (ha)    (cms)     (hrs)      (mm)
          ID1= 1 (0762):    646.80   1.753     24.33     26.89
        + ID2= 2 (1601):    552.24   1.451     15.50     13.21
          ==================================================
          ID = 3 (0895):   1199.04   2.360     20.67     20.59

 NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0205) |   Area    (ha)=   0.00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  35.00
--------------------
                             IMPERVIOUS    PERVIOUS (i)
 Surface Area     (ha)=         0.00          0.00
 Dep. Storage     (mm)=         1.00          1.50
 Average Slope     (%)=         1.00          2.00
 Length            (m)=         0.00         40.00
 Mannings n           =        0.013         0.250

 Max.Eff.Inten.(mm/hr)=        14.40          8.85
             over (min)         5.00         20.00
 Storage Coeff.  (min)=         0.00 (ii)    18.62 (ii)
 Unit Hyd. Tpeak (min)=         5.00         20.00
 Unit Hyd. peak  (cms)=         0.34          0.10
                                                        *TOTALS*
 PEAK FLOW       (cms)=         0.00          0.00         0.000 (iii)
 TIME TO PEAK    (hrs)=         0.00          0.00          0.00
 RUNOFF VOLUME    (mm)=       NaN           NaN           NaN
 TOTAL RAINFALL   (mm)=        47.80         47.80         47.80
 RUNOFF COEFFICIENT   =       NaN           NaN           NaN

***** WARNING: STORAGE COEFF. IS SMALLER THAN TIME STEP!

   (i) HORTONS EQUATION SELECTED FOR PERVIOUS LOSSES:
        Fo   (mm/hr)= 50.00           K  (1/hr)=  2.00
        Fc   (mm/hr)=  7.50     Cum.Inf.   (mm)=  0.00
  (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
       THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0023) |   Area    (ha)=  19.20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
--------------------
                             IMPERVIOUS    PERVIOUS (i)
 Surface Area     (ha)=        10.06          9.14
 Dep. Storage     (mm)=         0.10          5.00
 Average Slope     (%)=         0.99          0.99
```

```
 Length            (m)=       357.77        358.00
 Mannings n           =        0.013         0.250

 Max.Eff.Inten.(mm/hr)=        14.40          1.77
             over (min)        10.00        175.00
 Storage Coeff.  (min)=        11.95 (ii)   174.95 (ii)
 Unit Hyd. Tpeak (min)=        10.00        175.00
 Unit Hyd. peak  (cms)=         0.10          0.01
                                                        *TOTALS*
 PEAK FLOW       (cms)=         0.34          0.02         0.341 (iii)
 TIME TO PEAK    (hrs)=         5.92         14.50          5.92
 RUNOFF VOLUME    (mm)=        47.70          9.32         29.43
 TOTAL RAINFALL   (mm)=        47.80         47.80         47.80
 RUNOFF COEFFICIENT   =         1.00          0.20          0.62

   (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
        CN*  =  62.3    Ia = Dep. Storage  (Above)
  (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
       THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0004) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |       OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------        (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |   0.0280     0.2320
                           0.0130     0.0290    |   0.0290     0.2610
                           0.0160     0.0580    |   0.0320     0.2900
                           0.0190     0.0870    |   0.0340     0.3480
                           0.0210     0.1160    |   0.0360     0.4610
                           0.0230     0.1450    |   0.3056     0.5278
                           0.0250     0.1740    |   0.7985     0.5947
                           0.0270     0.2030    |   1.4368     0.6615

                             AREA      QPEAK      TPEAK       R.V.
                             (ha)      (cms)      (hrs)       (mm)
 INFLOW : ID= 2 (0023)      19.200     0.341       5.92       29.43
 OUTFLOW: ID= 1 (0004)      19.200     0.035      16.33       29.40

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 10.20
              TIME SHIFT OF PEAK FLOW        (min)=625.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  0.3922

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0021) |   Area    (ha)=  55.00
|ID= 1 DT= 5.0 min |   Total Imp(%)=  41.60   Dir. Conn.(%)=  41.60
--------------------
                             IMPERVIOUS    PERVIOUS (i)
 Surface Area     (ha)=        22.88         32.12
 Dep. Storage     (mm)=         0.10          5.00
 Average Slope     (%)=         0.42          0.42
 Length            (m)=       605.53        605.00
 Mannings n           =        0.013         0.250

 Max.Eff.Inten.(mm/hr)=        12.60          3.09
             over (min)        20.00        255.00
 Storage Coeff.  (min)=        22.36 (ii)   253.47 (ii)
 Unit Hyd. Tpeak (min)=        20.00        255.00
 Unit Hyd. peak  (cms)=         0.05          0.00
                                                        *TOTALS*
 PEAK FLOW       (cms)=         0.68          0.10         0.688 (iii)
 TIME TO PEAK    (hrs)=         6.08         15.00          6.08
 RUNOFF VOLUME    (mm)=        47.70         15.27         28.76
 TOTAL RAINFALL   (mm)=        47.80         47.80         47.80
 RUNOFF COEFFICIENT   =         1.00          0.32          0.60
```

```
   (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
        CN*  =  76.7    Ia = Dep. Storage  (Above)
  (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
       THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0018) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |       OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------        (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |   0.0728     0.9731
                           0.0130     0.1216    |   0.0778     1.0947
                           0.0275     0.2433    |   0.0825     1.2164
                           0.0389     0.3649    |   0.0870     1.3380
                           0.0476     0.4865    |   1.2731     1.4845
                           0.0550     0.6082    |   3.4420     1.6310
                           0.0615     0.7298    |   6.2505     1.7775
                           0.0674     0.8515    |   0.0000     0.0000

                             AREA      QPEAK      TPEAK       R.V.
                             (ha)      (cms)      (hrs)       (mm)
 INFLOW : ID= 2 (0021)      55.000     0.688       6.08       28.76
 OUTFLOW: ID= 1 (0018)      55.000     0.079      18.67       28.66

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 11.45
              TIME SHIFT OF PEAK FLOW        (min)=755.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  1.1222

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0022) |   Area    (ha)=  45.60
|ID= 1 DT= 5.0 min |   Total Imp(%)=  63.10   Dir. Conn.(%)=  63.10
--------------------
                             IMPERVIOUS    PERVIOUS (i)
 Surface Area     (ha)=        28.77         16.83
 Dep. Storage     (mm)=         0.10          5.00
 Average Slope     (%)=         0.97          0.97
 Length            (m)=       551.36        551.00
 Mannings n           =        0.013         0.250

 Max.Eff.Inten.(mm/hr)=        13.60          3.09
             over (min)        15.00        190.00
 Storage Coeff.  (min)=        15.95 (ii)   185.93 (ii)
 Unit Hyd. Tpeak (min)=        15.00        190.00
 Unit Hyd. peak  (cms)=         0.07          0.01
                                                        *TOTALS*
 PEAK FLOW       (cms)=         0.91          0.06         0.918 (iii)
 TIME TO PEAK    (hrs)=         5.92         14.33          5.92
 RUNOFF VOLUME    (mm)=        47.70         15.27         35.73
 TOTAL RAINFALL   (mm)=        47.80         47.80         47.80
 RUNOFF COEFFICIENT   =         1.00          0.32          0.75

