

Flow and Sediment Routing in Urban Channels with Flashy Hydrographs

by

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Author's Declaration

This thesis consists of material all of which I authored or co-authored: see Statement of Contributions included in the 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

Statement of Contributions

For Chapter 2 the data was collected by previous students in the lab. The program to clean, analyze, and plot the data was written by myself. Chapters were written by myself and edited by Bruce MacVicar. Figures of Chapter 2 were edited by Bruce MacVicar. Figures of Chapter 3 were created and edited by myself. Some of the figures of Chapter 4 were created and edited by Bruce MacVicar. The rest of thesis were created by myself and edited by Bruce MacVicar.

Abstract

Urbanization has a major impact on flow and erosion in open channels. By replacing rural land surfaces with impervious surfaces, urbanization causes a decrease in infiltration and an increase in surface runoff and faster routing to the channels. For two identical rain events on both rural and urban catchments, more water enters urban channels in a shorter period of time, which means that flow hydrographs [graphs that show changes of flow discharges over time] in urban channels are characterized by shorter times to peak, higher peak flows, and rapid changes in flow depth and discharge. However, while such rapid changes are known, they are not typically considered for the purposes of floodplain delineation and risk, which usually use steady flow assumptions. There are limited studies that focus on characterizing unsteady flows in urban catchments and the degree to which unsteadiness affects the channel hydraulics, i.e. the balance of forces on the water, bed and banks of a river that determine the flooding, sediment transport, and erosion during a flood. Numerous damaging floods occurring annually around the world shows that floodplain maps have not been delineated accurately. Urban floods also cause severe damage to nearby infrastructure in rivers and floodplains due to massive fluxes of sediment transport occurring under unsteady urban flows. Although it is known that sediment transport rate is high in urban channels and that the hydrographs are flashy, there is little research on sediment transport in urban channels. Due to the limitations and complexity of bedload transport, most studies investigate it under steady flows. Therefore, sediment transport rate under such unsteady flows requires a comprehensive further research.

The first objective of this thesis is to investigate the level of unsteadiness of flows in urban catchments versus rural catchments. For this purpose, we selected two urban catchments one with and one without Storm Water Management (SWM) facilities. We also selected one rural catchment with similar shape, watershed slope, and drainage area. All three watersheds are located near the City of Toronto not far from each other. Pressure gauges were installed to monitor flow depths with frequency of 2 minutes during summer months. This data was used to route flows downstream using the Saint Venant equations. By looking at both hydrologic and hydraulic parameters, it can be concluded that adding SWM facilities to urban creeks dampens hydrologic parameters, but not hydraulic parameters. In addition, it can be seen that the flows observed in the urban catchment have higher level of unsteadiness compared to that in the rural catchment.

The second objective of the thesis is to assess the accuracy of using a system of two gauges that measure the water surface elevation to predict, or ‘route’, the water discharge through an urban creek characterized by floods with high flood magnitudes and rapid rising limbs. We installed pressure gauges with 15-second frequency in a watershed with a high percentage of impervious land cover and used four different methods of flow routing to calculate discharge. The results were compared with direct discharge measurements to evaluate all the routing methods. The results of this chapter showed that lower frequency data in the urban channels cannot represent the high degree of unsteadiness of flows and that in comparison with a standard approach that uses one gauge measuring every 15 min, a two-gauge system measuring water surface elevation every 2-5 minutes would reduce the associated error by 20-30%. One-gauge monitoring systems remain the standard for developing monitoring stations for the purpose of establishing flow rating curves in most channels, but the results show that a two-gauge system would reduce error for the studied rural and urban rivers.

The final objective of the thesis is to investigate sediment transport under the types of unsteady flows that are observed in urban areas. Physical experiments were run in a scale model of an urban creek investigated in Chapter 2 and Chapter 3 of the current thesis. The results from experiments with a set of triangular hydrographs were compared to assess the impact of the rising limb duration on the morphology and sediment transport in the channel. The hydrographs all had the same duration and peak magnitude. Mixed sediment sizes were used for this project based on the actual sediments observed in the creek. The bed was covered evenly over the entire flume with a mixture of sand and gravel and no additional sediment inflows were added during the experiments. Three large sediment size ranges were each colored with a different fluorescent color. The movement of sediments was captured by a high-resolution camera under black lights, and tracked using an available object identification and motion detection algorithm. We show that the shape of the hydrograph is only of secondary importance as there is more variability between experiments with the same hydrograph than those with different hydrographs. This result indicates that the complexity of the bed sediment interactions was likely controlling export from the flume and that there is a random or chaotic component of the sediment transport. Analysis by particle size showed that there were some clear trends in sediment export and that the coarse sediment tended to be over represented in sediment output during the high peaks. Where the rising limb was rapid, the coarse sediment was even more over-represented in the output, which makes it surprising that rising limb rate was not the main driver in the difference of overall sediment export. These results support the need for additional research on the interaction between the flow,

transport and morphology. In particular the tracking techniques should better allow the spatial details and differences between the experiments to be extracted.

Overall, the thesis concludes that unsteady flows are observed both in urban and rural channels, but are more frequent in urban channels. There will be lower degree of errors for discharge estimation in unsteady floods if the two-gauge routing method presented in this thesis is used. Sediment transport under such unsteady flows require further investigation with the tracking method presented in this thesis. Therefore, additional laboratory and field research is required for the purpose of better understanding the characteristics and effects of such unsteady flows on the surrounding environment.

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Dedication

To the memory of my mother, Elaheh Jamalifard, who passed away before she could see me pursue my academic journey. Her unwavering love, support, and encouragement have been the driving force behind my pursuit of knowledge. I dedicate this thesis to her and will always be grateful for the impact she had on my life.

To the loving memory of my grandmother, Simin Asadsangabi, whose constant love and support helped me pursue my academic goals. Her words of wisdom and kindness have always helped me to do well in my life. Although she passed away two years ago, her memories continue to motivate me to achieve my dreams. I dedicate this thesis to her with love and gratitude.

To my brother Arian Montakhab and my father Mohammadreza Montakhab, who have both been a constant source of inspiration and support throughout my academic journey. Their unwavering encouragement, guidance, and love have been instrumental in shaping who I am today. My brother's accomplishments in his own field have inspired me to pursue my own academic goals, while my father's hard work and dedication have taught me the importance of perseverance and determination. I am grateful to have them both in my life and dedicate this thesis to them in recognition of their significant contributions to my success.

To my dearest friend Saeedeh Naziri, whose constant encouragement and understanding have been the foundation of my academic success. Her kindness, wisdom, and friendship have provided me with a source of inspiration and motivation that has helped me through the toughest times. Her belief in my abilities and her willingness to listen and provide guidance has been invaluable to me. I am grateful for her friendship and dedicate this thesis to her in recognition of her significant contribution to my journey.

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Chapter 1

Introduction

Open channels are important but sensitive environments because they integrate the hydrologic response of a watershed (Viessman, Harbaugh, & Wang 1972; Dingman, 2015). The hydrologic cycle shows this response as a set of processes in series and parallel that includes precipitation, evaporation, transpiration, infiltration, groundwater flow and surface flow (Maidment, 1993). In every watershed, water transfers towards catchment outlet using two different paths: groundwater flow and surface flow. The balance between surface and subsurface flow paths, in particular, strongly affects the physical and ecological processes in a river. Surface flow paths move water to open channels and the watershed outlet much faster than groundwater flow paths. Urbanization, which is characterized by extensive impervious land cover, is known to increase the rate of surface runoff and to decrease infiltration (Figure 1.1). This rapid routing of water to open channels can lead to extreme floods in urban areas and extensive geomorphologic changes in rivers.

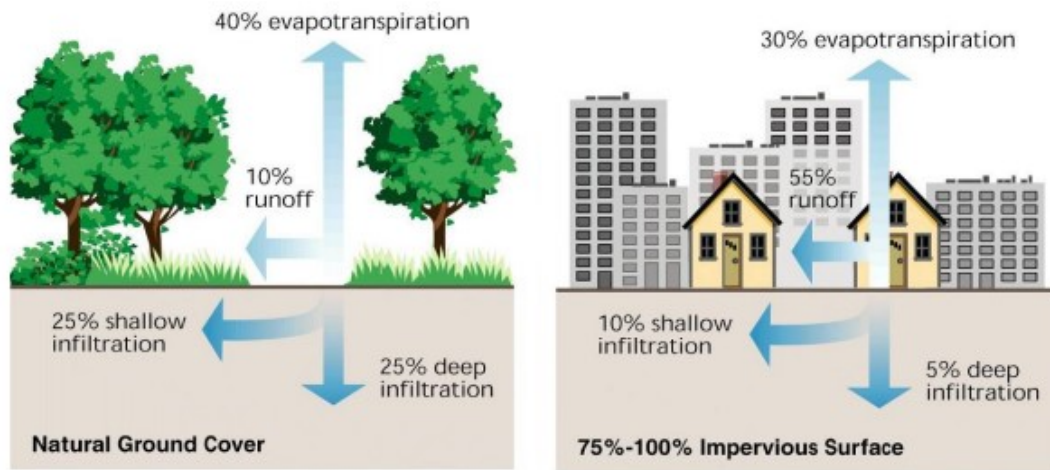


Figure 1.1 Changes in impervious lands and its effects on hydrologic cycle (Zevenbergen, C., Cashman, A., Evelpidou, N., Pasche, E., Garvin, S., Ashley, R., 2011)

The term “flashy” is often used to define the higher degree of unsteadiness of flows in urban rivers compared to ones observed in rural channels (Hasenmueller, Criss, Winston, & Shaughnessy, 2017; Schoonover, Lockaby, & Helms, 2006). These studies characterized flow hydrographs in urban channels by shorter times to peak (i.e. the time interval between the moment flow begins to increase and that when the peak discharge occurs), higher peak flows, and rapid changes in flow depth and

discharge compared to flows in rural catchments (Walsh, Roy, et al., 2005). While such rapid changes are known, unsteady flows are rarely considered in urban hydraulic models for floodplain delineation and risk management practices. Based on the data provided by the Government of Canada, the federal payments under disaster financial assistance arrangements for floods occurring since 1970 is approximately \$13.7 million annually (Government of Canada, 2013). These costly damage indicate that calculated floodplain maps might not be accurate. These damages could be reduced if a more accurate method of flow routing could be used to delineate floodplains and to assess the risk of each flood. Therefore, ensuring that flow routing methods are accurate should be a priority for assessing risks associated with flood events.

Urban floods occur frequently around the world causing major erosion issues and severe damage to infrastructure every year (O'Brien & Leichenko, 2000; Williams & Jacobs, 2011). Channel erosion is an ongoing problem in urban channels, putting neighboring structures in danger (Graf, 1975; Wolman, 1967). As mentioned previously, urban floods are known to have unsteady features. Flow unsteadiness is known to affect graded sediment transport with high annual sediment transport rate (Berta & Bianco, 2010). Discharge alteration, which happens frequently in urban areas, is the main reason for urban erosion issues and can affect channel morphology (Bombar, Elçi, Tayfur, Güney, & Bor, 2011; Gregory, 2006; Plumb, Annable, Thompson, & Hassan, 2017). Due to the complexity of bedload transport, studies are mostly focused on investigating erosion under steady flows. Previous studies have considered shear stress considering only steady flows. However, laboratory studies have shown that sediment transport can be affected by the shape of a hydrograph (Lee, Liu, & Cheng, 2004; Wang, Cuthbertson, Pender, & Cao, 2015). Further research on how sediment transport and erosion may be affected by the impact of flow unsteadiness is required.

In the current thesis, a comprehensive study was conducted to investigate flood waves in urban open channels and their effects on rivers. This thesis has three objectives: 1. use a database of high frequency (2-5 min) measurements to examine the characteristics of flood hydrographs in a set of small watersheds with different land use and history of stormwater management 2. Further investigate the two-gauge method developed in the current research and unique character of urban floods, and 3. use a physical model to examine the impact of rising limb slope on sediment transport. In Chapter 4 there was also a need to develop a novel particle tracking method that would allow us to characterize the spatial variability of sediment transport during a hydrograph. This thesis is presented in three chapters that each address an objective. In Chapter 2 the level of flow unsteadiness of hydrographs observed in

urban areas compared to that in rural areas is investigated. It addresses the question that how “flashy” urban floods are compared to rural floods. It also investigates the impacts of stormwater management facilities on hydrologic and hydraulic parameters in urban channels. Chapter 3 tests hydraulic routing methods using one or two gauges and high frequency measurements for an urban catchment that was found in Chapter 2 to have highly unsteady flows. This chapter also focuses on effects of hydrograph shape on the flow parameters by routing the flow using kinematic, diffusion, and dynamic routing methods. Chapter 4 describes a laboratory experiment where the effect of the unsteadiness on erosion and bedload transport is investigated.

The proposed study is motivated by previous research on flow and sediment transport in three urban creeks in Ontario (Figure 1.2) (Bevan et al., 2018; Papangelakis, MacVicar, & Ashmore, 2019; Papangelakis, MacVicar, Montakhab, & Ashmore, 2022). Of the three watersheds two are highly urbanized, one with SWM facilities to attenuate floods (Morningside Creek) and the other one without any SWM facilities (Wilket Creek). The third creek (Ganatsekaigon Creek) is located in a rural area near the city of Toronto and serves as a control for the urban sites. The Wilket Creek watershed has experienced extreme floods that cause extreme channel erosion and damage to urban infrastructure (Barr & Ashmore, 2017). Sections of the rivers have been repeatedly stabilized by the City of Toronto and efforts are ongoing through much of their lengths as part of a restoration master plan (Parish Aquatic Services, 2013). Due to the uncertainty about stabilization designs, a monitoring study was initiated approximately five years before this research was started (Chapuis, Bevan, Macvicar, & Roy, 2014; Papangelakis & MacVicar, 2020). Pressure gauges were installed to measure water depths during floods in all three watersheds with a frequency of 2 minutes during summers and 5 minutes during winters. Morningside Creek is an example of a contrasting scenario in which investments were made in stormwater management facilities, flow diversion structures, and channel reconstruction at the time of development. This strategy is also expensive with uncertain benefits. Previous work showed that the rising limb of typical floods in Wilket Creek was extremely fast in comparison with the neighboring rural catchment (Ganatsekaigon Creek Watershed) (Papangelakis et al., 2019). Morningside Creek showed changes in the hydrograph shape that were consistent with the design objectives of large retentions structures to control the volume of runoff, but sharp rising limbs and long durations of floods over possible transport thresholds motivate further research on the role of flow unsteadiness and hydrograph shape on flooding and sediment transport in the watersheds (Papangelakis et al., 2019).

These observations were a trigger to motivate us to further investigate floods occurring in the three watersheds.

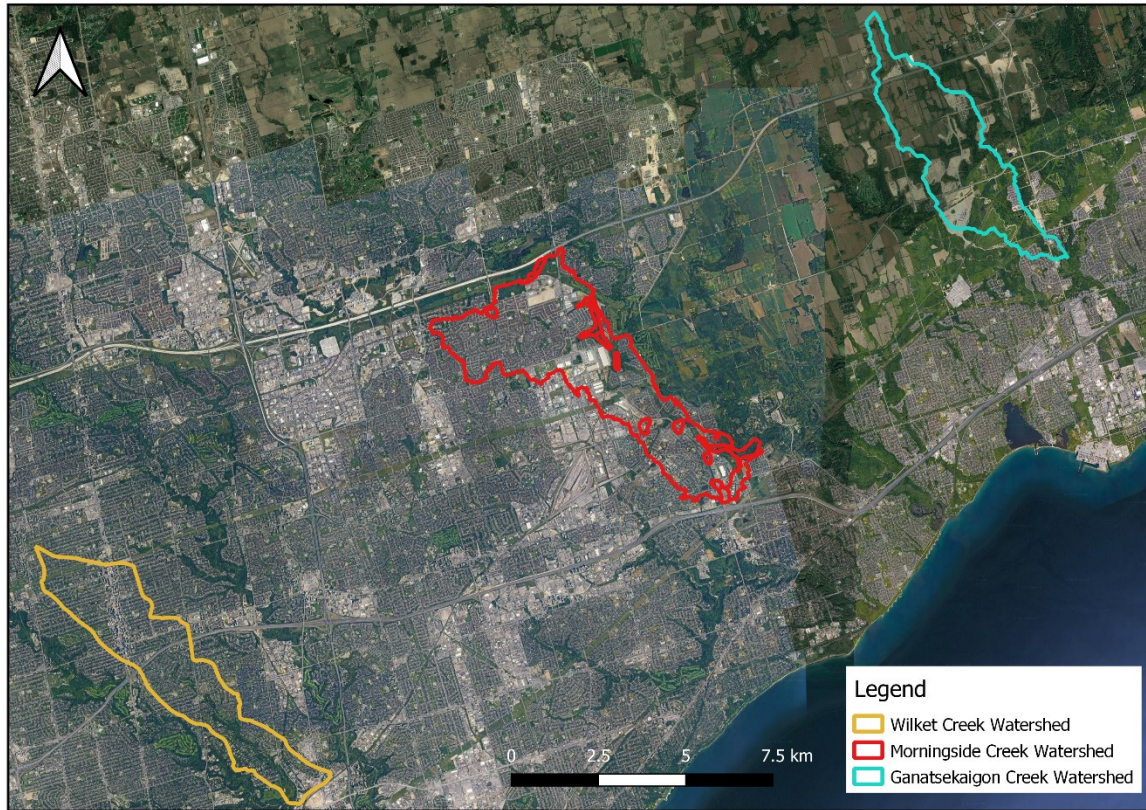


Figure 1.2 Location of the three watersheds with respect to each other

Previously collected water level data showed that even two-minute data frequency is not enough to capture more than one or two points on the rising limb in Wilket Creek. Therefore, higher frequency gauges with 15 seconds frequency were installed in Wilket Creek and the distance between sampling points was reduced to capture the unsteady flows and flood events occurring in this creek. To measure water surface slope, two high frequency gauges were installed in Wilket Creek. This value and Saint Venant equations were used to calculate flow downstream. To evaluate our method, we measured velocities during high flows using a PIV method and a radar gun at the downstream location.

As stated above, sediment transport is extensively investigated under steady floods but not under unsteady urban floods. For this purpose, we used a channel that was previously scaled and constructed in a flume in the University of Waterloo Hydraulics laboratory (Pierce, S., MacVicar, B. J., Papangelakis, E., Vervynck, L., & Ashemore, P., 2021). The channel is a 1/40 scale of Wilket Creek.

For this study, we used a hydrograph that was observed in Wilket Creek to come up with three triangular hydrographs. The channel bed was seeded evenly from upstream to downstream with scaled sediments. The sediments were painted with fluorescent colors and the sediment transport was recorded at one location in the middle of the flume. Black lights were used to make the sediments shine in the dark and to make them more visible in our videos (Figure 1.3). Previous experiments had mostly measured sediment output, but in our research a novel method was used using black lights to find sediment transport at any location at the flume.

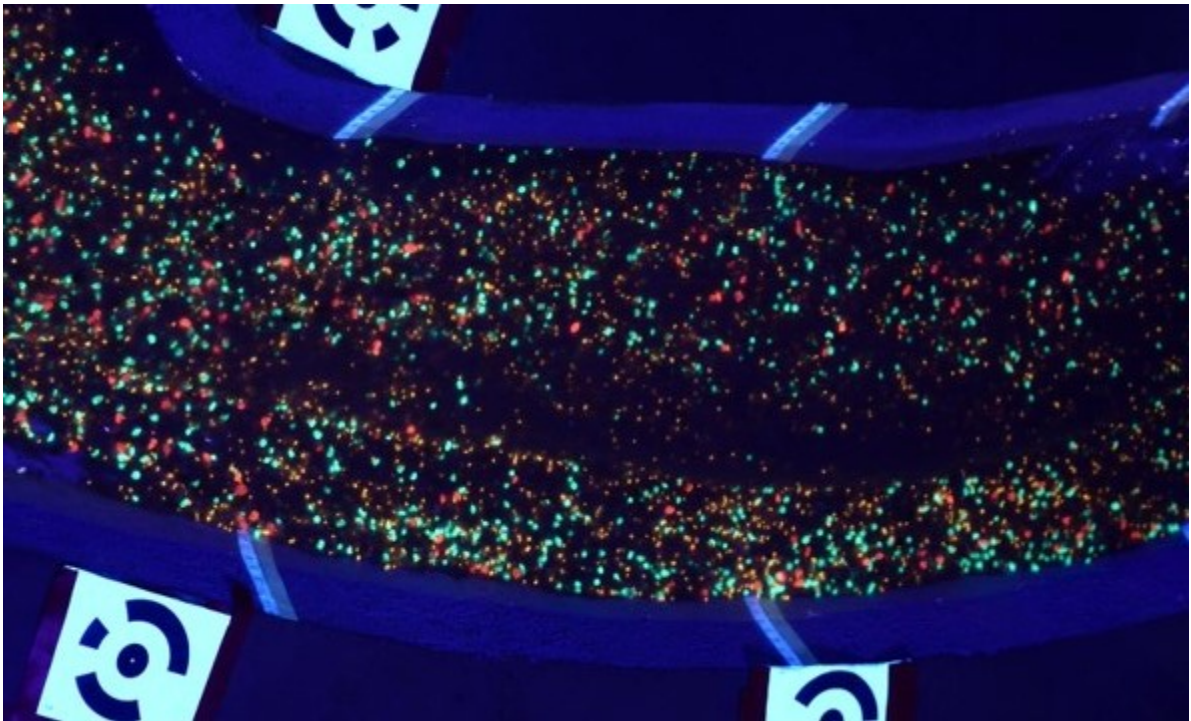


Figure 1.3 A screen shot of the video taken with all the sediments seeded.

This thesis is organized in five chapters, with chapters Chapter 2 through Chapter 4 comprised of the main results that have been prepared as manuscripts for journal submission. In Chapter 2 flow gauge measurements from the three watersheds with 2-5 min data are compared to assess the hydraulic characteristics of the observed floods. In Chapter 3 additional gauges recording at higher frequencies (15 seconds) are added to the most highly urbanized watershed (Wilket Creek) to assess the errors associated with one and two gauge routing methods and make recommendations for future gauging methods. In Chapter 4 Wilket Creek is modelled in the lab and the impacts of flow unsteadiness on

sediment transport are assessed. The key discussion points and conclusions are summarized in Chapter 5, where the practical implications of the work and future research directions are also discussed.