   (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
        CN*  =  76.7    Ia = Dep. Storage  (Above)
  (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
       THAN THE STORAGE COEFFICIENT.
 (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0019) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |       OUTFLOW    STORAGE   |   OUTFLOW    STORAGE
--------------------        (cms)     (ha.m.)   |    (cms)     (ha.m.)
                           0.0000     0.0000    |   0.0704     0.8335
                           0.0056     0.0926    |   0.0747     0.9262
                           0.0254     0.1852    |   0.0787     1.0188
                           0.0355     0.2778    |   0.0825     1.1114
```

```
                   0.0433      0.3705    |    0.0844      1.2040
                   0.0500      0.4631    |    1.4323      1.3120
                   0.0558      0.5557    |    3.8969      1.4200
                   0.0611      0.6483    |    7.0884      1.5280
                   0.0659      0.7409    |    0.0000      0.0000

                             AREA      QPEAK      TPEAK       R.V.
                             (ha)      (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (0022)     45.600      0.918       5.92      35.73
  OUTFLOW: ID= 1 (0019)     45.600      0.134      15.83      35.34

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 14.60
              TIME SHIFT OF PEAK FLOW         (min)=595.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  1.2083

--------------------------------------------------------------------------

--------------------
| ADD HYD    (0015) |
|   1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0018):      55.00    0.079     18.67     28.66
        + ID2= 2 (0019):      45.60    0.134     15.83     35.34
          ===================================================
          ID = 3 (0015):     100.60    0.211     15.83     31.69

  NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------

--------------------
| CALIB              |
| STANDHYD  (0612) |   Area     (ha)=  97.00
|ID= 1 DT=15.0 min |   Total Imp(%)=  63.10   Dir. Conn.(%)=  63.10
--------------------
                              IMPERVIOUS    PERVIOUS (i)
  Surface Area     (ha)=        61.21         35.79
  Dep. Storage     (mm)=         0.10          5.00
  Average Slope    (%)=          0.97          0.97
  Length           (m)=        804.16       1143.00
  Mannings n          =         0.013         0.250

     NOTE:  RAINFALL WAS TRANSFORMED TO  15.0 MIN. TIME STEP.

                         ---- TRANSFORMED HYETOGRAPH ----
         TIME    RAIN |  TIME    RAIN |' TIME    RAIN |  TIME    RAIN
          hrs   mm/hr |   hrs   mm/hr |'  hrs   mm/hr |   hrs   mm/hr
        0.250    0.80 | 4.500    3.20 | 8.750    1.60 | 13.00    0.80
        0.500    0.80 | 4.750    5.60 | 9.000    1.60 | 13.25    2.40
        0.750    0.80 | 5.000   10.40 | 9.250    2.40 | 13.50    4.00
        1.000    0.80 | 5.250    8.80 | 9.500    1.60 | 13.75    1.60
        1.250    0.80 | 5.500    8.80 | 9.750    3.20 | 14.00    0.00
        1.500    0.80 | 5.750   12.00 |10.000    0.80 | 14.25    2.40
        1.750    0.80 | 6.000   10.40 |10.250    4.80 | 14.50    0.80
        2.000    0.80 | 6.250    8.80 |10.500    1.60 | 14.75    0.80
        2.250    0.80 | 6.500    5.60 |10.750    2.40 | 15.00    0.80
        2.500    1.60 | 6.750    4.00 |11.000    3.20 | 15.25    0.00
        2.750    1.60 | 7.000    4.80 |11.250    2.40 | 15.50    0.80
        3.000    2.40 | 7.250    4.80 |11.500    2.40 | 15.75    1.60
        3.250    2.40 | 7.500    2.40 |11.750    3.20 | 16.00    0.80
        3.500    4.80 | 7.750    3.20 |12.000    1.60 | 16.25    0.80
        3.750    3.20 | 8.000    3.20 |12.250    3.20 | 16.50    0.00
        4.000    4.00 | 8.250    2.40 |12.500    2.40 | 16.75    0.00
        4.250    3.20 | 8.500    1.60 |12.750    3.20 | 17.00    3.20

  Max.Eff.Inten.(mm/hr)=       12.00           2.93
             over (min)        15.00         300.00
  Storage Coeff.  (min)=       21.03 (ii)    290.15 (ii)
  Unit Hyd. Tpeak (min)=       15.00         300.00
  Unit Hyd. peak  (cms)=        0.06           0.00
                                                          *TOTALS*
  PEAK FLOW       (cms)=        1.76           0.11         1.765 (iii)
```

```
  TIME TO PEAK    (hrs)=        6.00          15.75            6.00
  RUNOFF VOLUME    (mm)=       48.10          15.51           36.07
  TOTAL RAINFALL   (mm)=       48.20          48.20           48.20
  RUNOFF COEFFICIENT   =        1.00           0.32            0.75

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------

--------------------
| RESERVOIR (0050) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |      OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           0.0000     0.0000   |   4.8880      1.6100
                           0.1710     1.2880   |  15.0000     18.5000

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (0612)     97.000      1.765       6.00      36.07
  OUTFLOW: ID= 1 (0050)     97.000      0.962       6.83      36.05

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 54.53
              TIME SHIFT OF PEAK FLOW         (min)= 50.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  1.3420

--------------------------------------------------------------------------

--------------------
| RESERVOIR (0056) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |      OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)   |   (cms)     (ha.m.)
                           0.0000     0.0000   |   5.6570      2.7400
                           2.5000     2.1920   |  15.0000      2.9500

                              AREA      QPEAK      TPEAK       R.V.
                              (ha)      (cms)      (hrs)       (mm)
  INFLOW : ID= 2 (0050)     97.000      0.962       6.83      36.05
  OUTFLOW: ID= 1 (0056)     97.000      0.504      13.83      36.05

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 52.37
              TIME SHIFT OF PEAK FLOW         (min)=420.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  0.4409

--------------------------------------------------------------------------

--------------------
| ADD HYD    (0051) |
|   1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0001):     509.73    1.217     20.33     27.03
        + ID2= 2 (0056):      97.00    0.504     13.83     36.05
          ===================================================
          ID = 3 (0051):     606.73    1.434     20.17     28.47

  NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------

--------------------
| ROUTE CHN (0767) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
            <------ DATA FOR SECTION (  67.0) ------>
            Distance       Elevation        Manning
               99.00          152.00         0.1200
              100.00          150.00         0.1200
              125.00          149.12         0.1200
              130.00          148.99     0.1200 /0.0500  Main Channel
```

```
              131.00         148.23         0.0500      Main Channel
              131.75         148.12         0.0500      Main Channel
              132.50         148.19         0.0500      Main Channel
              133.00         148.66     0.0500 /0.1200  Main Channel
              138.00         149.10         0.1200
              149.00         150.00         0.1200
              150.00         152.00         0.1200

  <-------------------- TRAVEL TIME TABLE --------------------->
   DEPTH    ELEV      VOLUME     FLOW RATE     VELOCITY   TRAV.TIME
    (m)      (m)     (cu.m.)      (cms)         (m/s)       (min)
   0.18   148.30    .651E+02       0.1           0.28       17.58
   0.36   148.49    .169E+03       0.3           0.45       11.12
   0.54   148.66    .297E+03       0.6           0.57        8.82
   0.75   148.87    .542E+03       1.1           0.63        7.98
   0.96   149.08    .100E+04       2.0           0.61        8.19
   1.17   149.29    .199E+04       3.6           0.54        9.32
   1.37   149.50    .351E+04       5.9           0.50        9.91
   1.58   149.71    .557E+04       9.3           0.50        9.99
   1.79   149.92    .815E+04      13.9           0.51        9.80
   2.00   150.12    .112E+05      20.4           0.55        9.14
   2.21   150.33    .142E+05      28.6           0.60        8.30
   2.42   150.54    .173E+05      38.0           0.66        7.60
   2.62   150.75    .204E+05      48.5           0.71        7.02
   2.83   150.96    .236E+05      60.1           0.76        6.54
   3.04   151.17    .267E+05      72.6           0.82        6.13
   3.25   151.37    .298E+05      86.0           0.86        5.78
   3.46   151.58    .330E+05     100.3           0.91        5.48
   3.67   151.79    .362E+05     115.5           0.96        5.22
   3.87   152.00    .393E+05     131.5           1.00        4.98