Chapter 2

How fast is “flashy”? Hydraulics of flood hydrographs in small urban rivers

2.1 Introduction

Cumulative impacts of urbanization and stormwater management on channel hydraulics and sediment transport are poorly understood. Many studies have shown that floods in urban areas are more “flashy”, meaning that they have faster rising limbs (Walsh, Roy, et al., 2016; Xia, Falconer, Lin, & Tan, 2011). Laboratory studies have shown that sediment transport is sensitive to flow unsteadiness (Bombar, Elçi, Tayfur, Güney, & Bor, 2011; Lee, Liu, & Cheng, 2004), and the change in the rate at which floods in urban areas rise and fall has been linked with sedimentation and channel degradation (Trudeau & Richardson, 2015; Vietz et al., 2014; Walsh, Roy, et al., 2016). Urban flow unsteadiness may also contribute to the perturbation of fish habitat and river ecosystem function (Stanfield & Kilgour, 2006; Walsh, Fletcher, & Vietz, 2016), which are typical symptoms of the “urban stream syndrome” (Walsh, Roy, et al., 2005). Stormwater management is employed to mitigate some of the hydrologic and hydraulic impacts of urban floods, but the cumulative impacts of these efforts may be hard to discern at a watershed scale (Bell et al, 2016, Hopkins, Bhaskar, Woznicki, & Fanelli, 2020). The role of flow unsteadiness for driving channel degradation and the mitigative effects of stormwater management remain relatively unexplored areas of research that may be significant for managing urban rivers.

Considerable research has helped to characterize the impacts of urbanization on hydrologic flood metrics (Hollis, 1975; Meierdiercks, Smith, Baeck, & Miller, 2010; Shuster, Bonta, Thurston, Warnemuende, & Smith, 2005; E. S. Williams & Wise, 2006). Regional variation (Poff, Bledsoe, & Cuhacyan, 2006) and physical factors such as the topography and soils are important modifiers of the response (Hopkins et al., 2015), but increasing urban development is generally understood to result in increases to drainage slope and direct runoff while decreasing evapotranspiration, groundwater recharge and baseflow volume (Leopold, 1968). These changes generally lead to an increase of small frequent floods in urban areas (Hollis, 1975). Roesner, Bledsoe, & Brashear, (2001) showed that the peak flow for a given return period increases significantly as a function of the percent of impervious surfaces, while Walsh, Roy, et al., (2016) showed that urbanization results in an increase in the frequency of overland flow, the frequency of erosive flow, the magnitude of high flow, and a decrease

in the lag time to peak flow. A number of metrics have been advanced to try to capture the flashiness of a watershed system. Sanborn & Bledsoe, (2006), for example, propose a metric called ‘flash’ that represents the average annual 1-day flow peak discharge over the average flow for all years. Hopkins et al., (2015) calculated the high flow frequency and then defined the ‘flashiness baseline’ and the ‘flashiness response’ as the intercept and slope of the relation between high flow frequency and road density, where road density acted as a proxy for impervious cover. Stormwater management (SWM) facilities have been designed to mitigate some of the adverse hydrologic impacts of urbanization by decreasing peak flows and therefore, application of SWM techniques is now common in urban developments (Walsh, Fletcher, & Ladson, 2005). However, these facilities are generally designed for smaller watershed areas Roy et al., (2008), and several studies have suggested that the impacts of SWM are negligible for larger watersheds, which brings the question of whether it is worth the effort (Bell, McMillan, Clinton, & Jefferson, 2016; Lee & Huang, 2012). As recommended by Jefferson et al., (2017), more research is needed to understand flood hydraulics in urban channels conditioned by stormwater management.

As described by Kvocka, Falconer, & Bray, (2015) flashy floods have important implications for the choice of hydraulic modelling strategy. An over-simplified approach can lead to incorrect estimates of flood elevations and extents, with potentially devastating effects. If the floods rise and fall slowly, then assumptions implicit in gradually varied flow calculations will be sufficient to accurately model flood elevations and shear stresses (Neal, Schumann, & Bates, 2012). If flood elevations are ‘flashy’ enough, however, then hydraulic models that represent rapidly-varying flows are unavoidable for accurate representation of flood risk (Costabile, Costanzo, De Lorenzo, & Macchione, 2020a; Fraccarollo & Capart, 2002). Urban flood inundation models are currently grappling with these issues of model accuracy, data availability and ultimate purpose of the model (Teng et al., 2017). Rapidly varying flow analysis are already known to be required for some flood inundation overland flows (Fewtrell, Bates, Horritt, & Hunter, 2008; Hunter et al., 2008; Kvocka, Ahmadian, & Falconer, 2017; Singh & de Lima, 2018) and in stormwater drainage pipe networks (Hunter et al., 2008; Kvocka et al., 2017, 2015; Xia et al., 2011). In channels these effects are typically limited to steep narrow systems such as that measured by Kvocka et al., (2015), but the rapid upstream drainage and observations of rapidly-varying flow suggest that such dynamics could be relevant to flow in urban channels. To date there has been little research into the relevance of flow unsteadiness in urban channels for the estimation of bed shear stress for sediment transport modelling.

The goal of the current research is to investigate the unsteady hydraulics at urban sites with and without stormwater management facilities. Three urban creeks in southern Ontario were selected for this research, all of which had been the locations for sediment tracer studies (Papangelakis, MacVicar, & Ashmore, 2019; Cain et al., 2020). During the previous research, rapid rising limbs were noted so this work was motivated in part by the need to understand the significance of these rapid rising limbs for modelling sediment transport. To better capture the rapid rising limbs, the frequency of water depth sampling was increased so that the database includes multiple years of high frequency water surface elevation measurements that are suitable for hydraulic analysis. One watershed has stormwater management facilities, which allowed us to assess the impact of this management strategy for this case. A rural watershed with similar watershed area and slope was included for comparison. The hope is that this case study will give a better understanding on the significance of unsteadiness for modelling urban channel hydraulics and sediment transport.

2.2 Background

2.2.1 Saint-Venant Equations

The Saint-Venant or ‘shallow-water’ equations are commonly used to model steady and unsteady flows in open channels. In a 1D formulation, these equations can be derived based on five main assumptions: i) hydrostatic pressure distribution; ii) low channel slope; iii) uniform velocity distribution over the channel cross sectional area; iv) prismatic channel; and v) approximation of energy losses in unsteady flow using a flow resistance equation derived for steady flow (Chaudhry, 2008). The continuity and momentum equations can be written as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \quad (2.1)$$

$$\begin{array}{c} \text{Dynamic routing} \\ \hline \text{Diffusion routing} \\ \hline \text{Kinematic routing} \\ \hline \underbrace{S_f}_{\text{Term1}} = \underbrace{S_0}_{\text{Term2}} - \underbrace{\frac{\partial y}{\partial x}}_{\text{Term3}} - \underbrace{\frac{V}{g} \frac{\partial V}{\partial x}}_{\text{Term4}} - \underbrace{\frac{1}{g} \frac{\partial V}{\partial t}}_{\text{Term5}} \end{array} \quad (2.2)$$

Where Q is flow discharge ($\frac{m^3}{s}$), A is the cross sectional area (m^2), S_f is the frictional slope that represents the rate of energy loss along the flow path, S_0 is the channel bottom slope, y is the flow

depth (m), t is time (s), V is longitudinal flow velocity ($\frac{m}{s}$), and g is the gravitational acceleration ($\frac{m}{s^2}$). In this equation term 3 shows water surface slope and terms 4 and 5 are convective and local acceleration terms respectively. To solve the equations, V can be calculated using a flow resistance equation such as the Manning equation, written as:

$$V = \frac{1}{n} R^{2/3} S_f^{1/2} \quad (2.3)$$

where R is hydraulic radius (m) ($= A/P$, where P is the wetted perimeter (m)), and n is Manning's roughness ($\frac{s}{m^{1/3}}$), which can be calculated using different formulations such as the Keulegan approach:

$$n = \frac{R^{1/6}}{5.75 * \sqrt{g} \log(\frac{12.2R}{K'_s})} \quad (2.4)$$

where K'_s is equivalent roughness (m), written as:

$$K'_s = 2.8 * D_{84} \quad (2.5)$$

where D_{84} is the grain size that is larger than 84 percent of bed material (m) and the equivalent roughness coefficient is based on empirical results (López & Barragán, 2008). Using these 1D equations, the shear stress on the bed can be estimated as:

$$\tau = \gamma R S_f \quad (2.6)$$

where γ is the specific weight of water ($\frac{N}{m^3}$).

Formulated in this way, the momentum Equation (2.2) allows the unsteady effects to be represented in the estimates of Q and τ as a function of the streamwise changes in channel depth (Term 3) and the streamwise and temporal acceleration (Terms 4 and 5) such that S_f deviates from S_0 when these terms are nonzero. Where these terms are small they can be ignored, which leads to three routing methods referred to as kinematic, diffusion and dynamic routing (Chaudhry, 2008; Sturm, 2001). The three methods have tradeoffs in computer processing time, input data requirements, and accuracy, so it is important to choose a method that is sufficient to represent the flows being modelled.

The kinematic routing equation is derived using the uniform flow assumption ($S_f = S_0$) which results in a unique relationship between the discharge and depth for the rating curve, which tends to follow a

power function. This method considers the flow to be steady-uniform, and is able to track unsteady flows with gradual change in flow properties over time (Chaudhry, 2008; Le Coz, Renard, Bonnifait, Branger, & Le Boursicaud, 2014). This method is also compatible with available data in river channels, which typically consists of water level data over time recorded at intervals of 15 minutes or more. Sturm, (2001) introduces an equation to show the limit for the kinematic wave.

$$\frac{T_r V_0 S_0}{y_{ave}} > 85 \quad (2.7)$$

where T_r is rising limb duration (s), V_0 is the average velocity (m/s), and y_{ave} is average depth for that hydrograph (m). While the computational cost for routing flows with this method is low compared to the other methods, it may introduce error to the calculation of Q and τ when applied to flashy urban flows with short rising limbs.

The diffusion routing method includes the term $\frac{\partial y}{\partial x}$ in the momentum equation, which thus incorporates nonuniform flow and results in a looped rating curve such that different values for Q and τ will be calculated on the rising and falling limb of the flood. The dynamic routing method similarly results in a looped rating curve but includes all the terms in the momentum equation, which makes it suitable for unsteady flows with rapid changes in flow properties such as dam breaks, hydraulic jumps, and tsunamis (Creed & Borthwick, 2017; Gharangik & Chaudhry, 1992; Yasuda, 2010). Required boundary conditions for the diffusion and dynamic methods include high frequency depth and velocity data, which is expensive to collect and rarely available (Hunter et al., 2008). Even where the data is available, the computational costs can be high.

2.3 Methods:

2.3.1 Site Description:

Three small watersheds around the city of Toronto, Ontario, were chosen to assess the impact of urbanization and stormwater management on unsteady flow parameters (**Error! Reference source not found.**, Table 2.1, and Table 2.2). The watersheds were selected to have similar size, shape and geology but varied in terms of their urban history. All three have relatively high shape factors, indicating long and narrow watersheds with few tributaries, and average slopes between 2 and 5%, with Ganatsekaigon the steepest and narrowest of the watersheds. Ganatsekaigon Creek watershed has a low percentage of impervious land cover with a mix of agricultural and forested land cover. Wilket Creek is highly

urbanized with no significant stormwater management facilities such that the storm sewer networks are directly connected to the channel (Bevan et al., 2018). Morningside Creek is also urbanized, but has a set of stormwater management facilities that are designed to reduce the peak discharges and volume of storm runoff that is conveyed through the channel.

The SWM facilities in Morningside are comprised of seven SWM ponds/wetlands and two structures that impound or divert water on the main branch of the creek. Two large pond/wetland facilities with a total active volume of $2.82 \times 10^5 \text{ m}^3$ control the runoff from the area located roughly within the City of Markham (Drainage Area (DA) = 10.4 km^2). These SWM features (referred to herein as the upper ponds) were designed to control the peak flows from the 2 to 100 year storm events to their predevelopment levels (Badelt, 1999). The upper ponds are designed to reduce potential erosive flows downstream by storing the first 25mm of rainfall runoff, which corresponds with approximately 77% of the total active volume, and ensuring that it is released over a period of no less than 24 hours (Badelt, 1999). This storage volume within the ponds is called ‘extended detention’. A structure called the Tapscott Diversion was constructed downstream of the upper ponds where, due to a quirk of geography, Morningside Creek passes only 0.37 km from the main branch of the Rouge River. The Tapscott Diversion is designed to overflow during storm events and divert 50 to 80 % of the flow from the upstream area (DA = 11.5 km^2) to the Rouge River through a large pipe. Downstream of the diversion, an additional 5 ponds with a total active volume of $9.9 \times 10^4 \text{ m}^3$ control outflow from an additional 3.7 km^2 of largely residential urban area. These smaller ponds are designed only for extended detention that impounds and releases flow from a 2 year 33 mm event over a period of 24 hours (Raso, 2017). A final control structure upstream of the gauge location is an online stormwater pond controlled by a small culvert with a ditch inlet grate and a drop structure of large flows (Raso, 2017). Little design information was available for this structure, but the limited size of the reservoir would suggest that it does not have a major impact on flow routing through the watershed.

Characteristics of the gauged reaches are shown in Table 2.2. As for the watershed characteristics, an effort was made to match the channel slope and roughness at the three Toronto sites. Wilket is the coarsest of the other Toronto sites, particularly in a restored section of the channel where coarse riffles were added to reduce channel erosion (Papangelakis & MacVicar, 2020). For the current study, pressure transducer gauges located at two riffle crests were selected for analysis. The primary gauge was selected based on data quality and the length of the available record. A secondary gauge was also selected that was close enough to be sensitive to differences between water surface and mean bed slope. However,

the gauges were placed for previous studies having different objectives (Papangelakis et al., 2019; Papangelakis & MacVicar, 2020) and so the distance was not controlled. The different distances between the gauges at the three watersheds affect the final results as discussed in the discussion section. These gauges were model HOBO-U20 Water Level Data Loggers, manufactured by Onset (± 0.5 cm error). Water level was recorded every 1 or 2 minutes from April to November and every 7 minutes from December to March when it was challenging to download data regularly due to ice. Atmospheric pressure gauges at each creek were used to correct for barometric pressure variation and calculate water depth.

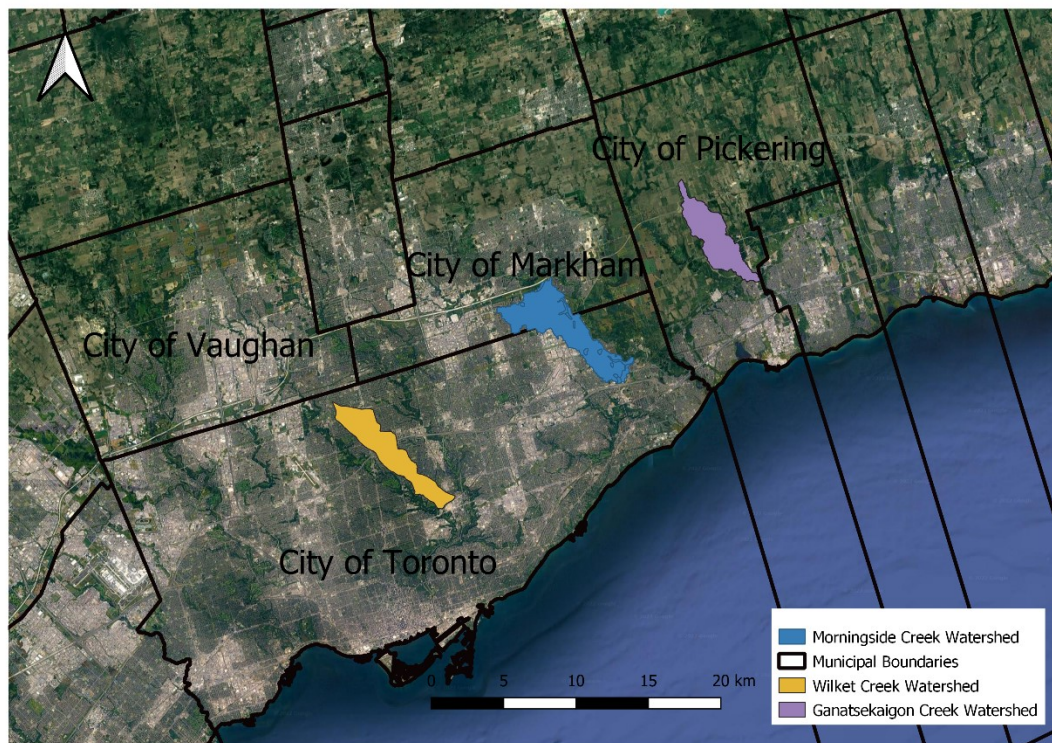


Figure 2.1 Location of the three watersheds in relation with each other and the City of Toronto, ON

Table 2.1 Watershed characteristics upstream of gauge locations. Unless otherwise indicated, information is from the Ontario Flow Assessment Tool

Watershed	DA (km ²)	Shape Factor ⁺	Mean slope (%)	Land cover (%)					SWM
				Lake/ Wetland	Forest	Agr./ Open	Developed	Impervious	
Wilket	12.9	9.9	3.0	0.6	3.1	1.7	94.6	47	NO*
Morningside	16.7	9.5	2.3	0.7	3.4	18.8	77.1	46	YES
Ganatsekaigon	12.0	10.1	4.9	2.6	20.1	72.5	4.5	4	NO

⁺calculated as the square of the main channel length divided by the drainage area. *Only small natural SWM ponds exist such as wetlands

Table 2.2. Reach Characteristics for gauge locations. Distance refers to the streamwise distance between the two gauges.

Creek	Chan nel slope	Sinuosity	Bankfull width (m)	Sediment Size (m)		Available Gauge data	Gauge names	Primary or 2 nd	Distance between gauges (m)	Distance between gauges /width
				D50	D84					
Wilket	0.01	1.37	13.56	0.040	0.138 (0.349)	2015- 2018	WCrk5	Primary (u)	347	26
							WCrk1	2 nd (d)		
Morningside	0.011	1.18	8.22	0.041	0.112	2016- 2018	MCrk20	Primary (u)	133	16
							MCrk10	2 nd (d)		
Ganatsekaig- on	0.012	1.02	12.33	0.037	0.12	2015- 2018	GCrk20	2 nd (u)	51	4
							GCrk10	Primary (d)		

2.3.2 Data preprocessing

Water elevation data was generally of good quality but some errors were noted due to occasional spikes and seasonal errors. Spikes were sometimes observed when data was downloaded due to the need to physically remove and replace the gauges to download the data. Such errors were removed manually. Step changes were sometimes observed coincident with the time when data was downloaded from the loggers, which were thought to be related to slight positioning errors of the gauges during replacement. These errors were eliminated by manually removing a step change so that the post-download water surface elevation matched the pre-download elevation. Certain periods within the data appeared to have

higher errors. During the winter, for example, the data were not reliable due to the impact of ice on the pressure data. For this reason, only the data between the months of April and November of each year were used for the analysis. In Morningside Creek, a particular problem was observed for the 2015 data where high baseflow and rapid fluctuations were noted, likely due to the construction and failure of an upstream beaver dam. For this reason, the Morningside Creek data from 2015 was not considered during the analysis.

2.3.3 Base flow separation

A three-step process was used to separate baseflows from the event-based flood hydrographs to allow analysis of the hydrographs for the current study. In the first step, a low-pass digital Butterworth filter using a tool in MATLAB called “butter”, which is derived by Frederick & Carlson, (1971) was applied to attenuate the high frequency oscillations in the measured water surface elevation data. This filter requires two inputs: the order for the Butterworth filter and a normalized cutoff frequency, which is the relationship between cutoff frequency and Nyquist frequency. For the current study, an eighth-order Butterworth filter was applied using the MATLAB “filtfilt” function to eliminate phase shift errors following (Restrepo & Kjerfve, 2000). A cutoff frequency of $5.5 * 10^{-4} \text{ HZ}$ (i.e. $\sim 2 \text{ hr}^{-1}$) was selected for this study based on the consideration that the shortest floods observed had durations of approximately 0.5 hours and qualitative assessments of the filter performance. The filtering step tended to reduce the peak discharge and dampen rising limb slopes, so the filtered water surface elevation was only used to separate the hydrographs. For further analysis the raw data was used in the analysis.

In the second step the base flow separation method proposed by Arnold, Allen, Muttiah, & Bernhardt, (1995) was applied. Other base flow separation methods have been developed (Aksoy, Kurt, & Eris, 2009; Arnold & Allen, 1999; Furey & Gupta, 2001; Indarto, Novita, & Wahyuningsih, 2016; Liu et al., 2019) but these were developed for daily streamflow data and so were not suitable for the high-frequency data gathered for this study. Other low pass filters used for base flow separation such as the two parameter recursive digital filter presented by Eckhardt, (2005) were also considered, but the calibration required to find the optimal value for the two parameters leads to an increase in the computational time (Eckhardt, 2012). Using the Arnold et al., (1995) method the baseflow (b) is separated from the total stream flow (Q) at each time step (t) as:

$$b_t = Q_t - q_t \quad (2.8)$$

where the surface runoff (q_t) is calculated as:

$$q_t = \beta q_{t-1} + \frac{1 + \beta}{2} * (Q_t - Q_{t-1}) \quad (2.9)$$

and β is the filter parameter. The filter was applied one time (only first pass) following the recommendations of (Arnold et al., 1995). This method can be adapted to different time steps by adjusting β . A value of $\beta = 0.925$ is suggested for daily stream flow data (Nathan & McMahon, 1990) but for our data a value of $\beta = 0.9992$ was used because it resulted in a minimal increase in baseflow during flashy events in the urban creeks while capturing some of the increase in baseflow that was likely for the slower events in Ganatsekaigon Creek.

The third step required for the event-based hydrograph analysis was to eliminate a series of low amplitude events that occurred even in the absence of any precipitation in the watershed (Figure 2.2). Such fluctuations commonly occurred during the daytime in Wilket Creek and were thought to be related to cross-connection to sanitary sewers or other anthropogenic sources that are unrelated to surface runoff. To eliminate these low amplitude events a threshold was chosen and surface runoff hydrographs were only considered if they exceeded this threshold. The start of the event was defined as the moment when the discharge exceeded this threshold for the first time while the end of the event was defined using the baseflow separation method.

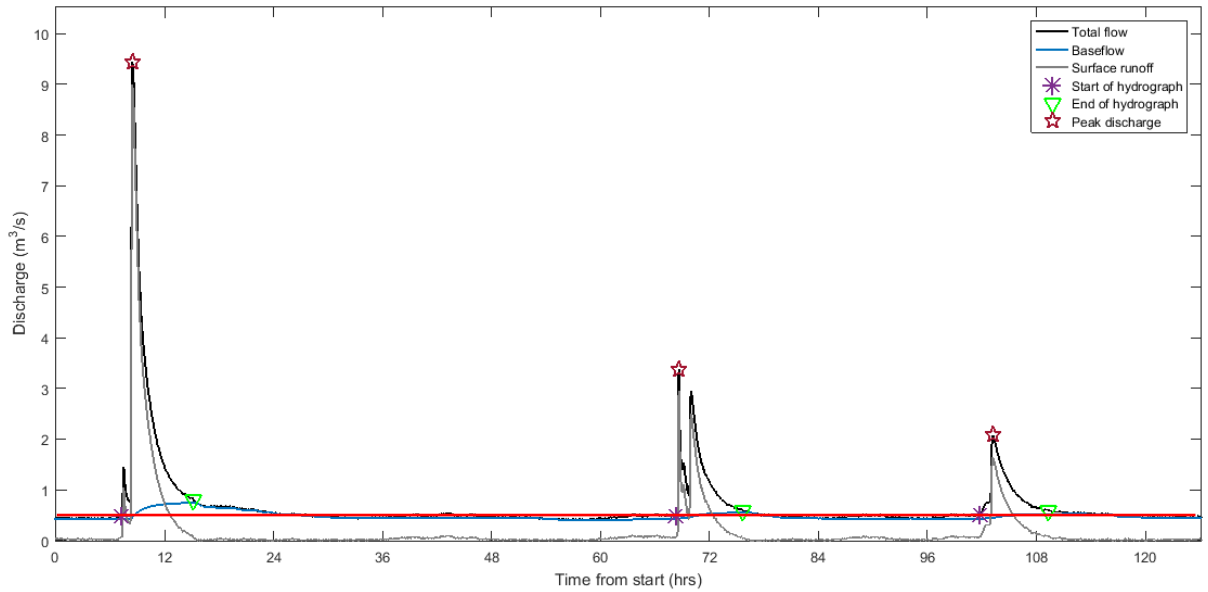


Figure 2.2 Base flow separation method. Total flow, baseflow, and surface runoff and a schematic threshold shown by the horizontal red line.

2.3.4 Hydrograph metrics

Few studies assess the validity of the flow routing approaches for urban flows using the terms in the Saint Venant equations. Water levels are also rarely recorded at the frequencies obtained for the current study, so it was necessary to compare using other metrics that have been developed to assess differences in the flashiness of a watershed's hydrologic response. Due to the high frequency data, metrics that used daily flows (e.g. Sanborn & Bledsoe, 2006) were not suitable, while other indirect measures that required measurements of road density, for example Hopkins et al., (2015) or assessments of the channel bed material and erodibility (e.g. Phillips, Rey, Marden, & Liébault, 2013) were also not considered. Instead we used metrics based on the shape of flood hydrographs such as the slope of the hydrograph rising limb (s_r), written as:

$$s_r = \frac{\Delta H}{T_r} \quad (2.10)$$

where ΔH is change of depth between peak depth and base flow depth and T_r is rising limb duration. This parameter has been normalized in some studies, particularly in lab studies, where researchers such as Wang et al., 2021 define an unsteadiness parameter (Γ_{HG}) as:

$$\Gamma_{HG} = \frac{1}{u_b^*} \frac{\Delta H}{\Delta T} \quad (2.11)$$

where u_b^* is shear velocity of baseflow ($= \sqrt{\tau_o/\rho}$), with ΔT defined using the total duration of the hydrograph.

2.3.5 Calculating Q and τ from water level data

The Saint Venant equations require discharge and velocity measurements to be solved directly. However, velocity measurements in rivers data are typically made intermittently and used to calibrate a $y - Q$ rating curve for a given hydrometric station location. The most widely accessible field data is water level data recorded over time using a pressure or float gauge, from which V and Q can be estimated using a flow routing method. A dynamic routing method with two gauges is used in this chapter for the purpose of showing flow unsteadiness parameters. This dynamic method, referred to as “F2” in the current thesis uses all terms in the momentum equation with water depth measurements from two locations in the channel (one upstream and one downstream). As described by Chaudhry, (1993), the combined continuity and momentum expression for Q can be written as:

$$Q = Q_r \sqrt{1 - \frac{1}{S_0} \left(\frac{1}{A} \frac{\Delta(A\bar{y})}{\Delta x} + \frac{V}{g} \frac{\Delta V}{\Delta x} + \frac{1}{g} \frac{\Delta V}{\Delta t} \right)} \quad (2.12)$$

This equation is written as below to eliminate the velocity term and to be solved mathematically.

$$Q = Q_r \sqrt{1 - \frac{1}{S_0} \left(\frac{1}{A} \frac{\Delta(A\bar{y})}{\Delta x} + \frac{Q}{Ag} \frac{\Delta(Q/A)}{\Delta x} + \frac{1}{g} \frac{\Delta(Q/A)}{\Delta t} \right)} \quad (2.13)$$

To find values of terms in equation (2.13) that change over time we used a forward scheme, written as:

$$\left. \frac{df}{dt} \right|^{t=t_0+\Delta t} = \frac{f(t_0 + \Delta t) - f(t_0)}{\Delta t} \quad (2.14)$$

where f is any dependent variable in the Saint Venant equation such as Q or A , t_0 is the time step with known or previously calculated variable value, and the $t_0 + \Delta t$ is the time step for which we are calculating the variable.

For the changes of flow parameters over distance, a first-order accurate forward scheme was used as described by Chaudhry, (2008). This method was selected over a central difference method because only two gauges were installed in each creek, while the central difference requires three data points in streamline direction. The equation for this scheme is shown below.

$$\left. \frac{df}{dx} \right|_{x=x_0} = \frac{f(x_0 + \Delta x) - f(x_0)}{\Delta x} \quad (2.15)$$

Where x is the distance in streamflow direction, x_0 is the location of the upstream cross section, and $x_0 + \Delta x$ is the location of the downstream cross section.

2.4 Results:

2.4.1 Flood hydrology

Table 2.3 - Average and maximum values of different characteristics of hydrographs observed in each catchment.

Metric	Units	Ganatsekaigon	Wilket	Morningside
Description		Rural	Urban	Urban with SWM
Events per year	#	43	118	54
Mean duration	hr	42.7	17.5	33.6
Mean event Peak Q	m ³ /s	1.8	4.9	1.5
Max Q	m ³ /s	29.8	39.8	8.7
Mean event Peak τ	N/m ²	23.9	40.3	27.3
Max τ	N/m ²	78.2	113.1	58.7

For an initial comparison between the watersheds, the frequency, duration, and peak Q and τ values were calculated using a conventional rating curve with kinematic routing (i.e. $S_o = S_f$). As shown, flood events are more frequent in the urban Wilket Creek than the rural Ganatsekaigon Creek, with higher peak parameters and shorter flood durations (Table 2.3). Morningside Creek, the urban site with SWM ponds, lies somewhere in the middle in terms of event frequency and duration, with peak discharges that are lower even than Ganatsekaigon Creek. Similar results have already been reported by Papangelakis et al., (2019), who also noted that the morphology of this Morningside Creek reach was shallower and narrower than the rural analog, a result that was interpreted to mean that the hydraulic geometry of the channel had already adjusted as a result of these lower peak flows.

The impact of urbanization and stormwater management on system hydrology is also clear from a plot of the normalized event peak discharge frequency (Figure 2.3a). As shown, Wilket Creek experienced flood events with higher peaks more frequently than the other watersheds. Looking at the vertical line

1 m³/s/km², for example, it is apparent that floods exceed this threshold approximately once every two years in Ganatsekaigon and were not observed in Morningside Creek during the study period. In the urban Wilket Creek, however such events occur approximately four times a year. More moderate flows are also affected. Looking at the vertical line of 0.2 m³/s/km², for example, such events occur approximately 1-2 times per year in the rural creek and the urban creek with SWM, but over 20 times per year in the urban creek without stormwater management. Stormwater management meant that the largest peaks were only 0.4 m³/s/km², almost an order of magnitude lower than the events in the rural and urban systems.

The normalized flood volume or runoff depth, which is defined as the runoff volume divided by the catchment area, similarly illustrates the impact of urbanization and stormwater management on flooding. Similar to Figure 2.3a, these distributions follow a roughly symmetrical distribution, with the urban site most noticeable for the high frequency of moderate runoff events. 0.5 to 15 mm runoff events are more common in Wilket Creek, with 5-10 mm nearly an order of magnitude higher than in the rural Ganatsekaigon Creek. The exceptional nature of the urban watershed tends to decrease for larger events, so that events with runoff ≥ 30 mm, which only occur about once per year for the study sites, are seen nearly equally at all sites, though the relatively short duration of the study makes it more difficult to assess the impact on these more extreme events. A comparison with the climate norms from the Toronto Environment Canada weather station (1981-2010-station ID 6158350, 7.4 km from the Wilket site) shows the upper limit of expected event frequencies, which would require a runoff coefficient of 1.0 to match. The urban site trends towards this line for relatively frequent events, indicating relatively high runoff coefficients for events with 10 to 25 mm of precipitation. Rural sites and the stormwater managed sites tend to have lower runoff in this range and only trend closer to the precipitation norms for exceptional events, where the relatively short duration of the study makes it difficult to assess runoff coefficients with this statistical approach.

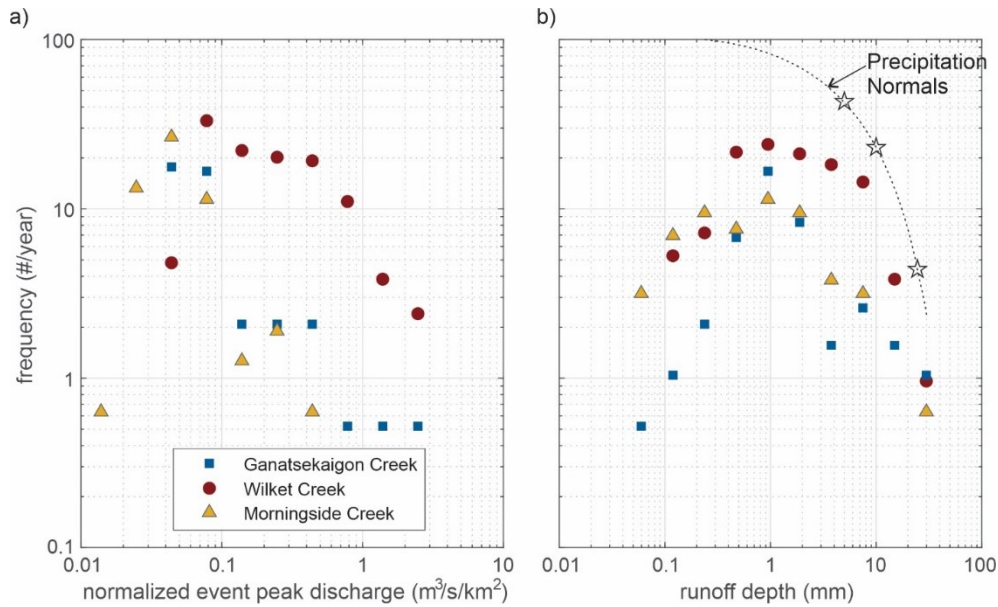


Figure 2.3 - Comparison of study period hydrology for study sites including a) frequency versus event peak discharge normalized by the drainage area; and b) frequency versus runoff depth. Precipitation normals are added for Environment Canada Weather Station (Climate ID 6158350).

2.4.2 Hydraulic parameters

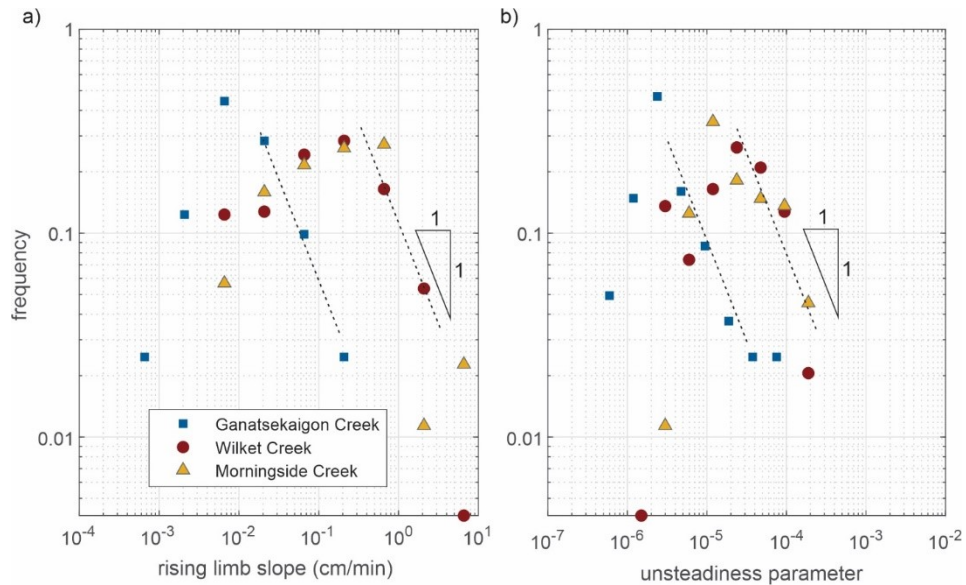


Figure 2.4 – Frequency of a) Rising limb slope, and b) Unsteadiness parameter (categories are based on $10^{(-4:0.25:1)}$). Visual guidelines plotted at a 1:1 slope are added to roughly correspond with the tail of different distributions to help with the description of the results.

The flashiness of a watershed can be expressed in the form of the rising slope, or the normalized versions of this parameter called the unsteadiness parameter (Equation (2.11)). The binned rising slope frequencies follow a roughly symmetrical distribution (Figure 2.4a), which means that differences between the watershed can be described based on the geometric mean and standard deviations of the frequency plots. The impact of urbanization is shown by a comparison of the rising limb slope for the rural and urban streams. For example, the mode is a slope of ~ 0.007 cm/min for Ganatsekaigon Creek (rural), while for Wilket Creek (urban) the mode is ~ 0.2 cm/min. The highest values in the rural watershed are approximately equal to this mode in the urban channel, while the peak values in this urban channel are ~ 6 cm/min. Somewhat surprisingly, there is no discernable impact of the stormwater management on the rising limb slope. Morningside Creek shows very similar mode and extreme values as Wilket Creek, with higher rising slopes even slightly more common.

The unsteadiness parameter does little to change the interpretation of the patterns described from the rising limb slope (Figure 2.4b). This parameter is similar to the rising limb slope, and it is based on a ratio of the peak of the flood and a time – in this case the entire duration of the flood. The rural site again shows a mode and extreme values are roughly an order of magnitude lower than the urban site, even when SWM is in place.

For comparison we plotted the frequency plots of $\Delta h/\Delta t$ where $\Delta h > 0$ to isolate only the depth changes on the rising limbs (Figure 2.5). These instantaneous depth changes are relevant for interpreting differences in the hydraulics of the different sites with respect to the Saint Venant Equations. They are also not sensitive to the separation of the flow hydrograph and identification of discrete events that is required to calculate the events for Figure 2.4. Overall the rivers shown similar trends, with a flat distribution below $\Delta h/\Delta t = 0.1$ cm/min and a negative slope above this level, such that rapid changes in depth are rarer. The accuracy of the gauges is given as ± 1 cm, so the flat distribution below this level can be attributed to white noise, which is frequently observed as random small fluctuations in recorded water level at base flow. Rapid changes in water level depth occur more often in the urban sites than at the rural site. For example, no observations were made of water level changing faster than 4 cm/min over the duration of the study at the rural Ganatsekaigon site. At the urban site, in contrast, water level was recorded as changing as fast as 25 cm/min in some extreme events. Stormwater management appears to reduce the frequency of rapid changes in water elevation in comparison with the urban creek, but events where water level changed faster than 10 cm/min were observed in some cases.

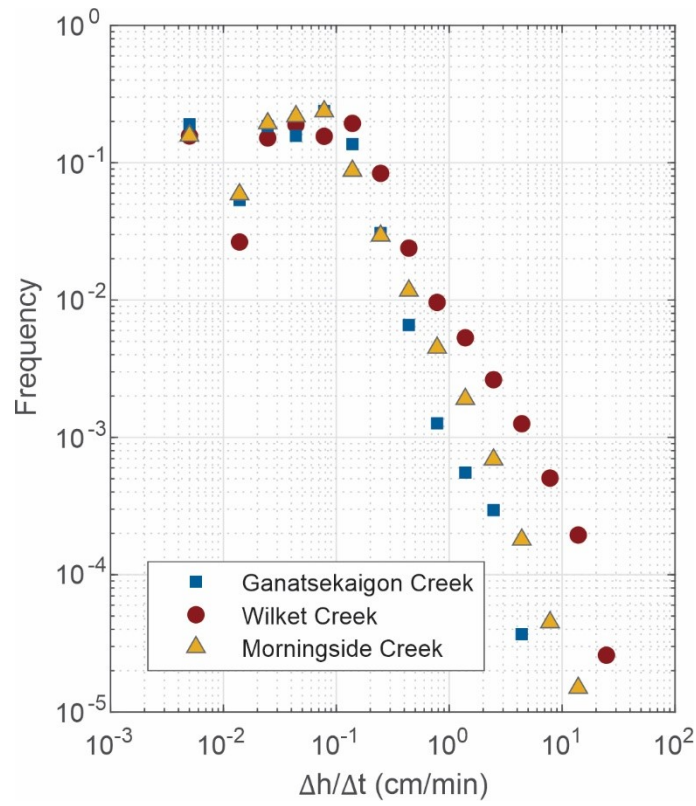


Figure 2.5 – Frequency of rate of water surface elevation change ($\Delta h/\Delta t$) for the three study watersheds.

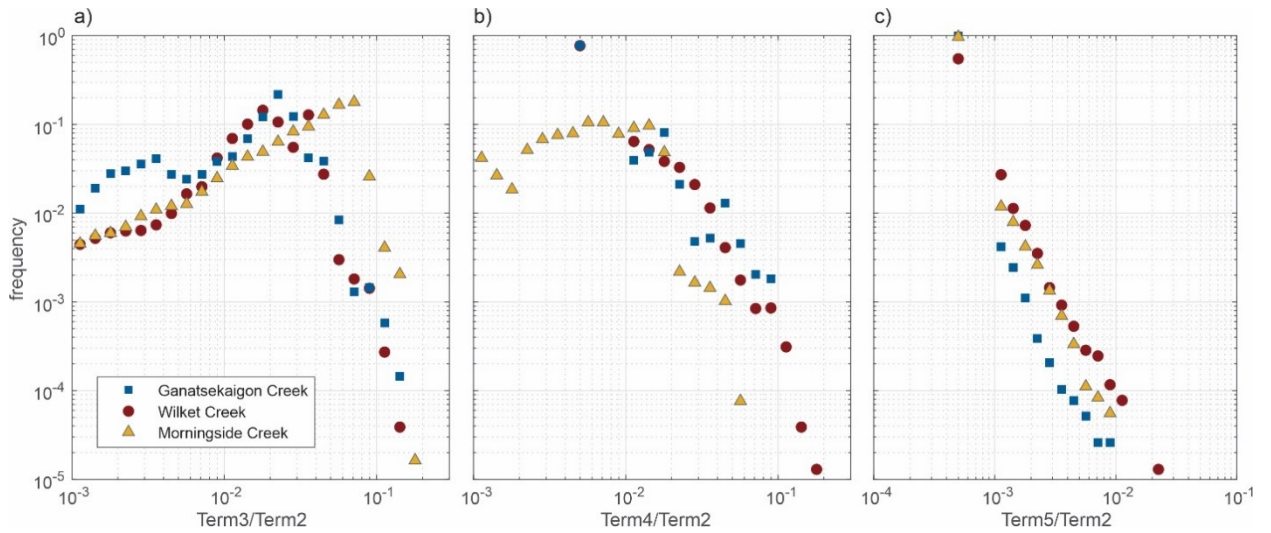


Figure 2.6 - Frequency of the relative weight of terms in the St Venant momentum function (Equation (2.2)) for each of the study watersheds including a) Term 3/Term2, b) Term4/Term2 c) Term5/Term2.

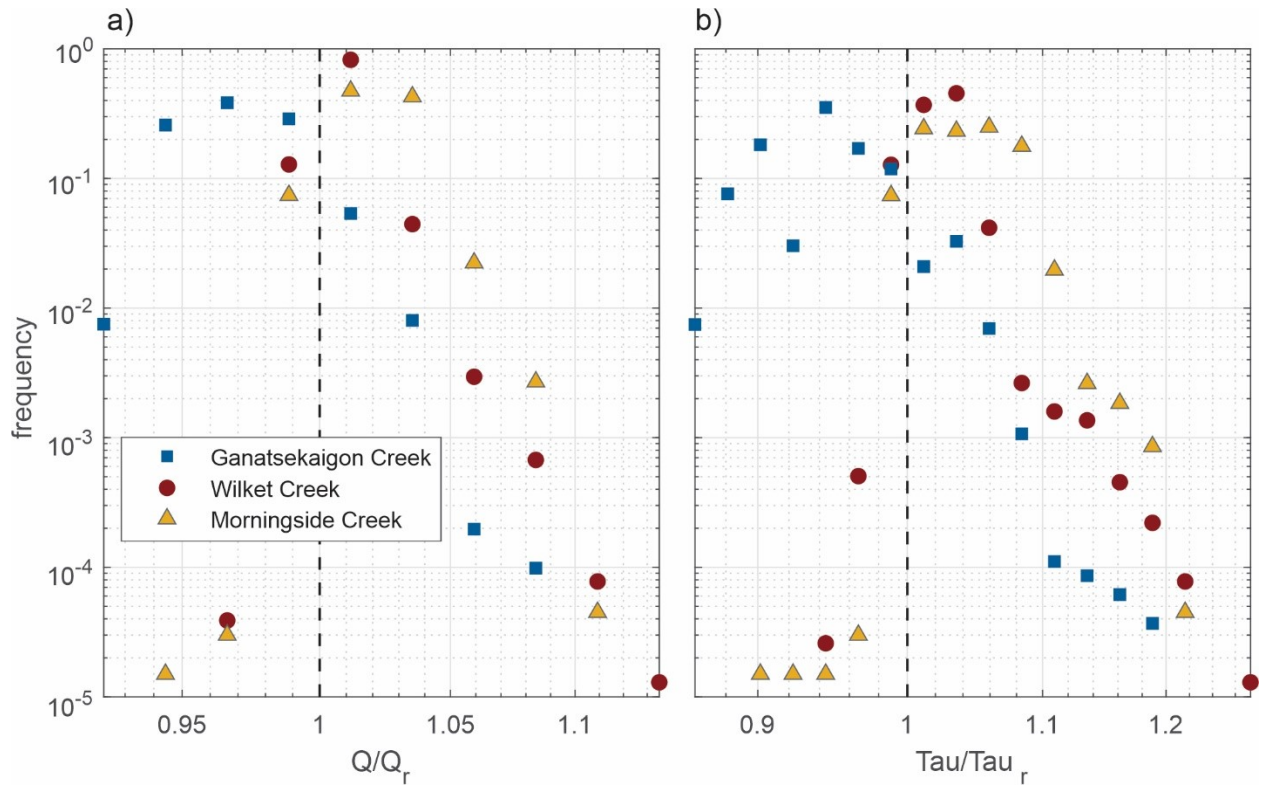


Figure 2.7– Comparison of complete St Venant Equations with steady state assumption f a) Q/Q_r and b) τ/τ_r where the subscript r indicates the steady state reference value.