                                  <---- hydrograph ---->   <-pipe / channel->
                           AREA    QPEAK   TPEAK   R.V.    MAX DEPTH   MAX VEL
                           (ha)    (cms)   (hrs)   (mm)       (m)       (m/s)
  INFLOW : ID= 2 (0051)  606.73     1.43   20.17  28.47      0.82        0.62
  OUTFLOW: ID= 1 (0767)  606.73     1.43   20.33  28.47      0.82        0.62

--------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0059) |   Area     (ha)=  12.10   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

     NOTE:  RAINFALL WAS TRANSFORMED TO   5.0 MIN. TIME STEP.

                         ---- TRANSFORMED HYETOGRAPH ----
         TIME    RAIN |  TIME    RAIN |' TIME    RAIN |  TIME    RAIN
          hrs   mm/hr |   hrs   mm/hr |'  hrs   mm/hr |   hrs   mm/hr
        0.083    0.00 | 4.333    2.40 | 8.583    2.40 | 12.83    0.00
        0.167    2.40 | 4.417    4.80 | 8.667    0.00 | 12.92    2.40
        0.250    0.00 | 4.500    2.40 | 8.750    2.40 | 13.00    0.00
        0.333    0.00 | 4.583    4.80 | 8.833    2.40 | 13.08    0.00
        0.417    0.00 | 4.667    4.80 | 8.917    2.40 | 13.17    4.80
        0.500    2.40 | 4.750    7.20 | 9.000    0.00 | 13.25    2.40
        0.583    0.00 | 4.833   12.00 | 9.083    4.80 | 13.33    2.40
        0.667    2.40 | 4.917   12.00 | 9.167    0.00 | 13.42    2.40
        0.750    0.00 | 5.000    7.20 | 9.250    2.40 | 13.50    7.20
        0.833    0.00 | 5.083    9.60 | 9.333    0.00 | 13.58    2.40
        0.917    0.00 | 5.167    9.60 | 9.417    2.40 | 13.67    0.00
        1.000    2.40 | 5.250    7.20 | 9.500    2.40 | 13.75    2.40
        1.083    0.00 | 5.333    9.60 | 9.583    2.40 | 13.83    0.00
        1.167    2.40 | 5.417    4.80 | 9.667    2.40 | 13.92    0.00
        1.250    0.00 | 5.500   12.00 | 9.750    4.80 | 14.00    0.00
        1.333    0.00 | 5.583    9.60 | 9.833    0.00 | 14.08    4.80
        1.417    0.00 | 5.667   12.00 | 9.917    2.40 | 14.17    0.00
        1.500    2.40 | 5.750   14.40 |10.000    0.00 | 14.25    2.40
        1.583    0.00 | 5.833   14.40 |10.083    4.80 | 14.33    0.00
        1.667    2.40 | 5.917    9.60 |10.167    4.80 | 14.42    0.00
        1.750    0.00 | 6.000    7.20 |10.250    4.80 | 14.50    2.40
```

```
        1.833    0.00 | 6.083   12.00 |10.333    0.00 | 14.58    0.00
        1.917    2.40 | 6.167    9.60 |10.417    4.80 | 14.67    0.00
        2.000    0.00 | 6.250    4.80 |10.500    0.00 | 14.75    2.40
        2.083    2.40 | 6.333    9.60 |10.583    2.40 | 14.83    0.00
        2.167    0.00 | 6.417    4.80 |10.667    2.40 | 14.92    0.00
        2.250    0.00 | 6.500    2.40 |10.750    2.40 | 15.00    2.40
        2.333    2.40 | 6.583    2.40 |10.833    4.80 | 15.08    0.00
        2.417    0.00 | 6.667    4.80 |10.917    2.40 | 15.17    0.00
        2.500    2.40 | 6.750    4.80 |11.000    2.40 | 15.25    0.00
        2.583    0.00 | 6.833    2.40 |11.083    0.00 | 15.33    2.40
        2.667    2.40 | 6.917    4.80 |11.167    4.80 | 15.42    0.00
        2.750    2.40 | 7.000    7.20 |11.250    2.40 | 15.50    0.00
        2.833    2.40 | 7.083    4.80 |11.333    2.40 | 15.58    4.80
        2.917    2.40 | 7.167    7.20 |11.417    0.00 | 15.67    0.00
        3.000    2.40 | 7.250    2.40 |11.500    4.80 | 15.75    0.00
        3.083    2.40 | 7.333    2.40 |11.583    0.00 | 15.83    0.00
        3.167    2.40 | 7.417    0.00 |11.667    4.80 | 15.92    0.00
        3.250    2.40 | 7.500    4.80 |11.750    4.80 | 16.00    2.40
        3.333    7.20 | 7.583    2.40 |11.833    2.40 | 16.08    2.40
        3.417    2.40 | 7.667    4.80 |11.917    2.40 | 16.17    0.00
        3.500    4.80 | 7.750    2.40 |12.000    0.00 | 16.25    0.00
        3.583    4.80 | 7.833    2.40 |12.083    4.80 | 16.33    0.00
        3.667    2.40 | 7.917    4.80 |12.167    2.40 | 16.42    0.00
        3.750    2.40 | 8.000    2.40 |12.250    2.40 | 16.50    0.00
        3.833    7.20 | 8.083    2.40 |12.333    4.80 | 16.58    0.00
        3.917    2.40 | 8.167    2.40 |12.417    2.40 | 16.67    0.00
        4.000    2.40 | 8.250    2.40 |12.500    0.00 | 16.75    0.00
        4.083    4.80 | 8.333    2.40 |12.583    2.40 | 16.83    0.00
        4.167    2.40 | 8.417    0.00 |12.667    4.80 | 16.92    4.80
        4.250    2.40 | 8.500    2.40 |12.750    2.40 |

  Unit Hyd Qpeak  (cms)=   2.311

  PEAK FLOW       (cms)=   0.050 (i)
  TIME TO PEAK    (hrs)=   6.167
  RUNOFF VOLUME    (mm)=   7.295
  TOTAL RAINFALL   (mm)=  47.800
  RUNOFF COEFFICIENT   =   0.153

  (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------

--------------------
| ADD HYD    (0901) |
|   1 +  2 =  3     |          AREA     QPEAK     TPEAK      R.V.
--------------------           (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0059):      12.10    0.050      6.17      7.30
        + ID2= 2 (0767):     606.73    1.432     20.33     28.47
          ===================================================
          ID = 3 (0901):     618.83    1.432     20.33     28.06

  NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------
--------------------
| CALIB              |
| NASHYD    (0060) |   Area     (ha)=  18.10   Curve Number   (CN)= 55.0
|ID= 1 DT= 5.0 min |   Ia       (mm)=   5.00   # of Linear Res.(N)= 3.00
--------------------   U.H. Tp(hrs)=   0.20

  Unit Hyd Qpeak  (cms)=   3.457

  PEAK FLOW       (cms)=   0.075 (i)
  TIME TO PEAK    (hrs)=   6.167
  RUNOFF VOLUME    (mm)=   7.295
  TOTAL RAINFALL   (mm)=  47.800
  RUNOFF COEFFICIENT   =   0.153