Hydraulic variability between watersheds will result from differences in the rising and falling limb slopes and the unsteadiness of the hydrographs. To assess the significance of the unsteadiness on prediction of S_f (Equation (2.2)), we calculated the frequency of the ratios of terms 3, 4 and 5 in relation to term 2 for the full duration of the study (Figure 2.6). The terms are calculated using the data from two gauges so that all terms are calculated directly. The gauges are different distances apart in the different watersheds and S_0 varies slightly between the watersheds (Table 2.2), but some interesting trends between the watersheds are apparent. First is the relative importance of the terms. As shown, the ratios of terms 3/2 and 4/2 reach a maximum of approximately 0.20, while the ratio of terms 5/2 reaches a maximum of only approximately 0.02. As expected, the rural site has the smallest peak value for all three terms, but there is little difference in the overall frequency distributions between Ganatsekaigon and Wilket Creek for terms 3/2 or 4/2. Morningside Creek stands out as somewhat unusual for these term ratios, showing higher and lower frequencies for terms 3/2 and 4/2 respectively.

To assess the significance of the unsteadiness for the calculation of the key hydraulic parameters Q and τ Figure 2.7 was plotted. With this presentation, a value of 1 indicates that the steady flow assumption produces a result identical to that of the full St Venant equations. The figures are similar in shape if not in magnitude because $Q \propto \tau^2$, which means that the results are always more sensitive for shear than discharge. The mode of both distributions is less than 1 for Ganatsekaigon Creek, which indicates that Q and τ are likely somewhat less than the steady state values, though typically within 5%. In contrast, the mode of both distributions is greater than 1 for the urban creeks, which indicates that Q and τ are likely somewhat more than the steady state values, though again typically by less than 5%. The extreme values are somewhat more worrisome. In some cases, a steady state assumption can result in a 10-15% underestimation of Q and a 20-30% underestimation of τ for the urban rivers.

2.5 Discussion

The presented research is a case study of the hydrologic and hydraulic alteration of river flow that occurs as a result of urbanization. The results are valuable because of the selection of three neighbouring watersheds with similar size, geology, and slope to compare the cumulative impacts of urbanization and SWM. Of the two urban sites, one was developed during an era where the conveyance of surface runoff was maximized, while the second was developed with large SWM retention facilities and a diversion structure built to reduce peak flows in the downstream channel. A high frequency of water surface measurements at two locations in each channel allowed us to investigate the hydraulic assumptions for estimating the discharge and shear stress in these channels. The results are most significant because they show that steady-state assumptions are not valid for the urban watershed where no SWM is applied. A second significant result is that the stormwater management does not completely eliminate flashy urban hydrographs, even though the design for the watershed is sufficient to moderate the peak flow and flow volume. The limitations of the results for general application include the variable distances between the pairs of water level gauges in the different watersheds, and the particulars of the urban watershed with stormwater management, which reduces the generality of the results. Both of the limitations will be discussed as part of this section.

The hydrologic results from Wilket Creek (the urban site without SWM) are consistent with other urban sites where stormwater management has not been employed. Using the database compiled by Leopold (1968), for example, Hollis (1975) showed that the recurrence interval decreases as a function of

impervious cover and sewerage, with events of $100 \text{ ft}^3/\text{s}/\text{mi}^2$ (or $1.09 \text{ m}^3/\text{s}/\text{km}^2$) dropping from a 3–4 year to a 0.25 to 0.3 year recurrence interval for the levels of impervious cover and sewerage that is present in Wilket Creek. The frequency of this type of runoff events is approximately 0.5 per year (2-year recurrence interval) for the rural Ganatsekaigon site and ~ 6 per year (0.17 year RI) for Wilket Creek, indicating quite a close correspondence. Bell et al., (2016) compared peak discharge values across watersheds with a range of urban development conditions and found that less than 1% of flood events have $Q_{\text{peak}} > 1 \text{ m}^3/\text{s}/\text{km}^2$ in rural watersheds, but $\sim 10\%$ of flood events reach this runoff rate if the imperviousness $\sim 50\%$, as is the case for Wilket Creek. Annable et al. (2011) also shows that in comparison with a morphologic bankfull condition, the discharge reaches bankfull depth more often in urban rivers and that the hydrologically defined 2-year return period flow systematically exceeds this bankfull condition for urban channels.

In comparison with unmitigated urban runoff, there are far fewer studies that model the cumulative effect of SWM in urban watersheds. The results from the current study show that, contrary to what was observed by Bell et al. (2016), SWM mitigation efforts can have a dominant effect on watershed hydrology. By many measures, the SWM efforts in Morningside Creek appear to have brought the hydrologic response in line with the rural site. Both the peak discharges and the runoff depth distributions in Morningside, for example, are indistinguishable from those of Ganatsekaigon. A key difference between the current study and the work of Bell et al. (2016), however, is the degree of SWM measures that were employed. In Morningside watershed fully 79% of the land drainage is routed first through offline stormwater management ponds designed for peak discharge control (Raso, 2017). This value exceeds what was observed for the Bell et al. (2016) watersheds, where only one small area (DA 0.15 km^2) has more than 50% of its area that was mitigated by stormwater management structures. In addition to the offline ponds, flow from 74% of the upper watershed was moderated by a flow diversion structure designed to further limit flow peaks. The occurrence of this structure is an opportunistic use of geography due to the unique shape of the Morningside catchment such that the upper watershed is on a plateau that comes very close to the larger river. High flow diversions are not thought to be a widely applied SWM technique. The impact of this structure is most clearly seen in the hydrologic data with the runoff depth. Morningside is most similar to Ganatsekaigon with respect to the distribution of total runoff volume (Figure 2.3b). The peak flow in the watershed is also far less than what was observed in the uncontrolled urban and even the rural system (Table 2.3). However, none of the facilities constructed for the watershed are designed for infiltration. Likely some infiltration does occur in the

ponds, and the extended detention times mean that the water is stored in the upper areas of the watershed for longer such that plant uptake and evaporation may account for some reduction in storm runoff volume. Errors in the rating curve calculations and differences in rainfall between the watersheds (which are nonetheless in close proximity) may have also occurred. Nevertheless, it does appear that peak flows have been drastically reduced in Morningside Creek due to the diversion of overflows to the Rouge River. In addition to the offline and diversion structures, Raso (2017) found that beaver dam construction in the urban floodplains was also significant for routing. The wide corridor left as a part of the design of the urban development appears to be ideal habitat for these ecological engineers. Finally, nearly 100% of the flow was routed through an inline pond upstream of the study reach. This structure does not appear to have a large retention area, but the structure did still appear to have a significant impact on flow hydrographs in a modelling study (Raso, 2017). Papangelakis et al (2019) found that the morphology at the site has been characterized by channel narrowing and bed armouring such that little erosion and movement of coarse sediments occurs. These results thus support the idea that large wet ponds do have a cumulative impact on reducing the magnitude of urban peak flows. However, the extent of SWM control and the particulars of flow peak modification in this watershed mean that this is a regional case study that may be difficult to generalize to other watersheds.

The hydraulic impacts of urbanization are rarely considered in the literature outside of a few isolated studies. Costabile, Costanzo, De Lorenzo, & Macchione, (2020), for example, use a laboratory experiment and a flood of the city of Crotona in Italy on the Esaro River to show that fully hydrodynamic modelling should be the unavoidable tool when the goal is not limited to flood extent mapping but involves local estimate of flood hazard. Kvočka et al. (2015) similarly argue that more advanced modelling methods are necessary in steep catchments, in their case based on a study of a flash flood in the village of Boscastle in UK with post-flood ‘wrack marks’ on infrastructure to estimate the peak water depth. They show that comparable water surface elevations could be modelled using a steady hydraulic model by inflating roughness coefficients, but such approaches will induce high model errors in other flow conditions. They show that a dynamic shock capturing modelling approach was able to accurately model a range of flow conditions without significant error. For these cases, the interest is in reproducing flood extent. Such data is notoriously difficult to obtain, though attempts are being made to extract information from aerial surveys (Zhang and Crawford, 2020). In our case we are looking at using widely available gauge data to assess the necessity of advanced hydraulic methods.

In our case we did not require shock capturing methods as the flow remained subcritical in all cases. Results that are generated from the rate at which water level changes at a single location tend to emphasize the differences between the urban Wilket Creek and the rural Ganatsekaigon Creek sites. For example, Wilket has higher rising limb slopes on the hydrographs (Figure 2.4a) and instantaneous rates of water surface change (Figure 2.5) than the other sites. This results in higher acceleration terms ($\frac{\partial V}{\partial t}$) in Equation (2.2) such that a comparison of the ratio of Terms 5/2 also tends to stand out in comparison with the other sites (Figure 2.6c). However, when translated into error estimates for Q and τ , Wilket does not look exceptional in comparison with Morningside Creek (Figure 2.7). The explanation for this seeming discrepancy is that Morningside is exceptional compared to the other sites in the comparison of Term 3 (Figure 2.6a), which is a function of the changing slope of the water surface during a flood and calculated using data from two gauges. In Morningside Creek the gauges were 133 m apart (Table 2.2), which is higher than Ganatsekaigon (51 m), but much less than what was available for Wilket Creek (347 m). It is likely that the higher distance at Wilket Creek meant that Terms 3 and 4 are less sensitive to changes in water slope than at the other sites, meaning that the rising limb slopes of the waves are not fully being captured. Interestingly, it is only at the Wilket site that the maximum value for Term 4/2 is higher than that for Term 3/2. Despite the likely muted importance of Terms 3 and 4 on Wilket Creek due to the distance between the gauges, the analysis shows that errors of 4-10% for Q and 10-20% for τ occur frequently at the urban sites (Figure 2.7). Though rarer at the rural site, such errors are possible, indicating that the frequency of floods and high surface runoff events are likely the reason for the difference in hydraulics between the catchments. Fully hydrodynamic models are thus recommended for flood conditions in all three of these watersheds when assessing discharge and shear stress.

The current study is limited in scope because of the short term gauging records that were used to assess the hydraulics. Such records rely on continuous high-frequency sampling of water surface elevations, which are not generally available at gauging stations with longer records. Future work should also control for the distance between gauges between watersheds to better compare terms that are a function of the difference between measurements at two locations.

2.6 Conclusions

The following conclusions can be drawn from the results and discussion of this chapter:

1. The urbanization of a watershed leads to an increased frequency of flood events with a high peak discharge. In comparison with a rural watershed, the studied urban channel had 10 times more moderate events with peak magnitudes between $0.1 - 1 \text{ m}^3/\text{s}/\text{km}^2$, which is equivalent to 5-10 mm of direct runoff.
2. Stormwater management helps to reduce the frequency of moderate events such that their frequency distribution approximately matches that of a comparable rural watershed. The variety of the techniques used in the studied watershed and their application to a high percentage of the drainage area is likely the reason that the current study found a high effectiveness of the techniques in comparison with some previous studies.
3. The studied urban river was subject to higher flow unsteadiness than a comparable rural stream. This unsteadiness means that steady flow assumptions will lead to an underestimation of discharge of 10-15% and an underestimation of shear stress of up to 30%. Such calculation errors can also occur in rural systems but are less common by an order of magnitude. The rapid rising limb slope was the most significant factor in the flow unsteadiness calculations. However, the importance of the spatial gradients of velocity and depth were somewhat obscured by differences in the distances between the gauges at the different sites.
4. Despite the effectiveness of the SWM techniques as described in point 2, the SWM techniques did not affect the unsteadiness of the floods relative to the urban channel where no such SWM techniques were applied. The lack of difference between the two urban systems despite their radically different management of storm waters suggests that the areas that are directly connected to the channel are the most important for driving these rapid responses in both basins.

Chapter 3

Urban Unsteady Flow Routing

3.1 Introduction

Urban floods are characterized by shorter times to peak, higher peak flows, and rapid changes in flow depth and discharge compared to floods in rural catchments (Walsh, Roy, et al., 2005). While such rapid changes are known, kinematic routing methods with steady flow assumptions are usually considered in urban hydraulic models for floodplain delineation and risk management practices. Based on the data provided by the Government of Canada, the federal payments under disaster financial assistance arrangements for floods occurring since 1970 is approximately \$13.7 million annually (Government of Canada, 2013). These costly damages are evidence that current floodplain mapping and hazard assessment are insufficient. Damage could be reduced if a more accurate method of flow routing is used to delineate floodplains and assess flood hazard. In the current study, urban floods are routed with kinematic, diffusion, and dynamic methods to find the most appropriate method.

Over the last century, many studies have investigated simplified wave models to find the most appropriate routing method (Aricò, Nasello, & Tucciarelli, 2009; Tsai, 2003). Continuous measurement of velocity and/or discharge in natural rivers is challenging because it requires expensive instruments with human operators (for example when using Acoustic Doppler Current Profilers - ADCPs) or post processing analysis when using video feeds to support Large Scale Particle Image Velocity (LSPIV) estimates (De Serio & Mossa, 2015; Dramais, Le Coz, Camenen, & Hauet, 2011). Therefore, direct measurement of velocity and discharge is usually only realized at discrete points in time, and continuous monitoring is accomplished using water level sensors and rating curves. These rating curves typically assume a one-to-one relationship between water depth and discharge, which is only strictly accurate under the kinematic or quasi-steady flow assumption. Rating curves are also often poorly validated or extrapolated for higher discharges, where the rarity of the floods and hazardous sampling conditions may limit the accuracy or even the possibility of velocity and discharge measurements. Channel morphology and cross-section can also change over time and should be updated frequently to maintain accuracy (Burguete J, Garcia-Navarro P, Murillo J, Garcia-Palacin I., 2007). For all of these reasons, the estimation of discharge using rating curves comes with many uncertainties that may result in an inaccurate value.

An alternate strategy that has been implemented over the past couple of decades is to use two water level sensors placed some distance apart in a river to estimate discharge (Aricò et al., 2009; Muste et al., 2020). Such studies have used relatively simple methods to obtain a downstream rating curve from upstream and downstream measured water levels and a known upstream rating curve. For example, two studies used a “black box” model, which required using a known rating curve to calculate modelling parameters (Moramarco T, Barbetta S, Melone F, and Singh VP, 2005; Tayfur G and Moramarco T., 2008) while others have routed unsteady flows using the Muskingum method (Birkhead AL, and James CS, 1998). Although this method simplifies the computational process, it does not result in the most accurate results. Newer studies used a diffusive model and applied a more general numerical scheme than the Muskingum method (Franchini M. and Ravagnani F., 2007). One additional study used a diffusion model by considering the changes of water depth over distance in the momentum equation, but implemented a Muskingum numerical scheme (Perumal, Moramarco, Sahoo, & Barbetta, 2007). Another study used water level data from two sections of the river and found water depth at both locations and water level slope to calculate discharges (Arico C. Tuciarelli T. Dottori F. Martina M. Todini E., 2008). This latter research concluded that the two gauges must be far enough apart to reliably capture the changes in water level slope, but close enough to assume that lateral inflows between the gauges are negligible. In the current research a distance of approximately 100m was a sufficient distance for the creek characterization.

The goal of the current study was to assess the accuracy of routing schemes in an urban creek characterized by floods with high flood magnitudes and rapid rising limbs. Two pressure gauges were installed in the creek and high frequency (15 s) water level measurements were used to calculate discharge as a function of i) only depth (kinematic method) ii) depth and slope (diffusion method) iii) depth, slope and acceleration terms (dynamic method). The objectives of the study are to a) compare the methods and b) assess the importance of measurement frequency in the accuracy of the results. The results focus on two hydrographs with a high peak that exhibit the characteristic traits of a rapid increase of water level and discharge during the rising limb. Surface velocities were measured during the flood using Surface Velocity Radar (SVR) and LSPIV techniques to help validate the results.

3.2 Methods

3.2.1 Site Description

The study site was Wilket Creek, a highly urbanized watershed in Toronto, Ontario. The watershed area is 15.5 km² with 96% developed areas and 47% imperviousness. The upper two-thirds of the watershed is drained by underground sewers with no significant storm water management facilities (Bevan et al., 2018). As shown in Figure 3.1 an open channel main stem approximately 6 km long remains in the lower third of the watershed. Hydrographs observed in this creek are characterized by short times of rise and high peaks, and a high level of flow unsteadiness in comparison with other watersheds of similar size, shape, and geology, as shown in the previous chapter.

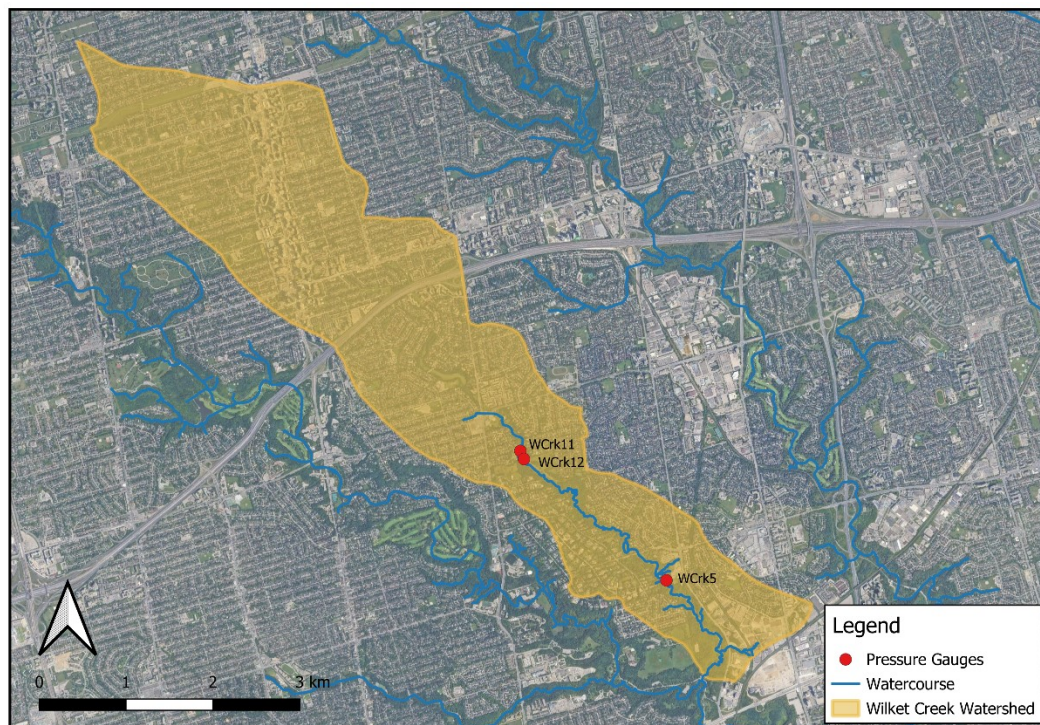


Figure 3.1 Location of Wilket Creek watershed with respect to the City of Toronto. The blue line shows Wilket Creek. The two points are showing the location of the two gauges on Wilket Creek.

3.2.2 Surface water elevation measurements

Water surface elevation was measured at two locations in Wilket Creek, and they are 134.6m apart as shown in Figure 3.1. Pressure gauge characteristics are listed in Table 3.1. The gauges were installed in stilling wells and hung so that the gauge was continuously below the water level. The gauges were

located in a reach with low sinuosity (defined as the length of the river channel divided by the length of the valley) and positioned so that no urban structures such as bridges or weirs were located between them. The main gauge (WCrk11) was located at a cross section where the rating curve was established while the backup gauge (WCrk12) was used to calculate water surface slope. The gauges were installed in October 2018 and the data was collected until November 2019. They were set to measure pressure every 15 seconds. With this frequency, the gauges only held three days of data due to storage limitations. Because of this limitation the gauging record was discontinuous, but efforts were made to go to the creek the day before each forecasted rainfall event to reset the gauge. A gauge installed far downstream (WCrk5) as part of an earlier study was also maintained to measure continuous data at a lower frequency (15-minute time interval) over our study duration. Cross sections for the main and backup gauges are shown in Figure 3.2, with the main section somewhat narrower and deeper than the backup section. The bank of the creek is protected using riprap (large boulders) upstream of the study reach. Bank erosion is observed just downstream of the protected section. A third pressure gauge was installed on the underside of a nearby pedestrian bridge above the water level to allow water level estimates to be corrected for barometric pressure fluctuations. The channel survey and water depth estimates were used to calculate flow depth and water surface elevation at each cross section using a custom Matlab program called FlowTrack.

Table 3.1 Characteristics of the two pressure gauges installed in Wilket Creek with high frequencies.