  (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------
```

```
------------------------------------------------------------------------------

--------------------
| ADD HYD   (0002) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK      R.V.
--------------------        (ha)    (cms)    (hrs)      (mm)
         ID1= 1 (0015):   100.60   0.211    15.83     31.69
       + ID2= 2 (0060):    18.10   0.075     6.17      7.30
         ==================================================
         ID = 3 (0002):   118.70   0.225    15.83     27.97

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0002) |
|  3 +  2 =  1     |        AREA    QPEAK    TPEAK      R.V.
--------------------        (ha)    (cms)    (hrs)      (mm)
         ID1= 3 (0002):   118.70   0.225    15.83     27.97
       + ID2= 2 (0901):   618.83   1.432    20.33     28.06
         ==================================================
         ID = 1 (0002):   737.53   1.594    20.33     28.04

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0010) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance       Elevation        Manning
                  0.00          152.00          0.1200
                  1.00          150.00          0.1200
                 26.00          149.12          0.1200
                 31.00          148.99       0.1200 /0.0500  Main Channel
                 32.00          148.23          0.0500       Main Channel
                 32.75          148.12          0.0500       Main Channel
                 33.50          148.19          0.0500       Main Channel
                 34.00          148.66       0.0500 /0.1200  Main Channel
                 39.00          149.10          0.1200
                 50.00          150.00          0.1200
                 51.00          152.00          0.1200

    <--------------------- TRAVEL TIME TABLE --------------------->
     DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
      (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
     0.18    148.30     .108E+03        0.1           0.28       29.30
     0.36    148.49     .282E+03        0.3           0.45       18.54
     0.54    148.66     .495E+03        0.6           0.57       14.70
     0.75    148.87     .903E+03        1.1           0.63       13.29
     0.96    149.08     .167E+04        2.0           0.61       13.64
     1.17    149.29     .332E+04        3.6           0.54       15.53
     1.37    149.50     .585E+04        5.9           0.50       16.51
     1.58    149.71     .928E+04        9.3           0.50       16.65
     1.79    149.92     .136E+05       13.9           0.51       16.33
     2.00    150.12     .186E+05       20.4           0.55       15.23
     2.21    150.33     .237E+05       28.6           0.60       13.84
     2.42    150.54     .289E+05       38.0           0.66       12.67
     2.62    150.75     .341E+05       48.5           0.71       11.70
     2.83    150.96     .393E+05       60.1           0.76       10.90
     3.04    151.17     .445E+05       72.6           0.82       10.22
     3.25    151.37     .497E+05       86.0           0.86        9.64
     3.46    151.58     .550E+05      100.3           0.91        9.13
     3.67    151.79     .603E+05      115.5           0.96        8.69
     3.87    152.00     .656E+05      131.5           1.00        8.31

                                 <---- hydrograph ---->   <-pipe / channel->
                          AREA    QPEAK   TPEAK   R.V.    MAX DEPTH  MAX VEL
                          (ha)    (cms)   (hrs)   (mm)       (m)      (m/s)
   INFLOW : ID= 2 (0002) 737.53    1.59   20.33  28.04      0.85       0.62
   OUTFLOW: ID= 1 (0010) 737.53    1.59   20.58  28.04      0.85       0.62
```

```
------------------------------------------------------------------------------

--------------------
| ADD HYD   (0009) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK      R.V.
--------------------        (ha)    (cms)    (hrs)      (mm)
         ID1= 1 (0004):    19.20   0.035    16.33     29.40
       + ID2= 2 (0010):   737.53   1.589    20.58     28.04
         ==================================================
         ID = 3 (0009):   756.73   1.623    20.58     28.07

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0006) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance       Elevation        Manning
                  0.00          152.00          0.1200
                  1.00          150.00          0.1200
                 26.00          149.12          0.1200
                 31.00          148.99       0.1200 /0.0500  Main Channel
                 32.00          148.23          0.0500       Main Channel
                 32.75          148.12          0.0500       Main Channel
                 33.50          148.19          0.0500       Main Channel
                 34.00          148.66       0.0500 /0.1200  Main Channel
                 39.00          149.10          0.1200
                 50.00          150.00          0.1200
                 51.00          152.00          0.1200

    <--------------------- TRAVEL TIME TABLE --------------------->
     DEPTH     ELEV       VOLUME     FLOW RATE     VELOCITY    TRAV.TIME
      (m)       (m)      (cu.m.)       (cms)         (m/s)       (min)
     0.18    148.30     .130E+03        0.1           0.28       35.16
     0.36    148.49     .339E+03        0.3           0.45       22.24
     0.54    148.66     .594E+03        0.6           0.57       17.64
     0.75    148.87     .108E+04        1.1           0.63       15.95
     0.96    149.08     .200E+04        2.0           0.61       16.37
     1.17    149.29     .398E+04        3.6           0.54       18.63
     1.37    149.50     .702E+04        5.9           0.50       19.81
     1.58    149.71     .111E+05        9.3           0.50       19.99
     1.79    149.92     .163E+05       13.9           0.51       19.60
     2.00    150.12     .223E+05       20.4           0.55       18.27
     2.21    150.33     .285E+05       28.6           0.60       16.60
     2.42    150.54     .347E+05       38.0           0.66       15.21
     2.62    150.75     .409E+05       48.5           0.71       14.05
     2.83    150.96     .471E+05       60.1           0.76       13.08
     3.04    151.17     .534E+05       72.6           0.82       12.26
     3.25    151.37     .597E+05       86.0           0.86       11.56
     3.46    151.58     .660E+05      100.3           0.91       10.96
     3.67    151.79     .723E+05      115.5           0.96       10.43
     3.87    152.00     .787E+05      131.5           1.00        9.97

                                 <---- hydrograph ---->   <-pipe / channel->
                          AREA    QPEAK   TPEAK   R.V.    MAX DEPTH  MAX VEL
                          (ha)    (cms)   (hrs)   (mm)       (m)      (m/s)
   INFLOW : ID= 2 (0009) 756.73    1.62   20.58  28.07      0.86       0.62
   OUTFLOW: ID= 1 (0006) 756.73    1.62   20.92  28.07      0.86       0.62

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0020) |   Area     (ha)=  97.70
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
--------------------
                           IMPERVIOUS    PERVIOUS (i)
   Surface Area     (ha)=     51.19         46.51
```

```
   Dep. Storage     (mm)=      0.10          5.00
   Average Slope     (%)=      0.99          0.99
   Length            (m)=    807.05        807.00
   Mannings n          =      0.013         0.250

   Max.Eff.Inten.(mm/hr)=     12.60          1.77
             over (min)       20.00        290.00
   Storage Coeff.  (min)=     20.54 (ii)   285.98 (ii)
   Unit Hyd. Tpeak (min)=     20.00        290.00
   Unit Hyd. peak  (cms)=      0.06          0.00
                                                        *TOTALS*
   PEAK FLOW       (cms)=      1.54          0.09         1.546 (iii)
   TIME TO PEAK    (hrs)=      6.08         15.92          6.08
   RUNOFF VOLUME    (mm)=     47.70          9.32         29.43
   TOTAL RAINFALL   (mm)=     47.80         47.80         47.80
   RUNOFF COEFFICIENT   =      1.00          0.20          0.62

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  62.3    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| RESERVOIR (0016) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)    (ha.m.)   |   (cms)    (ha.m.)
                         0.0000     0.0000   |   0.1430     1.2448
                         0.0380     0.0189   |   0.1520     1.4335
                         0.0530     0.1132   |   0.1560     1.5278
                         0.0640     0.2075   |   0.1650     1.7164
                         0.0830     0.3961   |   0.1770     2.0936
                         0.1040     0.5847   |   0.1800     2.2985
                         0.1100     0.6790   |   2.6063     2.4710
                         0.1220     0.8676   |   7.0425     2.6435
                         0.1280     0.9619   |  12.7872     2.8160
                         0.1380     1.1505   |  19.5901     2.9885

                           AREA     QPEAK      TPEAK       R.V.
                           (ha)     (cms)      (hrs)       (mm)
   INFLOW : ID= 2 (0020)   97.700    1.546       6.08       29.43
   OUTFLOW: ID= 1 (0016)   97.700    0.172      16.50       29.43

              PEAK   FLOW   REDUCTION [Qout/Qin](%)= 11.11
              TIME SHIFT OF PEAK FLOW         (min)=625.00
              MAXIMUM  STORAGE   USED       (ha.m.)=  1.9289

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0014) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK      R.V.
--------------------        (ha)    (cms)    (hrs)      (mm)
         ID1= 1 (0006):   756.73   1.617    20.92     28.07
       + ID2= 2 (0016):    97.70   0.172    16.50     29.43
         ==================================================
         ID = 3 (0014):   854.43   1.785    20.92     28.22

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (9204) |   Area     (ha)=  18.90
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  50.00
--------------------
                           IMPERVIOUS    PERVIOUS (i)
   Surface Area     (ha)=      9.45          9.45
   Dep. Storage     (mm)=      0.10          5.00
```

```
   Average Slope     (%)=      0.50          0.97
   Length            (m)=    354.96       1500.00
   Mannings n          =      0.013         0.250

   Max.Eff.Inten.(mm/hr)=     13.60          3.09
             over (min)       15.00        325.00
   Storage Coeff.  (min)=     14.94 (ii)   324.93 (ii)
   Unit Hyd. Tpeak (min)=     15.00        325.00
   Unit Hyd. peak  (cms)=      0.08          0.00
                                                        *TOTALS*
   PEAK FLOW       (cms)=      0.30          0.03         0.304 (iii)
   TIME TO PEAK    (hrs)=      5.92         16.17          5.92
   RUNOFF VOLUME    (mm)=     47.70         15.27         31.48
   TOTAL RAINFALL   (mm)=     47.80         47.80         47.80
   RUNOFF COEFFICIENT   =      1.00          0.32          0.66

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (4243) |   Area     (ha)=   6.10
|ID= 1 DT= 5.0 min |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
--------------------
                           IMPERVIOUS    PERVIOUS (i)
   Surface Area     (ha)=      3.20          2.90
   Dep. Storage     (mm)=      0.10          5.00
   Average Slope     (%)=      0.50          0.97
   Length            (m)=    201.66       1500.00
   Mannings n          =      0.013         0.250

   Max.Eff.Inten.(mm/hr)=     14.40          3.09
             over (min)       10.00        325.00
   Storage Coeff.  (min)=     10.40 (ii)   320.40 (ii)
   Unit Hyd. Tpeak (min)=     10.00        325.00
   Unit Hyd. peak  (cms)=      0.11          0.00
                                                        *TOTALS*
   PEAK FLOW       (cms)=      0.11          0.01         0.110 (iii)
   TIME TO PEAK    (hrs)=      5.92         16.17          5.92
   RUNOFF VOLUME    (mm)=     47.70         15.27         32.23
   TOTAL RAINFALL   (mm)=     47.80         47.80         47.80
   RUNOFF COEFFICIENT   =      1.00          0.32          0.67

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =  76.7    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0072) |
|  1 +  2 =  3     |        AREA    QPEAK    TPEAK      R.V.
--------------------        (ha)    (cms)    (hrs)      (mm)
         ID1= 1 (4243):     6.10   0.110     5.92     32.23
       + ID2= 2 (9204):    18.90   0.304     5.92     31.48
         ==================================================
         ID = 3 (0072):    25.00   0.414     5.92     31.66

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

------------------------------------------------------------------------------

--------------------
| ADD HYD   (0071) |
```

```
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (0014):     854.43   1.785    20.92     28.22
       + ID2= 2 (0072):      25.00   0.414     5.92     31.66
         =================================================
         ID = 3 (0071):     879.43   1.809    20.83     28.32

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0064) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance       Elevation       Manning
                   0.00          152.00        0.1200
                   1.00          150.00        0.1200
                  26.00          149.12        0.1200
                  31.00          148.99    0.1200 /0.0500  Main Channel
                  32.00          148.23        0.0500      Main Channel
                  32.75          148.12        0.0500      Main Channel
                  33.50          148.19        0.0500      Main Channel
                  34.00          148.66    0.0500 /0.1200  Main Channel
                  39.00          149.10        0.1200
                  50.00          150.00        0.1200
                  51.00          152.00        0.1200

         <--------------------- TRAVEL TIME TABLE --------------------->
          DEPTH     ELEV       VOLUME     FLOW RATE    VELOCITY   TRAV.TIME
           (m)       (m)      (cu.m.)      (cms)        (m/s)       (min)
          0.18    148.30     .141E+03       0.1          0.28       38.09
          0.36    148.49     .367E+03       0.3          0.45       24.10
          0.54    148.66     .643E+03       0.6          0.57       19.11
          0.75    148.87     .117E+04       1.1          0.63       17.28
          0.96    149.08     .217E+04       2.0          0.61       17.73
          1.17    149.29     .431E+04       3.6          0.54       20.19
          1.37    149.50     .761E+04       5.9          0.50       21.47
          1.58    149.71     .121E+05       9.3          0.50       21.65
          1.79    149.92     .177E+05      13.9          0.51       21.23
          2.00    150.12     .242E+05      20.4          0.55       19.79
          2.21    150.33     .309E+05      28.6          0.60       17.99
          2.42    150.54     .376E+05      38.0          0.66       16.47
          2.62    150.75     .443E+05      48.5          0.71       15.22
          2.83    150.96     .511E+05      60.1          0.76       14.17
          3.04    151.17     .578E+05      72.6          0.82       13.28
          3.25    151.37     .646E+05      86.0          0.86       12.53
          3.46    151.58     .715E+05     100.3          0.91       11.87
          3.67    151.79     .784E+05     115.5          0.96       11.30
          3.87    152.00     .852E+05     131.5          1.00       10.80

                                        <---- hydrograph ---->   <-pipe / channel->
                                 AREA    QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                                 (ha)    (cms)   (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0071)     879.43     1.81   20.83  28.32      0.90       0.61
     OUTFLOW: ID= 1 (0064)     879.43     1.80   21.17  28.32      0.90       0.62

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (4223) |   Area    (ha)=  47.80
|ID= 1 DT= 5.0 min |   Total Imp(%)=  50.00   Dir. Conn.(%)=  50.00
--------------------
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=       23.90         23.90
     Dep. Storage      (mm)=        0.10          5.00
     Average Slope      (%)=        0.50          0.97
     Length             (m)=      564.51       1143.00
     Mannings n           =        0.013         0.250