Gauges name	Sections (Main or backup section)	Location	Channel slope	Centerline distance between the two gauges (m)	Sinuosity	Sediment Size (m) D84
WCrk11	Main	Upstream	0.0084	134.632	1.12	0.22
WCrk12	Backup	Downstream				0.08

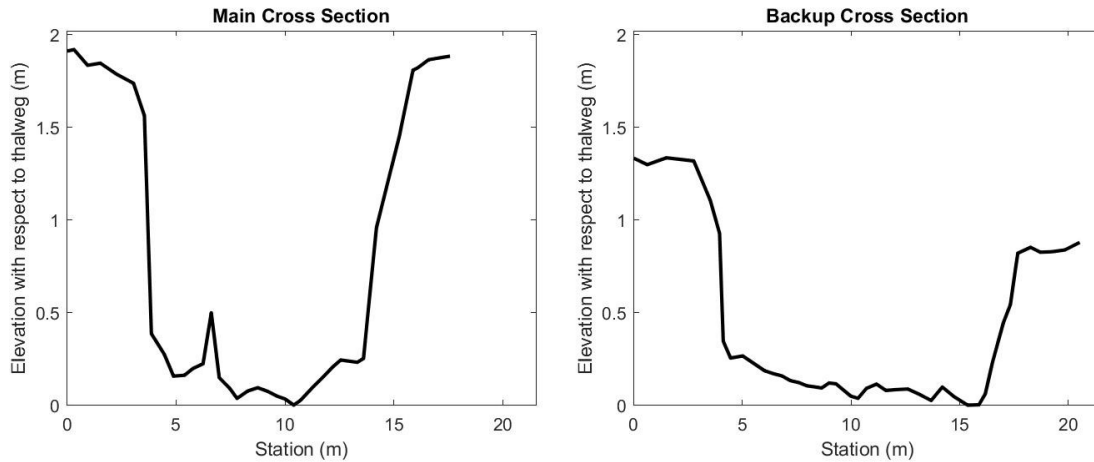


Figure 3.2 The cross section of the location of the two pressure gauges in Wilket Creek.

Water elevation data was generally of good quality but some errors were noted due to occasional spikes and seasonal errors. Seasonal errors are related to the effects of ice cover, which was eliminated from our data to ignore these effects. Spikes were sometimes observed when data was downloaded due to the need to physically remove and replace the gauges to download the data. Such errors were removed manually. Step changes were sometimes observed coincident with the time when data was downloaded from the loggers, which were thought to be related to slight positioning errors of the gauges during replacement. These errors were eliminated by manually removing a step change so that the post-download water surface elevation matched the pre-download elevation. Certain periods within the data appeared to have higher errors. The data was not collected during the winter since the data is not reliable due to the impact of ice on the pressure data. For this reason, the data was collected between the months of April and November of each year.

3.2.3 Velocity Measurement Methods

3.2.3.1 Surface Velocity Radar (SVR)

An SVR radar gun manufactured by Decatur Electronics, (2005) was used to measure velocity of the water surface during monitored flood following methods tested by Welber et al. (2016). The gun internally compensates for both vertical and horizontal angles and provides an estimate of water surface velocity. Both angles are shown in Figure 3.3, in which the horizontal angle is the angle between the flow direction and the direction of radar gun. The vertical angle is also shown as the vertical angle of

the radar gun with respect to the horizon. The horizontal angle should be relatively high (minimum 10°) to ensure accurate readings. For the current study, a horizontal angle of 50° was used (maximum possible due to trees on the banks of the creek) and markers were installed to ensure that the measurements were consistent during and between floods (Figure 3.3). A tripod was used to ensure that angles were consistent for the duration of the velocity measurement.

For the current study, the surface velocity measurements were made during flood events, and often during rainfall, which can affect the accuracy of the surface velocity measurements. For the low flows, rain drops are faster than flow discharge, which can lead to false velocity measurements. However, for high flows the effect of rain drops is eliminated and the measurements are reliable because the radar detects the fastest motion in the range of view (Decatur Electronics, 2005).

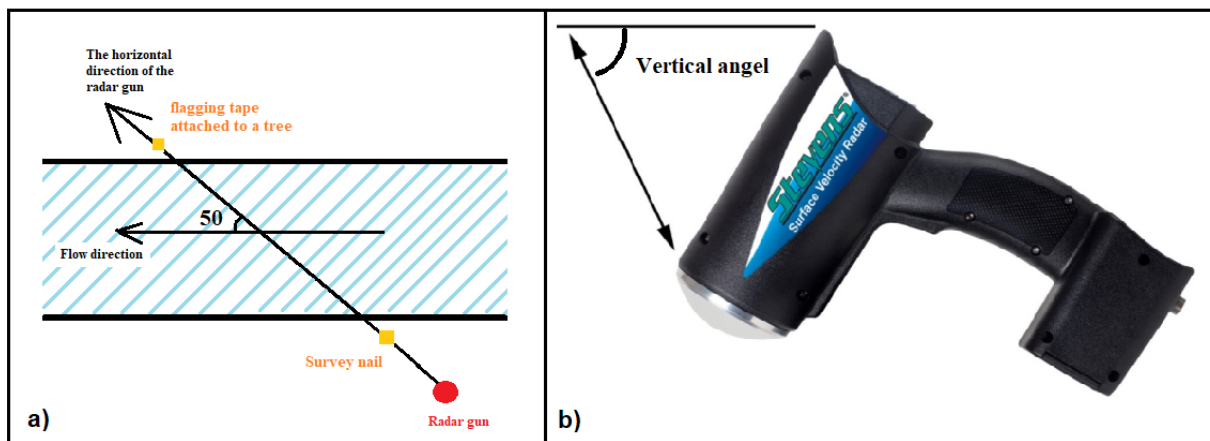


Figure 3.3 a) The location of the radar gun with respect to the flow direction. The horizontal angle of the gun. b) the vertical angle of the instrument.

The procedure for the velocity measurements, used a 60 second velocity measurement duration. In the first 10 seconds the SVR measures and reports the velocity every second. Between 10 and 60 seconds the instrument reports a velocity every 5 seconds for a total of 10 velocity measurements. For this study, we used the 5 second average measurements in our results.

3.2.3.2 PIVlab

The second method of velocity measurement is the most practical method to measure velocity during high flows and rain events. PIVlab is a particle image velocimetry (PIV) software with a Graphical User Interface (GUI), which calculates the velocity distribution of the particle image pairs (Thielicke

& Sonntag, 2021). This software is MATLAB based and it was used to find the surface velocity of a flood in this chapter. The software is user-friendly, and the output graphs can be extracted from the software easily. The average velocity component value was plotted over the entire study cross section. The average value for the cross section was outputted to plot on the rating curve. This value is the average surface velocity at the cross section which is multiplied by 0.85 to be converted to a mean velocity at the cross section based on a standard assumption for open channel flows (Dramais et al., 2011).

The videos were taken using a GoPro camera that was attached to a tree branch on the left bank. The camera was located inside a waterproof case, which made it possible to record videos under rainfall. The videos were recorded at 30 frames per second with separate durations of 60 seconds to be properly synced with the velocity measurements using SVR method.

3.2.4 Discharge Calculation methods

The Saint Venant equations require discharge and velocity measurements to be solved directly. However, velocity measurements in rivers data are typically made only intermittently and used rather to calibrate a $y - Q$ rating curve for a given gauging station location. The most widely accessible field data is water level data recorded over time using a pressure or float gauge, from which V and Q can be estimated using a flow resistance equation. The St-Venant equations have therefore been modified to allow them to be solved using water level data from either one or two water level gauges. Four methods that were used to apply diffusion and dynamic routing using water level measurements in rivers are presented here.

3.2.4.1 Discharge as a Function of Depth and Slope with One Gauge (J1)

This diffusion analytical method, referred to as “J1” in the current study, models discharge as a function of stage and water surface slope using the Jones, (1915) formula as described by Henderson, (1966). The momentum equation with reduced terms and continuity equation are combined and changes with respect to distance (x) are replaced with changes with respect to time (t) to reflect the nature of the available data so that:

$$Q = Q_r \sqrt{1 + \frac{1}{S_0 c} \frac{\Delta y}{\Delta t} + \frac{2}{3r^2}} \quad (3.1)$$

where Q_r is reference or normal discharge for a given stage (calculated using a flow resistance equation), c is the kinematic wave celerity ($= KV = K Q/A$, where V is the average velocity and A is the average area), and r is the ratio of channel bed slope to the average wave slope, written as (Lee, 2013):

$$r = \frac{43200(Q_p + Q_0)}{(y_p - y_0)\bar{A}} \Gamma S_0 K \quad (3.2)$$

where Q_p is the peak discharge, Q_0 is the flow discharge prior to the start of the floodwave, y_p is the peak flow depth, y_0 is the flow depth prior to the start of the floodwave, $\bar{A}$ is the average wetted area corresponding to the average stage, Γ is the time interval measured in days, and K is the wave celerity coefficient, written as:

$$K = \frac{5}{3} - \frac{2}{3} \frac{A \Delta P}{P \Delta A} \quad (3.3)$$

where P is the wetted perimeter. In this method, c is calculated using the equation $K Q/A$. Having the value of discharge on both sides of equation (3.1), leads into an iterative solution for discharge. The code finds the discharge value for the right side of the equation and the discharge value for the left side of the equation and it compares the two values with each other. It iterates the values until the two values for both sides of the equation match. For the changes over time in Equation (2.2) we used a forward scheme to capture the changes of flow parameters over time (Chaudhry, 2008). The equation is shown below.

$$\left. \frac{df}{dt} \right|^{t=t_0+\Delta t} = \frac{f(t_0 + \Delta t) - f(t_0)}{\Delta t} \quad (3.4)$$

where f can be any dependent variable in the Saint Venant equation such as Q or A , t_0 is the time step for which f is known, and $t_0 + \Delta t$ is the time step for which f is calculated.

3.2.4.2 Discharge as a Function of Depth and Slope with Two Gauges (J2)

This second diffusion routing method, referred to as “J2” in the current study, requires water level data from a second gauge so that Term 3 in the momentum equation can be represented by more directly

using the measured change in depth over a distance. As described by Chaudhry, (1993), the combined continuity and momentum expression for Q can then be written as:

$$Q = Q_r \sqrt{1 - \frac{1}{S_0(A)} \frac{\Delta(A\bar{y})}{\Delta x}} \quad (3.5)$$

where $\bar{y}$ is the distance between centroid of the channel cross section and the water surface.

For solving the equation above, the term $\Delta(A\bar{y})/\Delta x$ is solved using a numerical method. For the changes over distance a first-order accurate forward scheme was used as described by Chaudhry, (2008). This method was selected over a central difference method because only two gauges were installed in each creek, while the central difference requires three data points in streamline direction. The equation for this scheme is shown below.

$$\left. \frac{df}{dx} \right|_{x=x_0} = \frac{f(x_0 + \Delta x) - f(x_0)}{\Delta x} \quad (3.6)$$

and x is the distance in streamflow direction. x_0 is the location of the upstream cross section, and $x_0 + \Delta x$ is the location of the downstream cross section.

3.2.4.3 Discharge as a Function of Depth, Slope, and Acceleration Terms with one Gauge (F1)

In this dynamic method, referred to as “F1” in the current study, all the terms in the momentum equation are represented, but the measured data is assumed to come from a single point where depth is recorded. This method is similar to J1 in that a time-for-space substitution is required. A typical approach for this type of modelling, called the modified Fread method (Lee, 2013) writes the expression for Q as follows:

$$Q = Q_r \sqrt{S_f / S_0} \quad (3.7)$$

$$S_f = S_0 + \frac{A}{KQ} \frac{\Delta y}{\Delta t} + \left(1 - \frac{1}{K}\right) \frac{Q}{gA^2} \frac{\Delta A}{\Delta t} + \frac{1}{g} \frac{\Delta(Q/A)}{\Delta t} + \frac{2S_0}{3r^2} \left(1 - \frac{BQ^2}{gA^3}\right) \quad (3.8)$$

Since discharge is available on both sides of the equation an iterative method is used to solve the equation (3.8). The numerical method is not used here because there are no changes of flow properties over distance.

3.2.4.4 Discharge as a Function of Depth, Slope, and Acceleration Terms with two Gauges (F2)

This dynamic method, referred to as “F2” in the current study, again uses all terms in the momentum equation but utilizes water depth measurements from two locations in the same channel (one upstream and one downstream) similar to the J2 method. As described by Chaudhry, (1993), the combined continuity and momentum expression for Q can be written as:

$$Q = Q_r \sqrt{1 - \frac{1}{S_0} \left(\frac{1}{A} \frac{\Delta(A\bar{y})}{\Delta x} + \frac{V}{g} \frac{\Delta V}{\Delta x} + \frac{1}{g} \frac{\Delta V}{\Delta t} \right)} \quad (3.9)$$

This equation is written as below to eliminate the velocity term and to be solved mathematically.

$$Q = Q_r \sqrt{1 - \frac{1}{S_0} \left(\frac{1}{A} \frac{\Delta(A\bar{y})}{\Delta x} + \frac{Q}{Ag} \frac{\Delta(Q/A)}{\Delta x} + \frac{1}{g} \frac{\Delta(Q/A)}{\Delta t} \right)} \quad (3.10)$$

To solve this equation the numerical method explained in section 3.2.4.1 and 3.2.4.2 is used to find the terms with flow parameter changes over distance. In addition, since both sides of the equation contains discharge, an iterative method is also used to solve the equation.

3.2.5 Conservation of Mass

To ensure that mass is conserved in all methodologies, the left side of the continuity equation (Equation (2.1)) must be zero. The right side in this equation indicates inflow or outflow in the study reach. For mass conservation, this parameter was calculated by using two adjacent gauges and calculating the left side of the continuity equation for all the three creeks. If this parameter has a positive value, it shows that there might be a lateral inflow into our creeks. If this value is negative, it shows that there might be a lateral outflow in the system. The mean value calculated for Wilket Creek is nearly zero, which means that the mass conservation law was fulfilled for most of the study period in this watershed and the routing method used in this research is valid.

3.3 Results

3.3.1 Characteristics of the study water year

For the duration of the study period, a pressure gauge located far downstream of our study site (WCrk 5) was set to measure pressure continuously. For the current study, data from this gauge was used to assess the flood characteristics in the study period and compare with the data from the same gauge collected from 2015 to 2018 (Papangelakis et al., 2019) (Table 3.2). This earlier study was focused only on events that were relevant for sediment transport, so a higher shear stress threshold was used to isolate only events where sediment was likely to be mobile. Using hydrologic inflection points to identify the start of a flood resulted in a calculated frequency of ~80-90 events per year for both the previous (2015-2018) and current (2018-2019) period. The inflection points are the points where the graph changes from convex to concave or vice versa. As shown from previous results, this frequency is high relative to a rural site (Ganatsekaigon) and a site with stormwater management (Morningside). The events in the current period were on average longer than the previous period, and the peak discharge was only about half of the largest flood observed in the previous period, indicating that it was not an exceptional flooding period.

3.3.2 Characteristics of selected events

A set of hydrographs from significant floods shown on Figure 3.4 were extracted from the available record for analysis. The four largest events, each of which was greater than or close to 20 m³/s occurred on July 15th, October 20th, June 20th, and July 17th. Despite their similar peaks, the shape of the hydrographs varied widely such that some were isolated events that occurred over 1-2 hours (e.g. July 13th) while others were characterized by multiple peaks over 10 or more hours (e.g. July 17th). The overlapping of multiple events within a single extracted event shows that the urban events can be characterized by multiple rapidly rising peaks spread out over the duration of a storm, which gives rise to a sawtooth hydrograph shape.

Table 3.2 The characteristics of continuous data for the study duration at a gauge located downstream of the study site.

Site	Study	Period (Dates)	Events/year	Mean duration (h)	Mean Q_{peak} (m^3/s)	Mean τ_{peak} (Pa)	Max Q_{peak} (m^3/s)	Max τ_{peak} (Pa)
Wilket	Current	2018-2019	80	27.4	4.6	33.1	19.5	69.8
	Papangelakis et al (2019)	2015-2018*	30	2.1	9.5	45.2	39.7	103.7
	Chapter 2	2015-2018**	88	17.5	4.9	40.3	39.8	113.1
Ganatsikaigion	Chapter 2	2015-2018**	34	42.7	1.8	23.9	29.8	78.2
Morningside	Chapter 2	2015-2018**	63	33.6	1.5	27.3	8.7	58.7

* adapted from Papangelakis et al., 2019. Note that this study used a higher event threshold based on sediment mobility rather than hydrologic inflection points.

** The data that was collected by previous students and it was used in the previous chapter of this thesis. The results are calculated using the method we used in this thesis

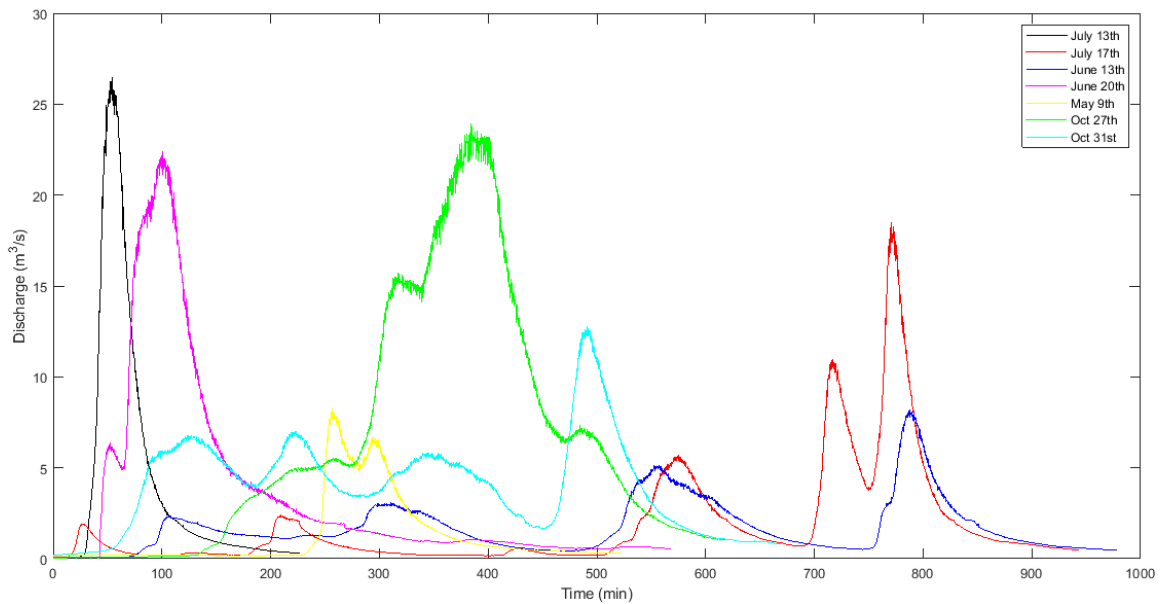


Figure 3.4 All the hydrographs plotted on top of each other. Time is measured from the beginning of the event and discharge is measured relative to baseflow

Extracted hydraulic metrics of the floods shown in Figure 3.4 are listed in Table 3.3. As shown, the July 13th event had the highest discharge and shear stress and the shortest duration, while the June 20th event had the fastest instantaneous rising rate (Maximum dh/dt), but a longer duration. Given that the focus was on the flow unsteadiness, we were also very interested by the difference between the maximum and minimum values in $Sf/S0-1$. The higher this difference the wider the hysteresis loop of the rating curve and the more the flow can be considered unsteady. The floods are sorted based on this difference in Table 3.3, and we use this metric as a proxy for flow unsteadiness.

Table 3.3 Flow characteristics for all the 7 hydrographs under study in the current chapter. High values in each column are bolded while low values are italicized. Events selected for detailed analysis in this chapter are indicated with an X.

Parameters	Maximum Discharge	Hydrograph Duration	Volume under hydrograph	Maximum dh/dt	Maximum Shear Stress (F2)	Maximum stream power (F2)	$\Delta(Sf/S0-1)$
Date of the hydrographs							
Units	(m^3/s)	(minutes)	(m^3)	(cm/min)	(N/m^2)	($N/(m.sec)$)	(---)
July 13 th (X)	26.5	227	61,520	24.4	73.5	32.5	0.45
July 17 th	18.5	944	106,521	14.4	59.2	22.4	0.41
June 20 th (X)	22.4	569	116,527	30.8	74.1	28.9	0.30
October 27 th	23.9	618	234,673	19.9	75.2	29.1	0.24
October 31 st	12.8	671	131,396	11.5	62.3	17.2	0.18
May 9 th	8.3	522	28,474	11.6	52.3	12.2	0.12
June 13 th	8.19	978	105,338	9.0	50.6	11.3	0.10

Two events (July 13th and June 20th) were selected for in-depth analysis because they had some of the most extreme values of peak discharge, shear stress, stream power, and instantaneous rising limb slope. The other events were not analyzed in detail, but a similar procedure for assessing flood hydraulics was followed and additional figures are presented in Appendix A. The July 13th event is representative of an intense isolated flood event where the flood rises to a discharge of 26.5 m^3/s in 26 minutes before falling to a value of 3 m^3/s in 45 minutes. The June 20th event was also relatively isolated and showed

only one peak, but several surges occurred that show as bumps on the hydrograph, which was common for other hydrographs observed for this urban channel.

3.3.3 July 13th 2019 event analysis

The key factor that affects flow unsteadiness is the rate at which flow depth changes with time (dh/dt). To begin the analysis we therefore plotted $\Delta h/\Delta t$ for the main gauge for all 15 second time intervals ($n = 908$) during the flood (Figure 3.5). As shown, the highest rate of change of depth is 24.4 cm/min, which occurs just once during the flood but demonstrates the rapid rise of the flood. Approximately 28 percent of data showed a rate of change in depth between 0 and 0.1 cm/min. 24% of data show a rate of change between 0.4 and 0.5 cm/min.

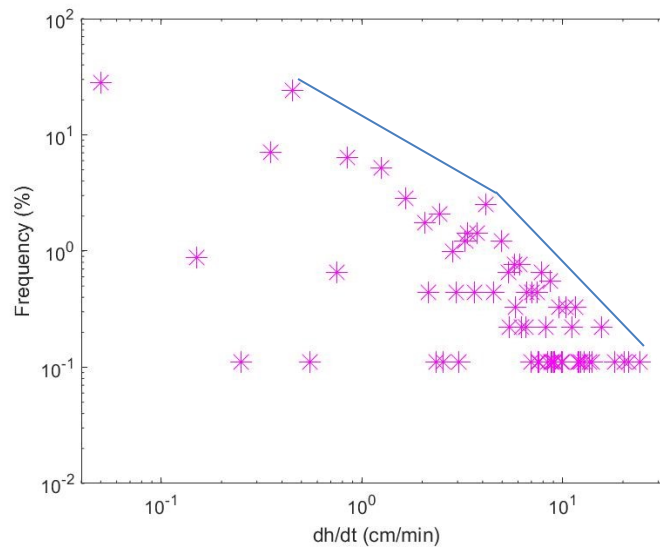


Figure 3.5 The frequency of rate of change of flow depth over the entire duration of the hydrograph.