     Max.Eff.Inten.(mm/hr)=       12.60          3.09
```

```
                over (min)       20.00         285.00
     Storage Coeff.  (min)=      20.35 (ii)    283.69 (ii)
     Unit Hyd. Tpeak (min)=      20.00         285.00
     Unit Hyd. peak  (cms)=       0.06           0.00
                                                             *TOTALS*
     PEAK FLOW       (cms)=       0.72           0.07          0.725 (iii)
     TIME TO PEAK    (hrs)=       6.08          15.50          6.08
     RUNOFF VOLUME    (mm)=      47.70          15.27         31.48
     TOTAL RAINFALL   (mm)=      47.80          47.80         47.80
     RUNOFF COEFFICIENT   =       1.00           0.32          0.66

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN* =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD  (0204) |   Area    (ha)= 142.20
|ID= 1 DT= 5.0 min |   Total Imp(%)=  40.00   Dir. Conn.(%)=  40.00
--------------------
                                IMPERVIOUS    PERVIOUS (i)
     Surface Area      (ha)=       56.88         85.32
     Dep. Storage      (mm)=        0.10          5.00
     Average Slope      (%)=        0.50          0.97
     Length             (m)=      973.65       1143.00
     Mannings n           =        0.013         0.250

     Max.Eff.Inten.(mm/hr)=       12.00          3.09
                over (min)       30.00         295.00
     Storage Coeff.  (min)=      28.78 (ii)    292.12 (ii)
     Unit Hyd. Tpeak (min)=      30.00         295.00
     Unit Hyd. peak  (cms)=       0.04           0.00
                                                             *TOTALS*
     PEAK FLOW       (cms)=       1.60           0.26          1.619 (iii)
     TIME TO PEAK    (hrs)=       6.17          15.67          6.17
     RUNOFF VOLUME    (mm)=      47.70          15.27         28.24
     TOTAL RAINFALL   (mm)=      47.80          47.80         47.80
     RUNOFF COEFFICIENT   =       1.00           0.32          0.59

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
           CN* =  76.7    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0035) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (0064):     879.43   1.802    21.17     28.32
       + ID2= 2 (0204):     142.20   1.619     6.17     28.24
         =================================================
         ID = 3 (0035):    1021.63   2.351     6.25     28.31

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0035) |
|   3 +  2 =  1    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 3 (0035):    1021.63   2.351     6.25     28.31
       + ID2= 2 (4223):      47.80   0.725     6.08     31.48
         =================================================
```

```
         ID = 1 (0035):    1069.43   3.042     6.17     28.45

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0069) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance       Elevation       Manning
                   0.00          152.00        0.1200
                   1.00          150.00        0.1200
                  26.00          149.12        0.1200
                  31.00          148.99    0.1200 /0.0500  Main Channel
                  32.00          148.23        0.0500      Main Channel
                  32.75          148.12        0.0500      Main Channel
                  33.50          148.19        0.0500      Main Channel
                  34.00          148.66    0.0500 /0.1200  Main Channel
                  39.00          149.10        0.1200
                  50.00          150.00        0.1200
                  51.00          152.00        0.1200

         <--------------------- TRAVEL TIME TABLE --------------------->
          DEPTH     ELEV       VOLUME     FLOW RATE    VELOCITY   TRAV.TIME
           (m)       (m)      (cu.m.)      (cms)        (m/s)       (min)
          0.18    148.30     .217E+03       0.1          0.28       58.60
          0.36    148.49     .565E+03       0.3          0.45       37.07
          0.54    148.66     .989E+03       0.6          0.57       29.39
          0.75    148.87     .181E+04       1.1          0.63       26.59
          0.96    149.08     .334E+04       2.0          0.61       27.28
          1.17    149.29     .663E+04       3.6          0.54       31.05
          1.37    149.50     .117E+05       5.9          0.50       33.02
          1.58    149.71     .186E+05       9.3          0.50       33.31
          1.79    149.92     .272E+05      13.9          0.51       32.67
          2.00    150.12     .372E+05      20.4          0.55       30.45
          2.21    150.33     .475E+05      28.6          0.60       27.67
          2.42    150.54     .578E+05      38.0          0.66       25.34
          2.62    150.75     .682E+05      48.5          0.71       23.41
          2.83    150.96     .785E+05      60.1          0.76       21.80
          3.04    151.17     .890E+05      72.6          0.82       20.43
          3.25    151.37     .995E+05      86.0          0.86       19.27
          3.46    151.58     .110E+06     100.3          0.91       18.27
          3.67    151.79     .121E+06     115.5          0.96       17.39
          3.87    152.00     .131E+06     131.5          1.00       16.62

                                        <---- hydrograph ---->   <-pipe / channel->
                                 AREA    QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
                                 (ha)    (cms)   (hrs)   (mm)      (m)       (m/s)
     INFLOW : ID= 2 (0035)    1069.43     3.04    6.17  28.45      1.09       0.56
     OUTFLOW: ID= 1 (0069)    1069.43     2.69    6.58  28.44      1.04       0.58

--------------------------------------------------------------------------------
--------------------
| RESERVOIR (0053) |
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |      OUTFLOW    STORAGE  |  OUTFLOW    STORAGE
--------------------       (cms)     (ha.m.)  |   (cms)     (ha.m.)
                          0.0000     0.0000   |   1.1000     1.6000
                          0.3000     0.0050   |   1.1800     2.3000
                          0.6000     0.0250   |   1.2270     2.7000
                          0.7500     0.2000   |   5.2800     3.0500
                          0.8500     0.5000   |  27.4400     3.9440
                          0.9500     1.0000   |  59.0000     4.8000

                                 AREA      QPEAK       TPEAK        R.V.
                                 (ha)      (cms)       (hrs)        (mm)
     INFLOW : ID= 2 (0069)    1069.427     2.689        6.58       28.44
     OUTFLOW: ID= 1 (0053)    1069.427     2.429       14.17       28.44

               PEAK   FLOW   REDUCTION [Qout/Qin](%)= 90.32
```

```
                  TIME SHIFT OF PEAK FLOW          (min)=455.00
                  MAXIMUM  STORAGE   USED        (ha.m.)=  2.8037

--------------------------------------------------------------------------------
--------------------
| ADD HYD   (0067) |
|   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
--------------------          (ha)    (cms)    (hrs)     (mm)
*** W A R N I N G :  HYDROGRAPH 0205 <ID= 2> IS DRY.
*** W A R N I N G :  HYDROGRAPH 0003 = HYDROGRAPH 0001
*** W A R N I N G :  HYDROGRAPH 0003 = HYDROGRAPH 0001
         ID1= 1 (0053):    1069.43   2.429    14.17     28.44
       + ID2= 2 (0205):       0.00   0.000     0.00    NaN
         =================================================
         ID = 3 (0067):    1069.43   2.429    14.17     28.44

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
--------------------
| ROUTE CHN (0025) |
| IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
--------------------
               <------ DATA FOR SECTION (   1.1) ------>
               Distance       Elevation       Manning
                   0.00          137.50        0.1200
                   1.00          135.50        0.1200
                  26.00          134.52        0.1200
                  29.80          134.54        0.1200
                  30.00          134.09        0.1200
                  30.50          134.03        0.1200
                  31.00          134.01    0.1200 /0.0500  Main Channel
                  31.50          134.01        0.0500      Main Channel
                  32.00          133.96        0.0500      Main Channel
                  32.50          133.95        0.0500      Main Channel
                  33.00          133.99        0.0500      Main Channel
                  33.50          134.06        0.0500      Main Channel
                  34.00          134.07    0.0500 /0.1200  Main Channel
                  35.00          134.06        0.1200
                  35.50          134.07        0.1200
                  36.00          134.30        0.1200
                  36.50          134.41        0.1200
                  37.00          134.44        0.1200
                  50.00          135.50        0.1200
                  51.00          137.50        0.1200