The other key factor that can drive flow unsteadiness in the shallow water equations are differences between the water surface slope (S_w) and the channel bottom slope (S_0). Typically, S_w is not measured during floods, but our experimental strategy allows us to demonstrate that the relationship between S_w and S_0 changes as a function of surface water elevation (SWE) and whether it is rising or falling (Figure 3.6). Plotted as a ratio, S_w/S_0 is about 0.5 at the start of the event but shows a pronounced hysteresis, increasing to 0.65 during the rising limb of the flood and dropping to ~ 0.30 during the falling limb.

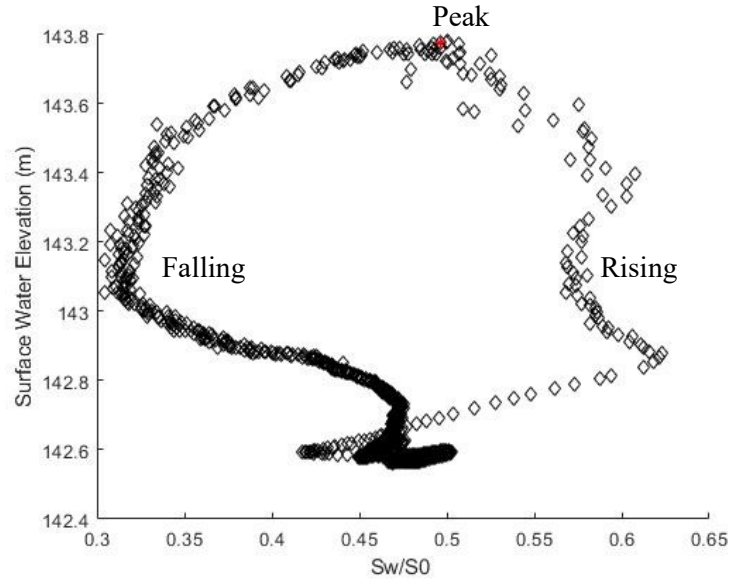


Figure 3.6 Surface water slope versus surface water elevation.

Figure 3.7 shows how the terms in Saint-Venant momentum equation (Equation (2.2)) vary over a hydrograph. The terms are expressed as a ratio over S_0 . Term 3 is $-\frac{\Delta y}{\Delta x}$ and calculated using the measured water surface elevations between the two gauges. Term 4 is $-\frac{V}{g} \frac{\partial V}{\partial x}$, which is calculated using the calculated discharge at both cross sections. The velocity is calculated as discharge over area for each measured surface water elevation. Term 5 is $-\frac{1}{g} \frac{\partial V}{\partial t}$, which is calculated using the calculated velocity (discharge over area) at the main cross section. Terms 3 and 4 (referred to herein as T3 and T4), both of which are functions of streamwise hydraulic gradient, rise during the rising limb where S_w is increasing relative to S_0 . The increase in T3 and T4 is compensated for somewhat by a decrease in Term 5 (T5) due to the increasing discharge at the main gauging station (e.g. between $t=20$ to 30 min), but the sum of these three terms will be positive, which shows that the actual discharge and shear stress will exceed the steady state values. As expected, all three terms are ~ 0 at the peak of the flood, which indicates that the peak discharge is not as significantly affected by the unsteady nature of the flood as earlier in the rising limb. On the falling limb, the hysteresis exhibited by S_w/S_0 means that S_f becomes equal to S_0 just before Q reaches its peak value. The trend for all three terms changes during falling limb. T3 and T4 decrease to a low value with respect to channel bottom slope, while T5 increases slightly during the falling limb. The data range for T5/T2 is only between 0.05 to -0.05 for this

hydrograph and short frequency noise tends to obscure the trends over the flood. However, it is apparent that the lowest and highest values happen during the rising and flooding limbs, respectively, which tends to somewhat counteract the trends in the other two terms of the momentum equation..

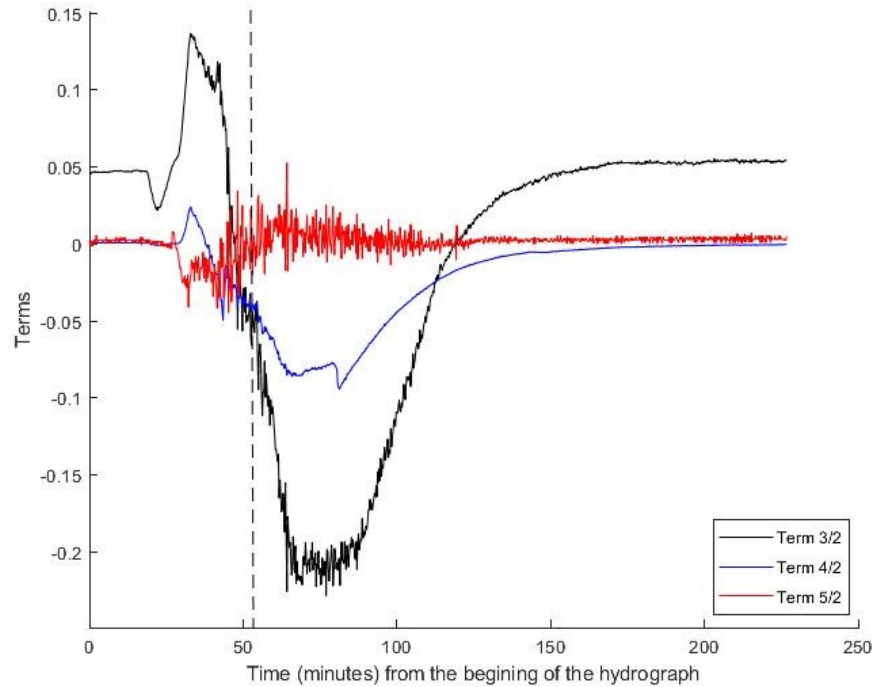


Figure 3.7 The terms of Saint-Venant equation with respect to Term 2 (S_0). The dashed line corresponds with the time of the peak discharge.

Figure 3.8 shows the net effects of the unsteady terms in the Saint-Venant equation on S_f over the flow hydrograph. As shown, $S_f/S_0 - 1$ rises at the beginning of the rising limb and reaches a maximum early in the rising limb. It then decreases nearly linearly through the peak of the flood and reaches a minimum negative value at the inflection point of the hydrograph (point 3). $S_f/S_0 - 1$ becomes zero somewhere between point 2 and the peak flow discharge. After point 3, $S_f/S_0 - 1$ remains low and negative for much of the falling limb before recovering towards a small positive value on the falling limb. Shear stress is proportional to this parameter and discharge is proportional to the square root of this value. Therefore, the level of error in this graph also shows the level of error in shear stress and discharge. As shown in this figure, the maximum value of this parameter is 0.15 (15%) and -0.3 (-30%) over the entire hydrograph. This shows that error in the range of 15% and -30% occurs for shear stress. The discharge error is between $(0.15^2=0.0225=2.25\%)$ and $(-(0.3^2)=-0.09=-9\%)$ for this hydrograph.

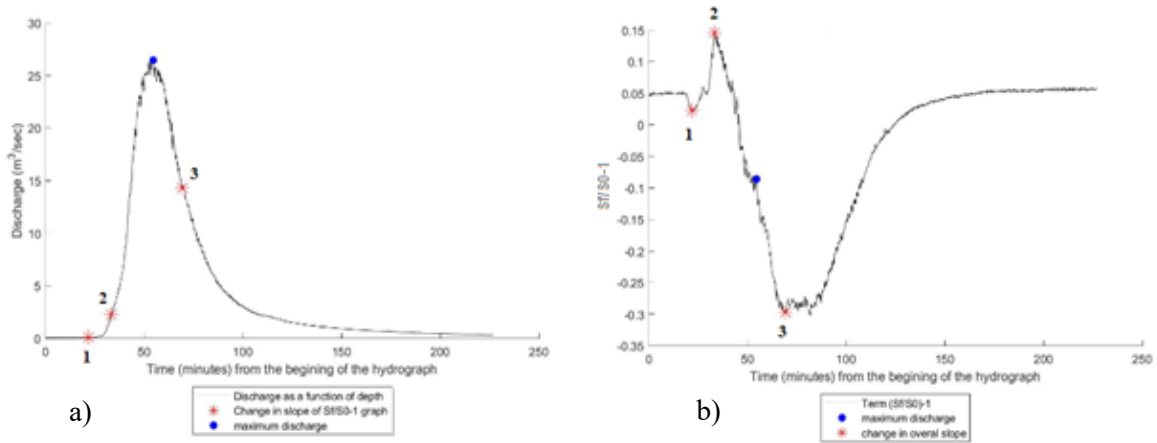


Figure 3.8 Variation of a) Sf/S_0-1 and b) Q for July 13th 2019 event. Inflection points in Sf and the time of the peak discharge are highlighted on a and b to facilitate the comparison.

The unsteady terms also have a pronounced effect on the estimation of Q . Using data from 1 gauge, for example, the relative discharge $\frac{Q-Q_r}{Q} * 100$, where Q_r is the steady discharge calculated using Manning's equation) rises to between 5-10% error on the rising limb and is nearly zero for the falling limb (Figure 3.9). The acceleration term appears to make little difference, and most of the error can be captured by modelling Q as a function of SWE and S_w . The relative discharge for the two methods using data from 2 gauges show a similar rise in relative discharge for rising limb, but also shows a fall to -12 and -18% during the falling limb (see J2 and F2 on Figure 3.9). The inclusion of the acceleration term thus significantly decreases the relative discharge for the 2-gauge calculations. When plotted as a rating curve it can be seen that the unsteady discharge for the discharge calculated as a function of depth, slope and acceleration terms is higher than the steady Q for the rising limb and lower on the falling limb (Figure 3.10). Therefore, we can see that this curve in Figure 3.10 stays on the right side of the rating curve for steady discharge (blue asterisks points). The horizontal width of the loop in the rating curve shows the difference in discharge values at the same water depth during the rising and falling limbs .

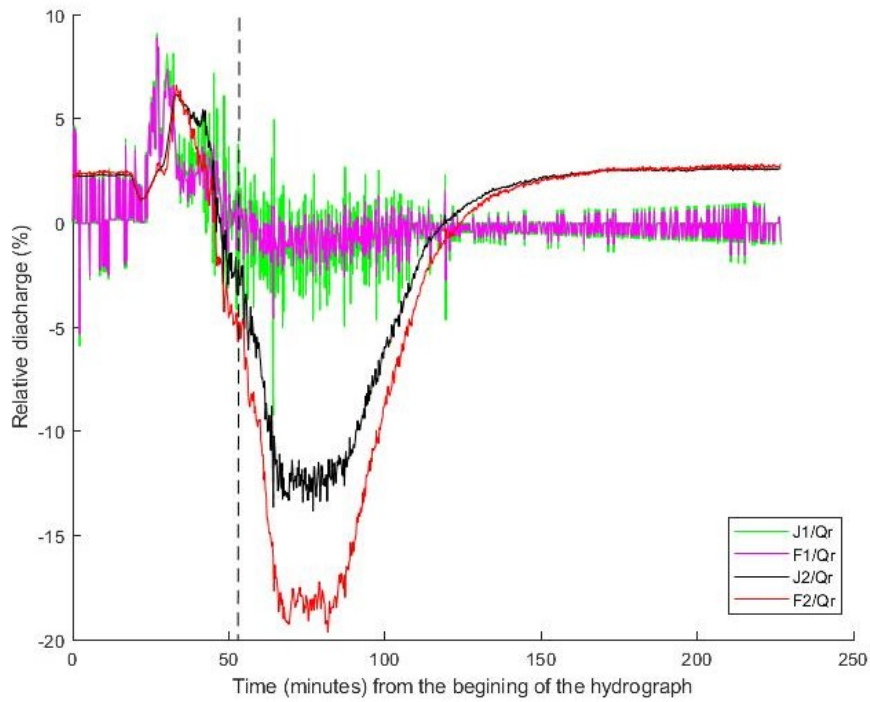


Figure 3.9 Relative discharge for the hydrograph occurred on July 13th 2019. For the positive values on this graph, the unsteady discharge is higher than the steady discharge and vice versa.

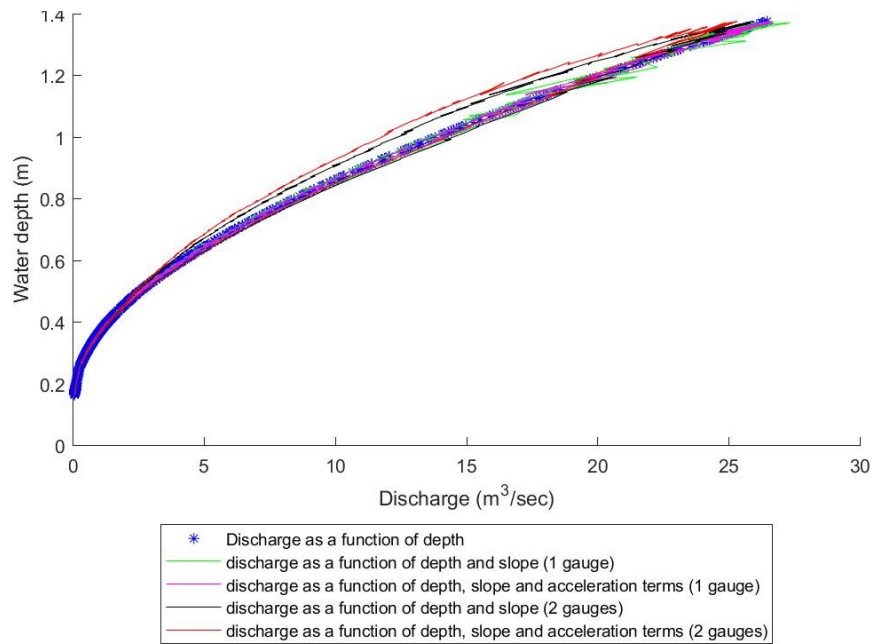


Figure 3.10 Rating curve for the hydrograph observed on July 13th 2019.

3.3.3.1 Shear Stress and Stream Power

Table 3.4 shows the values for stream power and shear stress for the rising limb, falling limb and the values for the entire hydrograph. These values are shown in percentage with respect to its value using a steady discharge assumption. This percentage is calculated using the equation below.

$$\frac{(SS_{(sf)} - SS_{(s0)})}{SS_{(s0)}} * 100 \quad (3.11)$$

In this equation, SS is either shear stress or stream power, $SS_{(sf)}$ is the shear stress or stream power calculated using one of the four methods that is used in the current study. $SS_{(s0)}$ is the shear stress that was calculated using steady flow discharge formula (Manning's equation).

For the shear stress it can be seen that the values calculated for all the four methods are positive for the rising limb and they are negative for the falling limb. The positive values indicate that the shear stress found using S_f is higher than the shear stress calculated using S_0 . However, this is not the case for the falling limb in this hydrograph. The shear stress found using S_f for the falling limb is much smaller than the shear stress using S_0 . This means that S_0 underestimates shear stress on the rising limb and overestimates it on the falling limb. Over the entire hydrograph S_0 overestimates the shear stress. In addition, using the F2 method shear stress value is lower than other methods for rising limb. This trend can also be seen for stream power. The reason is related to the timing of the balance point on the hydrograph, where the predicted discharge from the unsteady method is equal to the prediction using the average bed slope. As shown on Figure 3.9, this balance point occurs before the peak of the flood for the two-gauge methods. The strong negative trend through the peak of the flood means that the net for the rising limb is relatively low, despite the fact that the maximum relative discharge is of similar magnitude for all four methods.

Stream power also shows a similar trend where its values are mostly positive for the rising limb and negative for the falling limb. The stream power found using the data from 1 gauge (J1 and F1) has values of nearly zero for the total duration of the hydrograph. However, this value is positive for the rising limb and negative for the falling limb. This shows that the unsteady stream power is different from its steady value for rising and falling limbs, but they are the same if we only look at their values for the entire hydrograph. This shows the importance of calculating stream power and shear stress using unsteady discharges rather than steady discharges, which can help the prediction of sediment transport. This result is significant because the model of Papangelakis et al. (2022) shows a linear relation between

excess stream power and sediment transport. The model is likely too simple because excess power is likely higher on the rising limb and lower on the falling limb. One hypothesis is that the Papangelakis et al (2022) model works well because the net result correlates well with the overall excess despite errors related to hysteresis because the errors tend to cancel each other out when the results are integrated over an entire flood hydrograph.

Table 3.4 The values of Shear Stress and Stream power for the falling limb, rising limb and total values during the hydrograph occurred on July 13th 2019. The column headings J1, J2, F1, and F2 refer to the four methods used to calculate discharge.

Parameters	Rising Limb (%)				Falling Limb (%)				Total (%)			
	J1	J2	F1	F2	J1	J2	F1	F2	J1	J2	F1	F2
Shear Stress	3.7	3.4	3.2	0.5	-64.1	-66.5	-64	-68	-56.6	-58.8	-56.6	-60.4
Stream Power	3.3	2.5	1.3	-1.17	-1.9	-8.8	-0.7	-11.3	-0.05	-4.8	-0.005	-7.8

3.3.3.2 Effect of sampling frequency on analysis of unsteady flow

The surface water elevation was measured with a frequency of 4 measurements per minute for this study, which far exceeds what is available from typical water level gauges. To assess the importance of the sampling frequency for the results, the frequency of data was downgraded by resampling the measured data with a time interval of 0.25 minutes to 0.5, 1, 2, 5, and 15 minute intervals. The analysis of event hydraulics was repeated at each time interval to see the importance of frequency of data in unsteady flows observed in urban channels.

One initial difference is that the identified time at which the event begins can change as a result of data frequency. This difference is apparent in flood duration calculations. The peak discharge can also be affected depending on the time the peak occurred. As shown the peak discharge varied by ~10% during the July 19th flood when the sampling interval was increased from 15s to 15 min (Figure 3.11). The other figures related to the frequency analysis is shown in Appendix B.

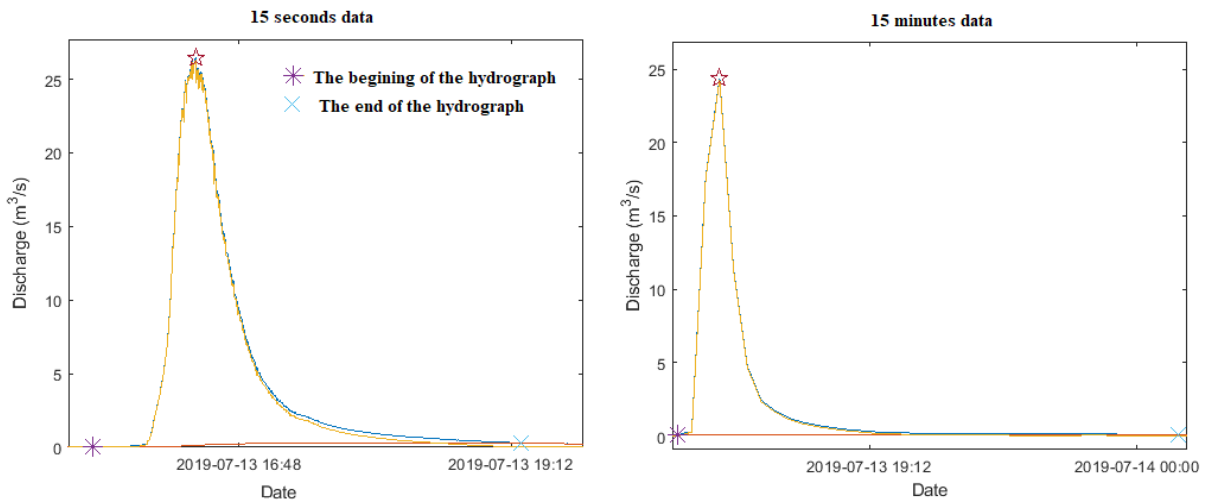


Figure 3.11 The start, end and the peak value of the hydrograph for 15 seconds and 15 minutes data frequencies

Figure 3.12 shows the three terms of Saint-Venant equation for all the six sampled frequencies. Overall, less frequent data means that the variation of these unsteady terms is muted relative to the 15 second data. The maximum value for $T3/T2$ on the rising limb decreases from ~ 0.14 for 15 second data to 0.08 for 15 minute data, while the minimum value on the falling limb increases from ~ -0.23 for the 15 second data to -0.19 for the 15 minute data. Similar trends are noted for all factors. $T4/T2$ is notable because the small rise observed early in the rising limb for the 15 second data is eliminated and only the relatively low values on the falling limb are observable with the lower frequency sample. A second notable change is that the noisiness of the signal decreases, so that $T5/T2$ for example, which has many high frequency fluctuations that give it a spiky appearance with 15 second sample, is characterized by a smooth rise and fall for the 15 minute sampling. The noisy appearance of the $T3/T2$ data that is apparent during the falling limb for 15 second data is also eliminated with measurements made only once every 15 minutes.

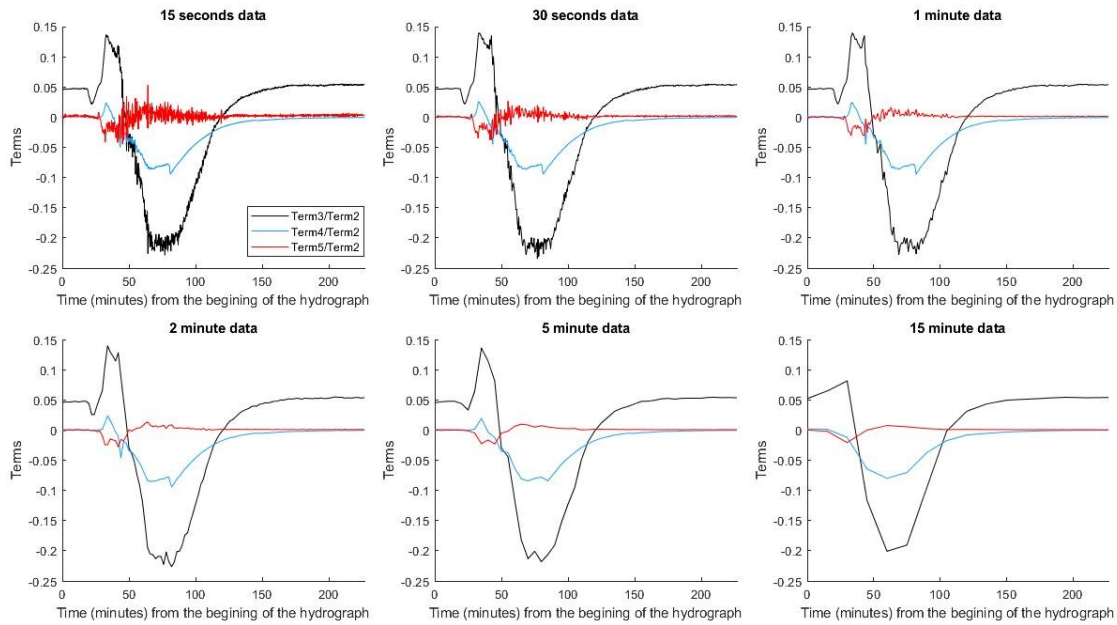


Figure 3.12 The three terms of Saint-Venant equation for the frequencies of 15 seconds, 30 seconds, 1 minute, 2 minutes, 5 minutes and 15 minutes.