         <--------------------- TRAVEL TIME TABLE --------------------->
          DEPTH     ELEV       VOLUME     FLOW RATE    VELOCITY   TRAV.TIME
           (m)       (m)      (cu.m.)      (cms)        (m/s)       (min)
          0.06    134.01     .475E+02       0.0          0.19       65.59
          0.24    134.20     .717E+03       0.5          0.49       25.30
          0.43    134.38     .156E+04       1.5          0.73       17.21
          0.61    134.56     .270E+04       2.9          0.80       15.70
          0.79    134.75     .503E+04       5.2          0.78       15.99
          0.98    134.93     .833E+04       8.8          0.79       15.83
          1.16    135.11     .126E+05      13.6          0.81       15.38
          1.34    135.30     .178E+05      20.0          0.84       14.80
          1.53    135.48     .240E+05      28.2          0.88       14.18
          1.71    135.66     .307E+05      39.5          0.97       12.95
          1.90    135.85     .375E+05      52.8          1.06       11.84
          2.08    136.03     .443E+05      67.6          1.14       10.93
          2.26    136.22     .511E+05      83.8          1.23       10.16
          2.45    136.40     .580E+05     101.5          1.31        9.52
          2.63    136.58     .649E+05     120.5          1.39        8.97
          2.81    136.77     .718E+05     140.7          1.47        8.50
          3.00    136.95     .787E+05     162.2          1.55        8.09
          3.18    137.13     .857E+05     184.9          1.62        7.72
          3.36    137.32     .926E+05     208.7          1.69        7.40

                                        <---- hydrograph ---->   <-pipe / channel->
                                 AREA    QPEAK   TPEAK   R.V.   MAX DEPTH   MAX VEL
```

```
                         (ha)     (cms)    (hrs)    (mm)        (m)        (m/s)
    INFLOW : ID= 2 (0067) 1069.43      2.43    14.17   28.44       0.55        0.77
    OUTFLOW: ID= 1 (0025) 1069.43      2.42    14.33   28.44       0.55        0.77

 ---------------------------------------------------------------------------------
 --------------------
 | CALIB             |
 | STANDHYD  (0610)  |   Area     (ha)= 431.20
 |ID= 1 DT= 5.0 min  |   Total Imp(%)=  52.40   Dir. Conn.(%)=  52.40
 --------------------
                          IMPERVIOUS    PERVIOUS (i)
    Surface Area     (ha)=    225.95       205.25
    Dep. Storage     (mm)=      0.10         5.00
    Average Slope     (%)=      0.99         0.99
    Length            (m)=   1695.48      2298.00
    Mannings n          =     0.013        0.250

    Max.Eff.Inten.(mm/hr)=     12.00         1.77
               over (min)     30.00       535.00
    Storage Coeff.  (min)=     32.70 (ii)  530.03 (ii)
    Unit Hyd. Tpeak (min)=     30.00       535.00
    Unit Hyd. peak  (cms)=      0.04         0.00
                                                           *TOTALS*
    PEAK FLOW       (cms)=      6.21         0.28           6.214 (iii)
    TIME TO PEAK    (hrs)=      6.25        19.25            6.25
    RUNOFF VOLUME    (mm)=     47.70         9.32           29.43
    TOTAL RAINFALL   (mm)=     47.80        47.80           47.80
    RUNOFF COEFFICIENT   =      1.00         0.19            0.62

     (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
          CN*  =   62.3    Ia = Dep. Storage  (Above)
    (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
         THAN THE STORAGE COEFFICIENT.
   (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

 ---------------------------------------------------------------------------------

 --------------------
 | ADD HYD   (0024) |
 |  1 +  2 =  3     |         AREA     QPEAK     TPEAK      R.V.
 --------------------         (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0025):   1069.43    2.420     14.33     28.44
        + ID2= 2 (0610):    431.20    6.214      6.25     29.43
          ===============================================
          ID = 3 (0024):   1500.63    7.048      6.25     28.72

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ---------------------------------------------------------------------------------
 --------------------
 | ROUTE CHN (0027) |
 | IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
 --------------------
              <------ DATA FOR SECTION (   1.1) ------>
              Distance      Elevation        Manning
                  0.00         152.00         0.1200
                  1.00         150.00         0.1200
                 26.00         149.12         0.1200
                 31.00         148.99     0.1200 /0.0500  Main Channel
                 32.00         148.23         0.0500      Main Channel
                 32.75         148.12         0.0500      Main Channel
                 33.50         148.19         0.0500      Main Channel
                 34.00         148.66     0.0500 /0.1200  Main Channel
                 39.00         149.10         0.1200
                 50.00         150.00         0.1200
                 51.00         152.00         0.1200

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV      VOLUME    FLOW RATE    VELOCITY   TRAV.TIME
         (m)       (m)     (cu.m.)      (cms)        (m/s)       (min)
```

```
        0.18    148.30    .362E+03        0.1           0.28       97.68
        0.36    148.49    .942E+03        0.3           0.45       61.80
        0.54    148.66    .165E+04        0.6           0.57       49.00
        0.75    148.87    .301E+04        1.1           0.63       44.32
        0.96    149.08    .556E+04        2.0           0.61       45.48
        1.17    149.29    .111E+05        3.6           0.54       51.77
        1.37    149.50    .195E+05        5.9           0.50       55.05
        1.58    149.71    .309E+05        9.3           0.50       55.53
        1.79    149.92    .453E+05       13.9           0.51       54.45
        2.00    150.12    .621E+05       20.4           0.55       50.77
        2.21    150.33    .792E+05       28.6           0.60       46.13
        2.42    150.54    .964E+05       38.0           0.66       42.25
        2.62    150.75    .114E+06       48.5           0.71       39.02
        2.83    150.96    .131E+06       60.1           0.76       36.33
        3.04    151.17    .148E+06       72.6           0.82       34.06
        3.25    151.37    .166E+06       86.0           0.86       32.12
        3.46    151.58    .183E+06      100.3           0.91       30.45
        3.67    151.79    .201E+06      115.5           0.96       28.99
        3.87    152.00    .219E+06      131.5           1.00       27.70

                                   <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
    INFLOW : ID= 2 (0024) 1500.63     7.05    6.25  28.72      1.44      0.50
    OUTFLOW: ID= 1 (0027) 1500.63     5.55    6.83  28.72      1.34      0.51

 ---------------------------------------------------------------------------------

 --------------------
 | ADD HYD   (0903) |
 |  1 +  2 =  3     |         AREA     QPEAK     TPEAK      R.V.
 --------------------         (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0027):   1500.63    5.555      6.83     28.72
        + ID2= 2 (0895):   1199.04    2.360     20.67     20.59
          ===============================================
          ID = 3 (0903):   2699.67    6.146      7.00     25.11

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ---------------------------------------------------------------------------------
 --------------------
 | ROUTE CHN (0768) |
 | IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
 --------------------
              <------ DATA FOR SECTION (  68.0) ------>
              Distance      Elevation        Manning
                  0.00         105.00         0.2100
                  8.00         100.00         0.2100
                 14.00          97.00         0.2100
                 32.00          96.00         0.2100
                 75.00          96.00         0.2100
                120.00          95.50         0.2100
                134.00          95.50         0.2100
                149.00          93.00         0.2100
                189.00          92.00         0.2100
                190.00          91.00         0.2100
                201.00          90.00     0.2100 /0.0700  Main Channel
                203.00          89.60         0.0700      Main Channel
                215.00          89.60         0.0700      Main Channel
                222.00          91.00     0.0700 /0.1950  Main Channel
                230.00          90.00         0.1950
                232.00          92.10         0.1950
                234.00          92.00         0.1950
                244.00          92.00         0.1950
                252.00          95.00         0.1950
                284.00         100.00         0.1950