Figure 3.13 shows the relative discharge calculated using all four methods and all six sampling frequencies. An important result is that the range of relative discharge calculated using methods based on 1 gauge (J1 and F1) decreases when the sampling frequency decreases. On the rising limb, for example the relative discharge is as high as ~10% for the 15 seconds data, but decreases to < 5% for the 15 minutes sampling. Variability on the falling limb is nearly eliminated with the 15-minute data. The red and black lines show the relative discharges calculated using two gauges. Their maximum value decreases and their minimum value increases by decreasing the data frequency, particularly on the rising limb of the flood. Nevertheless the 2-gauge methods are better able to capture the decreasing discharge due to a reduced Sw on the falling limb of the flood. This better representation of the flow unsteadiness also leads to a larger hysteresis loop in the rating curve for higher frequency data (Figure 3.14).

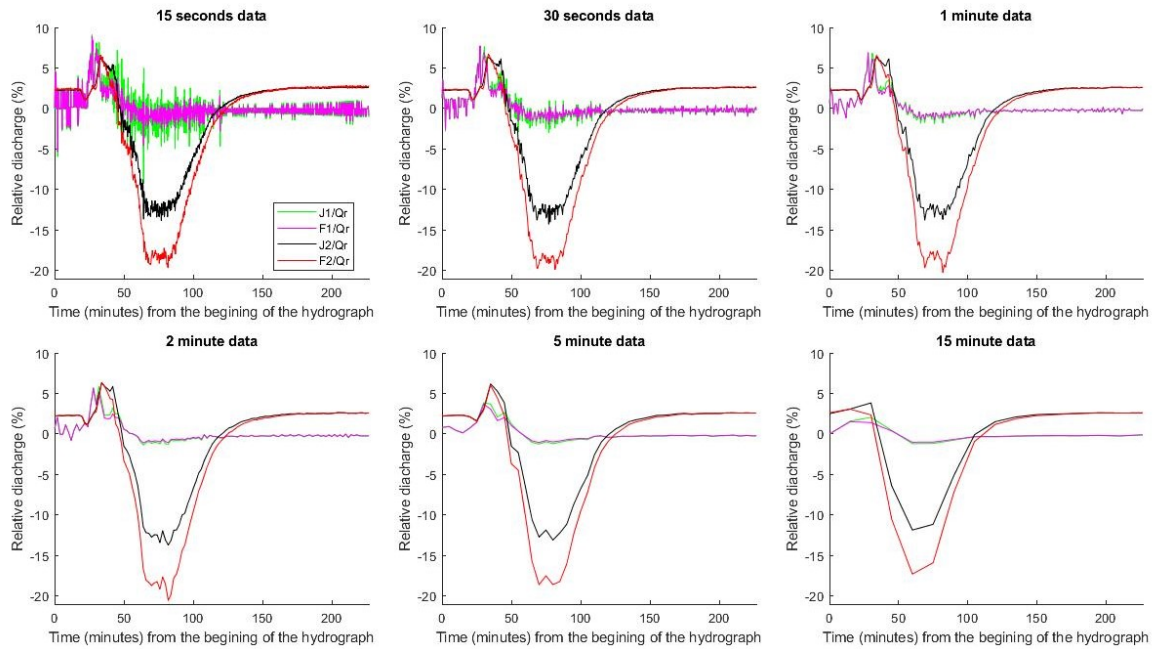


Figure 3.13 Relative discharge for all the six data frequencies

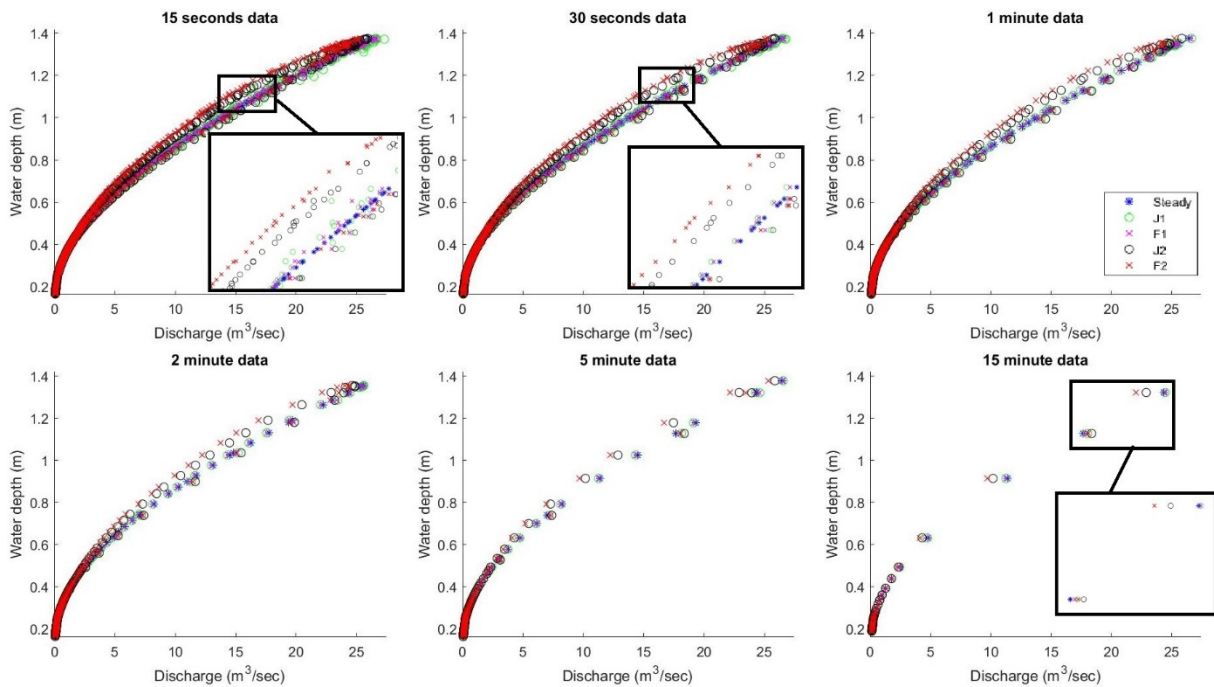


Figure 3.14 Rating curves of the six data frequencies

Figure 3.15 shows the histogram of the changes of depth over time for all the six data frequencies. As shown, the maximum value for dh/dt reduces from 11 cm/min for a 15 seconds sampling interval to 6.1 cm/min at 30 seconds and 1 cm/min at 15 minutes. At a 15-minute sampling interval the shape of the plot is relatively flat, which is characteristic of a random white noise. At higher frequencies, however it is possible to establish an upper limit to the changes in depth such that extreme events with high $\Delta h/\Delta t$ are less likely to occur. This shows that higher frequencies are very important to record the real changes of water surface elevation over time. The changes of flow over time is what affects the unsteadiness of data.

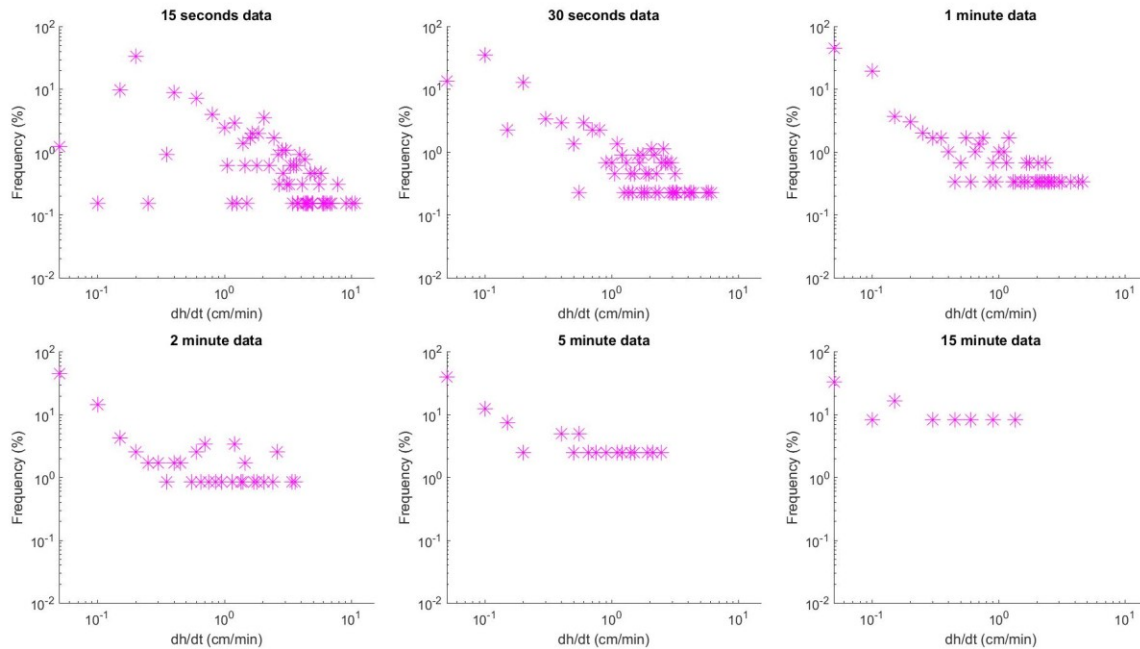


Figure 3.15 Histogram of the depth changes over time for all the six data frequencies.

3.3.4 June 20th, 2019 event analysis

3.3.4.1 Changes in water depth over time

The histogram of the changes of depth over time for the June 20th, 2019 event is shown in Figure 3.16. The maximum rate of change in water depth over time is recorded as 30.8 cm/min, which is higher than the July 13, 2019 hydrograph, but the total number of recorded data points is also higher at $n=2276$. The next two maximum values recorded for this hydrograph are 25.5 and 24.7 cm/min, which are close to the maximum value recorded for the July 13, 2019 hydrograph, which indicates that the rising limb

dynamics were similar for the two floods. The visual lines added to represent the data envelope from the July event also accurately represent the limits of the June event, which again supports the idea that they follow a similar distribution.

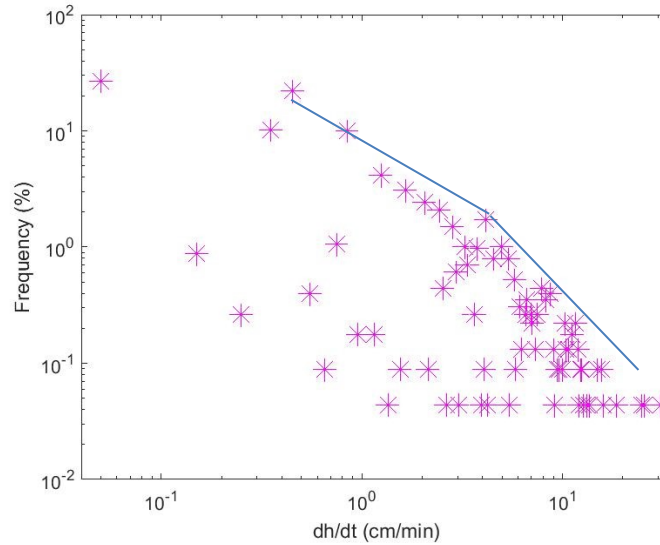


Figure 3.16 Frequency of rate of change of depth (h) over time (t) for the hydrograph observed on June 20th 2019. The blue lines represent the upper limit envelope drawn for the July 13th event.

3.3.4.2 High frequency single hydrograph (15 seconds data)

Figure 3.17 shows the relationship between S_w and SWE, with S_w again normalized by S_0 to show the hysteresis effects due to flow unsteadiness. As for the July event, a hysteresis loop is created at the initial stages of the rising limb, with S_w/S_0 rising to above 0.80 in this first rapidly rising limb. Unlike the previous event, however, there is a bump in the hydrograph and a second higher peak later in the event. The second peak produces a second hysteresis loop with a local maximum value of $S_w/S_0 \sim 0.65$. The two maximum values of S_w/S_0 are inflection points on the graph.

Figure 3.18 shows the relative discharge in percent. Calculated using 1 gauge, these values rise to a high value of 25% at the beginning of the rising limb but fall quickly and are near zero for most of the hydrograph. The high value on the rising limb is likely due to the high slope dy/dt , which exceeded what was observed in all the other flood events (Table 3.3). Calculated using 2 gauges, the relative discharge is non-zero at the beginning of the event due to the difference between S_f and S_0 at baseflow,

but the magnitude of the peak is less ($\sim 10\%$). The relative discharge shows a clear double peak for the two-gauge methods, which is the result of the double loop in the S_w/S_0 curve. In the falling limb the relative discharge decreases to a value of approximately zero and then rises back to a value of approximately 5%. This persistent difference between the discharge estimates is a function of the difference between the bed and water surface slopes that is observed at baseflow for the gauge locations.

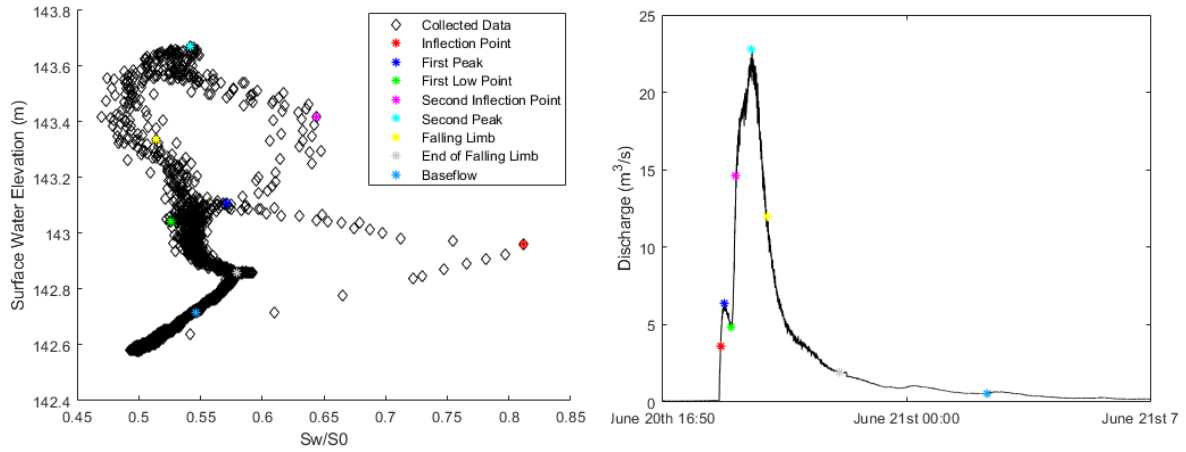


Figure 3.17 Measure data: Surface water slope versus surface water elevation with respect to hydrograph

Figure 3.19 shows the three terms in the Saint-Venant equation with respect to channel bottom slope. This figure also shows a similar trend to what we saw in Figure 3.7 in which the T3 and T4 rise during the beginning of rising limb and they drop down to a minimum value in the falling limb. For the rising limb T5 follows an opposite trend compared to the other two curves. During the falling limb this term shows fluctuation around zero.

The cumulative effect of the flow unsteadiness within the event hydrograph (Figure 3.20a) is shown as $S_f/S_0 - 1$ in Figure 3.20. In this figure, the peak Q is similar to the July hydrograph, and $S_f/S_0 - 1$ before dropping to 0 between times 3 and 4 and then remaining below 0 through the peak of the flood and the beginning of the rising limb. $S_f/S_0 - 1$ begins to increase toward 0 after hydrograph inflection point 6 and eventually returning to a value of ~ 0.10 . The differences with the previous event are that $S_f/S_0 - 1$ has a higher maximum for this event, again likely because of the rapid change of depth over time (12.75 cm/min) at the beginning of the rising limb, and also that the falling limb was much milder

in the sense that $S_f/S_0 - 1$ only reached a minimum of ~ -0.05 , which is much higher than the July event. The reason for the moderate value on the rising limb is likely a function of the complexity of the June flood. As shown on the hydrograph, there were three phases of rapid rise in the discharge, each of which was characterized by a local peak in the $S_f/S_0 - 1$ (points 1, 3 and 5 of Figure 3.20). These three periods can be understood as pulses within the flood where the water level and discharge are increasing rapidly and then ebbing slightly before they are joined by another pulse in the flow. The latter two pulses where $S_f/S_0 - 1$ was increasing were thus superimposed on a period of time in which the hydrograph from the first wave of the flood was ebbing and, based on the July flood, would be expected to have a negative $S_f/S_0 - 1$ value. In this flood, the rising limb is thus a series of three rising limbs, and the falling limb for the entire flood only begins after the third pulse has ended. As described in the description of Figure 3.8 we can calculate errors for shear stress and discharge. In Figure 3.20 the error for shear stress is between -7% and 25%. The error for discharge is between -0.49% and 6.25% for this hydrograph.

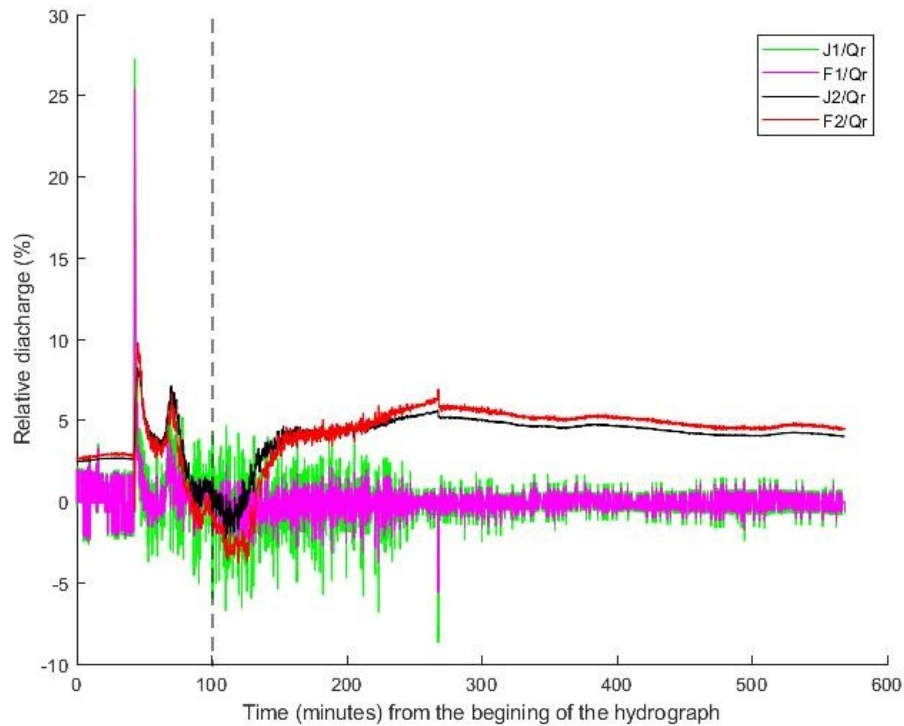


Figure 3.18 Relative discharge shown for all the four methods of discharge calculation. The relative discharge is calculated as $\frac{Q-Q_r}{Q} * 100$, where Q is the unsteady discharge calculated using any of these four methods, and Q_r is the discharge calculated using a steady flow assumption and Manning's roughness equation.

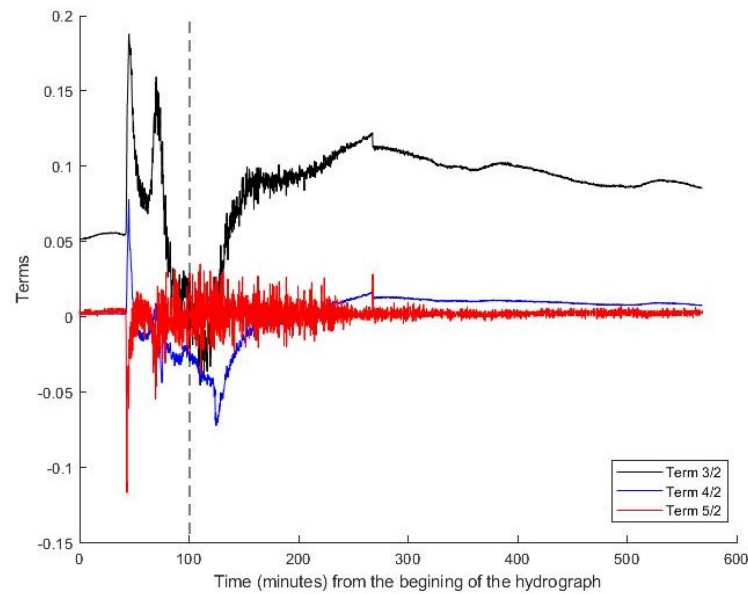


Figure 3.19 The terms of equation 12 listed in the last chapter for the hydrograph that occurred on June 20th 2019

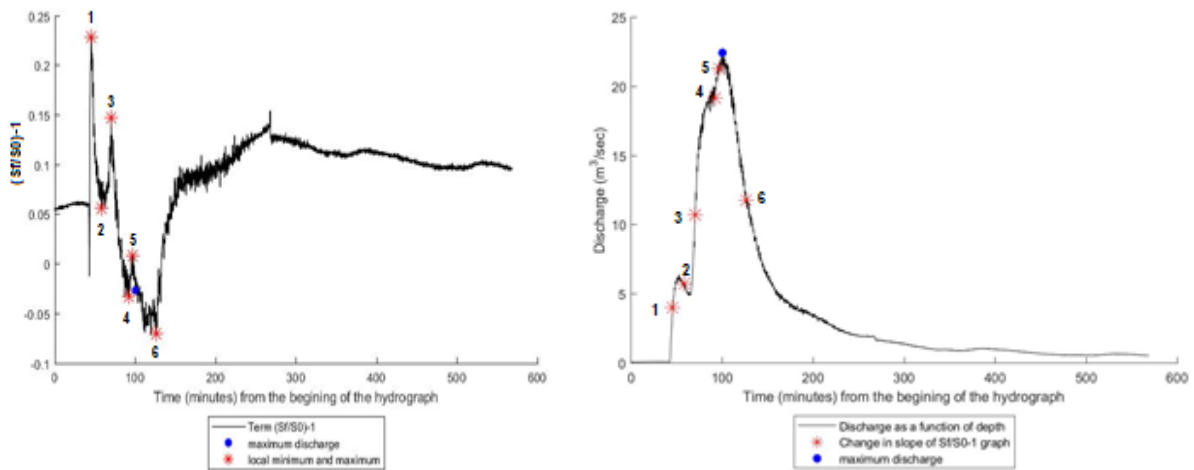


Figure 3.20 Both S_f/S_0-1 and hydrograph plotted together. The red markers show the points where the slope of S_f/S_0 changes on both figures. The blue marker shows the maximum discharge.