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV      VOLUME    FLOW RATE    VELOCITY   TRAV.TIME
         (m)       (m)     (cu.m.)      (cms)        (m/s)       (min)
        0.40     90.00    .390E+05        2.5           0.45      259.68
```

```
        0.93     90.53    .122E+06       11.7           0.67      174.20
        1.45     91.05    .253E+06       27.8           0.77      151.17
        1.98     91.58    .405E+06       53.6           0.92      125.83
        2.51     92.11    .571E+06       83.1           1.01      114.49
        3.03     92.63    .830E+06      122.5           1.03      112.94
        3.56     93.16    .117E+07      174.4           1.04      111.72
        4.08     93.68    .154E+07      239.1           1.08      107.41
        4.61     94.21    .193E+07      313.6           1.13      102.53
        5.14     94.74    .233E+07      397.6           1.19       97.84
        5.66     95.26    .276E+07      489.8           1.24       93.81
        6.19     95.79    .326E+07      560.9           1.20       96.73
        6.72     96.32    .402E+07      655.8           1.14      102.07
        7.24     96.84    .490E+07      803.3           1.14      101.64
        7.77     97.37    .582E+07      973.7           1.17       99.63
        8.29     97.89    .676E+07     1164.2           1.20       96.78
        8.82     98.42    .772E+07     1372.3           1.24       93.71
        9.35     98.95    .869E+07     1597.5           1.28       90.63
        9.87     99.47    .968E+07     1839.4           1.32       87.66

                                   <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
    INFLOW : ID= 2 (0903) 2699.67     6.15    7.00  25.11      0.61      0.51
    OUTFLOW: ID= 1 (0768) 2699.67     5.02   16.67  25.09      0.54      0.49

 ---------------------------------------------------------------------------------

 --------------------
 | ADD HYD   (0904) |
 |  1 +  2 =  3     |         AREA     QPEAK     TPEAK      R.V.
 --------------------         (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0600):    822.24    4.265      6.33     18.07
        + ID2= 2 (0768):   2699.67    5.016     16.67     25.09
          ===============================================
          ID = 3 (0904):   3521.91    6.222     13.92     23.45

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ---------------------------------------------------------------------------------
 --------------------
 | CALIB              |
 | WILHYD    (0500) |   Area     (ha)= 261.11   Curve Number   (CN)= 76.5
 |ID= 1 DT= 5.0 min |   Ia       (mm)=   5.00   Recession const.(K)= 1.06
 --------------------   U.H. Tp(hrs)=   0.89

    U.H. peak       (cms)=   8.97

    PEAK FLOW       (cms)=   1.29 (i)
    TIME TO PEAK    (hrs)=   7.00
    RUNOFF VOLUME    (mm)=  15.12
    TOTAL RAINFALL   (mm)=  47.80
    RUNOFF COEFFICIENT   =   0.32

    (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
 ---------------------------------------------------------------------------------

 --------------------
 | ADD HYD   (0956) |
 |  1 +  2 =  3     |         AREA     QPEAK     TPEAK      R.V.
 --------------------         (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0500):    261.11    1.289      7.00     15.12
        + ID2= 2 (0904):   3521.91    6.222     13.92     23.45
          ===============================================
          ID = 3 (0956):   3783.02    7.188      6.58     22.87

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ---------------------------------------------------------------------------------
 --------------------
 | ROUTE CHN (7102) |
 | IN= 2---> OUT= 1 |     Routing time step (min)'=  5.00
```

```
 --------------------
              <------ DATA FOR SECTION ( 102.0) ------>
              Distance      Elevation        Manning
                  0.00          87.00         0.1800
                  2.00          86.00         0.1800
                  8.00          85.00         0.1800
                 19.00          80.00     0.1800 /0.0600  Main Channel
                 30.00          75.30         0.0600      Main Channel
                150.00          75.30         0.0600      Main Channel
                200.00          75.60         0.0600      Main Channel
                286.00          75.30         0.0600      Main Channel
                312.00          75.30         0.0600      Main Channel
                318.00          76.00         0.0600      Main Channel
                342.00          77.00     0.0600 /0.1800  Main Channel
                350.00          80.00         0.1800
                367.00          85.00         0.1800

       <-------------------- TRAVEL TIME TABLE --------------------->
        DEPTH     ELEV      VOLUME    FLOW RATE    VELOCITY   TRAV.TIME
         (m)       (m)     (cu.m.)      (cms)        (m/s)       (min)
        0.43     75.73    .259E+06        9.1           0.09      473.60
        0.85     76.15    .576E+06       34.0           0.15      282.46
        1.28     76.58    .904E+06       70.2           0.20      214.51
        1.70     77.00    .124E+07      116.7           0.24      177.62
        2.23     77.53    .168E+07      191.9           0.29      145.91
        2.77     78.07    .212E+07      281.4           0.34      125.55
        3.30     78.60    .256E+07      384.3           0.39      111.19
        3.83     79.13    .301E+07      499.6           0.43      100.44
        4.37     79.67    .346E+07      627.0           0.47       92.03
        4.90     80.20    .392E+07      766.5           0.50       85.16
        5.43     80.73    .438E+07      918.5           0.54       79.39
        5.97     81.27    .484E+07     1081.5           0.58       74.56
        6.50     81.80    .531E+07     1255.1           0.61       70.44
        7.03     82.33    .578E+07     1439.1           0.64       66.89
        7.57     82.87    .625E+07     1633.3           0.67       63.79
        8.10     83.40    .673E+07     1837.3           0.70       61.05
        8.63     83.93    .721E+07     2051.1           0.73       58.62
        9.17     84.47    .770E+07     2274.4           0.76       56.43
        9.70     85.00    .819E+07     2507.1           0.79       54.46

                                   <---- hydrograph ---->   <-pipe / channel->
                            AREA    QPEAK   TPEAK   R.V.   MAX DEPTH  MAX VEL
                            (ha)    (cms)   (hrs)   (mm)      (m)      (m/s)
    INFLOW : ID= 2 (0956) 3783.02     7.19    6.58  22.87      0.34      0.09
    OUTFLOW: ID= 1 (7102) 3783.02     5.29   22.50  22.85      0.24      0.09

 ---------------------------------------------------------------------------------

 --------------------
 | ADD HYD   (0957) |
 |  1 +  2 =  3     |         AREA     QPEAK     TPEAK      R.V.
 --------------------         (ha)     (cms)     (hrs)      (mm)
          ID1= 1 (0700):    307.23    1.379      6.50     23.28
        + ID2= 2 (7102):   3783.02    5.294     22.50     22.85
          ===============================================
          ID = 3 (0957):   4090.25    5.573     17.42     22.88

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

 ---------------------------------------------------------------------------------

  FINISH
 ==========================================================================================
 ================
```

Appendix B

Photos of Morningside Creek

Photos of Online Pond L1, Looking north from Morningview Trail. Top: May 11, 2017, low flow. Bottom: May 5, 2017, high water level, water flowing through overflow structure.

Photos taken of the study reach at MCRK 20, facing downstream. Top: May 20, 2015, low flow. Bottom: May 5, 2017, during a period of high discharge.

Photo of study reach. Taken May 20, 2015, during low flow.