Based on what is shown in Figure 3.20, the $S_f/S_0 - 1$ has a higher maximum value in the rising limb compared to the last hydrograph. This results in a looped rating curve shown in Figure 3.21, where the two black and red lines are located on the right side of the steady flow graph (blue markers). The falling limb curve in the rating curve however, is very close to the steady flow curve in this hydrograph, which

is expected since the $S_f/S_0 - 1$ value does not go to a low minimum value similar to the last hydrograph. The rating curve of this hydrograph compared to the rating curve of the last hydrograph shows a smaller loop, while on the rising limb the discharges are much higher than the steady flow discharges.

The cyan dots on Figure 3.21 represent discharges estimated from the measured surface velocities using SVR gun and standard assumptions about the surface velocity relative to the average velocity. Corresponding depths to calculate flow area were obtained from the depth gauge data based on the time of measurement. As shown, the discharges measured using this method result in a looped rating curve, which at high discharges correspond with the estimates from the 2 gauges method. During this event, the SVR gun appeared to detect the falling rain drops instead of water surface, which explains the overestimation of discharge below $Q \sim 5.0 \text{ m}^3/\text{s}$ using the SVR data.

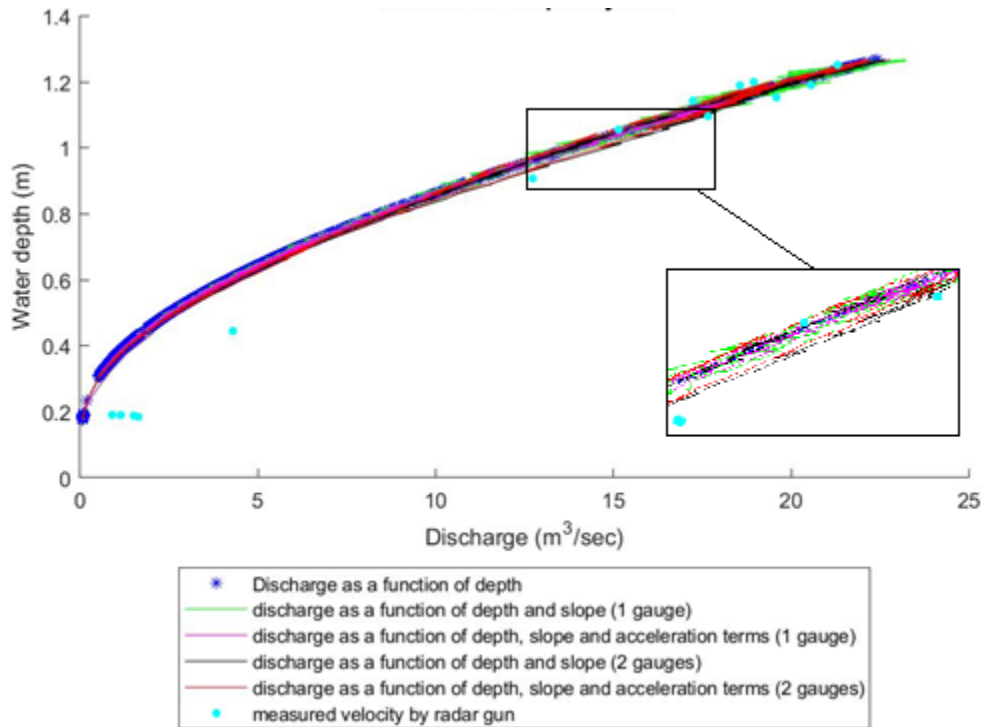


Figure 3.21 Rating curve observed for the hydrograph happening on June 20th 2019

3.3.4.3 Shear Stress and Stream Power

Table 3.5 shows the percentage of shear stress and stream power over rising limb, falling limb, and the entire hydrograph. For the shear stress it can be seen that overall it is similar to the July hydrograph because the cumulative values are positive on the rising limb and negative on the falling limb. However, even this accumulation by limb masks some of the variability within the respective limbs because the single event represents three pulses of discharge rather than a single event.

Stream power found using one-gauge methods for calculating Q show similar results to those for the July hydrograph because they are positive for the rising limb, negative for the falling limb, and near zero when accumulated over the entire hydrograph. In this hydrograph the stream power found using the data from two gauges (J2 and F2) are higher compared to its steady values in rising limb, falling limb and the entire hydrograph but the cumulative values again mask variability over the flood and within each of the rising and falling limbs.

Table 3.5 The values of Shear Stress and Stream power for falling limb, rising limb and total values during the hydrograph occurred on June 20th 2019

Parameters	Rising Limb (%)				Falling Limb (%)				Total (%)			
	J1	J2	F1	F2	J1	J2	F1	F2	J1	J2	F1	F2
Shear Stress	2.1	6.4	1.9	4.7	-63.2	-60.1	-63.1	-60.2	-56.6	-53.4	-56.6	-53.7
Stream Power	1.3	3.6	0.5	1.8	-1	3.4	-0.4	2.9	-0.05	3.5	-0.01	2.4

3.4 Discussion

This chapter examines unsteady flow assumptions for an urban channel using a unique high frequency data set. The results show that steady flow assumptions will lead to systematic error in the calculation of discharge and shear stress. The following subsections will discuss i) the effect of hydrograph shape on flow unsteadiness; ii) flow routing assumptions for urban hydrographs; iii) lower sampling frequencies and their effect on flow unsteadiness; and iv) a comparison with a rural channel.

3.4.1 Shape of Hydrographs

3.4.1.1 Single Peak Hydrograph

In order to better understand the effect of shape of our hydrographs on the level of unsteadiness of them, in this section a single hydrograph is analyzed. Figure 3.8a shows the hydrograph of the July 13th 2019 event. The inflection points on this graph was marked by asterisks. The corresponding points are also plotted on the Figure 3.8b, which shows the $S_f/S_0 - 1$ graph. In this graph it can be seen that the inflection points on the Figure 3.8a corresponds with the maximum and minimum points on the Figure 3.8b. To better explain the ideas of this section, an schematic hydrograph is plotted in Figure 3.22. This figure shows an example of a hydrograph. The red points on this graph indicate the inflection points. Based on our observations in the results section, it looks like the inflection points on the hydrograph create a relative maximum or minimum point on the $S_f/S_0 - 1$ graph. The concave part of the hydrograph is when the $S_f/S_0 - 1$ curve decreases, and the convex part of the hydrograph is when the $S_f/S_0 - 1$ curve increases to its relative maximum.

The shape of the hydrograph is therefore a good indicator to determine the unsteadiness of the flow. If the relative maximum of the $S_f/S_0 - 1$ goes to a positive value higher than 0.1 and the relative minimum in this curve goes to a value smaller than -0.1 (similar to what observed for the July 13th 2019 hydrograph), the rating curve shows a looped rating curve and the unsteady discharges are far different with the steady flow discharges.

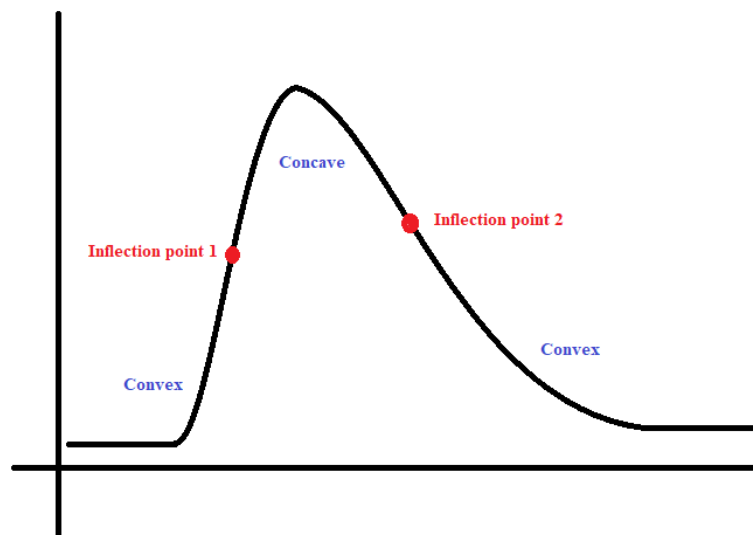


Figure 3.22 An example of a hydrograph with its inflection points

Three types of rating curves are observed in this study for a single shaped hydrograph, as shown conceptually on Figure 3.23. Panel (a) shows the flows where the rising limb stays near the blue line, but slightly on the right side of the graph, and the falling limb in this type is the same as the steady discharge rating curve. The rating curves presented for the hydrographs happening on June 20th and October 27th are from this type of the rating curve. By looking at their $S_f/S_0 - 1$ curve, it can be seen that for both hydrographs the $S_f/S_0 - 1$ is positive for the beginning of the rising limb and it is negative for the beginning of the falling limb and $S_f/S_0 - 1 > 0.1$ for the rising limb. This leads to rising limb discharges being higher than the steady discharges in the rating curve. But because the falling limb values for the $S_f/S_0 - 1$ do not go lower than -0.1, the curves do not get separated from the steady flow rating curve for the falling limb.

The type b rating curve (Figure 3.23b) is the curve where rising limb is slightly on the right side of the steady rating curve and the falling limb is on the left side of the rating curve. This pattern is observed for the hydrographs occurred on July 13th and July 17th 2019. By looking at their $S_f/S_0 - 1$ it can be seen that for the rising limb this value is positive and it exceeds 0.1 and for the falling limb this value becomes negative and lower than -0.1. This shows that when $S_f/S_0 - 1$ goes to a lower negative value for the falling limb, the unsteady discharges are much smaller than the steady flow discharges. The rating curve presented for July 17th has another type of rating curve for the last rising and falling limb that happens on the graph. In this rating curve, both rising limb and falling limbs are on the left side of the steady rating curve, but this is due to all the bumps that this graph has before it reaches its last peak. Looking at the $S_f/S_0 - 1$ curve it can be seen that this value is negative for both rising and falling limbs of this last bump.

The type c rating curve (Figure 3.23c) occurs when both the rising and falling limbs of the hydrograph stay on the right side of the rating curve. The last three hydrographs listed in the appendix have this type of the rating curve. By looking at their $S_f/S_0 - 1$ curves it can be seen that in these hydrographs the $S_f/S_0 - 1$ is mostly positive for the entire hydrograph. The size of the loop in the rating curve depends on the difference between the maximum and minimum values in the $S_f/S_0 - 1$ figure. If the difference between the maximum and minimum is high in this figure, the loop of the rating curve becomes larger. Since the rise and the drop in the $S_f/S_0 - 1$ figure depends of the shape of the hydrograph, it can be concluded that the size of the loop of the rating curve is also predictable by looking at the shape of the hydrograph.

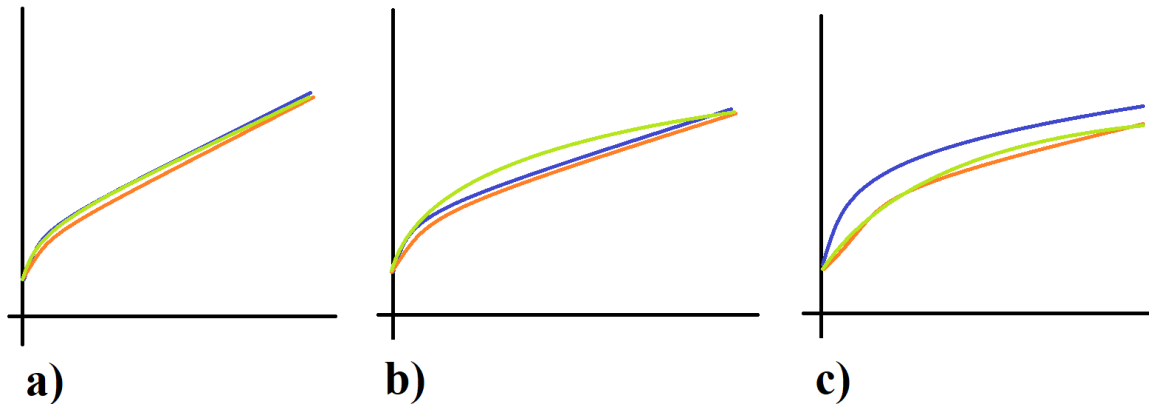


Figure 3.23 The conceptual figure for the rating curve. For these plots the blue line is the steady flow rating curve, while the orange line shows the rising limb and the green line shows the falling limb.

3.4.1.2 Sawtooth Shaped Hydrograph

Urban floods occur quickly, with rapid rising and falling limbs, which has an important consequence for hydrographs and representation of unsteady flows. The July flood was selected because it had a single significant discharge peak, which made it relatively easy to analyze. Assuming the July flood can represent a unit hydrograph for the watershed, the time from the start of the rise to the inflection point on the falling limb is only about 50 minutes. For the June event, point 2 occurs only 71 min after the start of the rise of the flood, while point 4 occurs at 24 minutes after point 2. These pulses are thus overlapping events despite the fact that they occur relatively close to each other. Other events shown in the appendices also multiple overlapping hydrographs. An event from July 17th 2019 is shown in Figure 3.24 for comparison. Over the initial 500 minutes, two minor peaks are noted in the discharge, and these result in fluctuations in $S_f/S_0 - 1$ up to 0.1. A larger peak of $Q \sim 6 \text{ m}^3/\text{s}$ at ~ 580 minutes results in a similar positive peak but then a $S_f/S_0 - 1 \sim -0.2$ through the peak of the flood, and two larger peaks in Q produce a sawtooth pattern with $S_f/S_0 - 1 \sim -0.3$. Other events with different discharge magnitudes and different times between peaks can produce much different floodwaves that variable impacts on the flow unsteadiness. Floods on May 9th, 2019 and Oct 31st 2019, for example, have lower peak discharges but higher positive values of $S_f/S_0 - 1$.

A physical explanation for the extreme variability in the response of the watershed to floods appears to be linked with the magnitude and the sequence of the flood events. A single strong pulse of flow will lead to an increase in the friction slope because the flood is moving downstream into the channel and floodplain that has not yet had high water levels and so is relatively dry. This would explain why the

highest slopes occur for the June and July events shown, where the peaks were relatively isolated. Such a mechanism could also explain why more moderate floods such as the May 9th and Oct 31st flood have higher friction slopes than the larger July 17th event. Such a mechanism would also explain the negative values of $S_f/S_0 - 1$ that were found for the July 17th event, where the largest peak came after four more moderate peaks. The moderate peaks would serve to wet the downstream the downstream channel and floodplain and are shown to reduce the friction slope relative to the bed slope through the peak of the flood and on the falling limb.

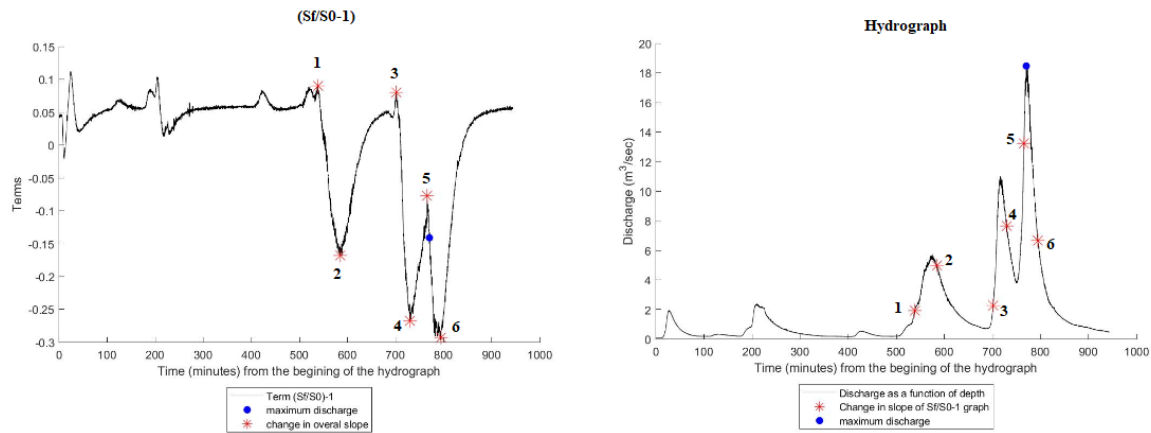


Figure 3.24 Both S_f/S_0-1 and hydrograph plotted together for the event occurred on July 17th 2019. The red markers show the points where the slope of S_f/S_0 changes on both figures. The blue marker shows the maximum discharge.

3.4.2 Steady vs Unsteady and One-Gauge vs Two-Gauge Routing Method

Based on the rating curves of the flows in Wilket Creek observed in this chapter, it can be seen that if the steady flow assumption is used for calculating discharge in this creek, the changes of discharge over rising and falling limbs are not detected. This leads to calculating incorrect discharges for the series of unsteady peaks in discharge that are characteristic of this urban creek.

Based on the results of the current study using two gauges for calculating discharge is necessary for the hydrographs with rating curve types of b and c. In these two types of rating curves, the rising limb discharges are far greater than the falling limb discharges. Using 1-gauge data does not calculate correct discharges for rising and falling limb.

Based on the data presented in the current study, it can be seen that adding acceleration terms to flow routing using 1-gauge data decreases the fluctuation of relative discharge. Adding it to the flow routing using 2-gauge data decreases the relative discharge during the falling limb. This means adding acceleration terms leads to lower discharges for the falling limb. In some hydrographs adding the acceleration terms to the routing method using 2-gauge data gives higher peak relative discharges for the rising limb. The difference between relative discharges found using 2-gauge data with and without acceleration terms is negligible ($<1\%$) for most hydrographs observed in this chapter. However, for the falling limb of the hydrographs happening on July 17th and July 13th the difference between the routing method using 2-gauges data with and without acceleration terms reaches $\sim 5\%$. This difference is $\sim 2\%$ for the hydrograph happening on June 20th. This shows that adding acceleration terms to routing methods using 2-gauges data is necessary for the hydrographs happening on July 13th, July 17th and June 20th. For the rest of the hydrographs observed in this chapter using acceleration terms to calculate discharges does not significantly change the results.

3.4.3 Water Surface Elevation Data Frequency

In comparison with the 15 second interval measurements gathered as part of this study, lower frequency water surface measurements will dampen the magnitudes of unsteady terms. With lower frequency data, for example, the fluctuation of the acceleration term (T5) vanishes. However, since the magnitude of this term is smaller than the diffusion terms (T3 and T4), the inability to accurately resolve the acceleration term does not typically affect flow and shear stress estimates in a significant way. By looking at Table 3.6 it can be seen that while the maximum rate of depth change decreases steadily with decreasing data frequency, the level of unsteadiness shown by $\Delta(Sf/S0-1)$ is relatively similar from 15 seconds to 2 minutes before decreasing at 5 and 15 minute frequencies. The peak discharge similarly stays relatively similar from 15 seconds to 5 minutes before dropping about $2 \text{ m}^3/\text{s}$ when the data frequency decreases to one recording every 15 minutes. Therefore, it can be concluded that the 15 minute frequency data is not reliable for such unsteady flows and the ideal time intervals for accurate flow and stress estimates are 2 to 5 minutes. These are intervals for the method that uses two gauges to calculate discharge. Therefore, for the purpose of studying in-channel floodwaves we recommend a 2-5 minute data interval with the two-gauge method of routing floodwaves.

Table 3.6 The characteristics of the floods based on the six studies frequencies for the hydrograph happening on July 13th 2019. In this table, $\Delta(S_f/S_0-1)$ means the difference between the maximum and minimum values in the S_f/S_0-1 graph.

Frequencies Parameter	15 seconds	30 seconds	1 minute	2 minutes	5 minutes	15 minutes
Peak discharge (m ³ /s)	26.46	26.46	26.46	25.6	26.46	24.37
Maximum dh/dt (cm/min)	10.7	6.1	4.6	3.6	2.45	1.35
$\Delta(S_f/S_0-1)$	0.4475	0.4533	0.4530	0.4506	0.4213	0.3372

Similar conclusions can be drawn for the June 20th event. As shown on Table 3.7, the peak discharge does not change between 15 second and 2 minute data, but then decreases at 5 and 15 minutes. The unsteadiness ($\Delta(S_f/S_0 - 1)$) begins decreasing after 1 minute data reduces by ~0.02 by decreasing the data frequency to 2 minutes. This indicates that using 1 minute data frequency is the lowest frequency we can use for the hydrograph happening on June 20th.

Table 3.7 The characteristics of the floods based on the six studies frequencies for the hydrograph happening on June 20th 2019. In this table, $\Delta(S_f/S_0 - 1)$ means the difference between the maximum and minimum values in the $S_f/S_0 - 1$ graph.

Frequence Parameter	15 seconds	30 seconds	1 minute	2 minutes	5 minutes	15 minutes
Peak discharge (m ³ /s)	22.43	22.43	22.43	22.43	21.56	20.43
Maximum dh/dt (cm/min)	12.75	7.2	5.7	3.1	2.65	1.45
$\Delta(S_f/S_0-1)$	0.2997	0.3028	0.3139	0.2812	0.2463	0.1700

3.4.4 Flashy Floods in Urban and Rural Creeks

This chapter of the thesis is dedicated to the hydrographs observed in urban channels. The main reason that we focused only on urban flows is that the floods with high peak flows and with looped rating curves happen more often in urban channels compared to rural channels. Therefore, we assumed that using the 2-gauge method is more critical for urban creeks compared to rural creeks. However, the analysis of the previous chapter showed that rapid water surface level changes were also possible in a rural creek, even if they were far fewer.

To test this assumption, a hydrograph was selected at Ganatsekaigon Creek with the highest observed peak flow during the 4 years of data collection period (2 minute data collection frequency). In this creek, we observed a flow with a high peak that was comparable to the flows observed at Wilket Creek. Both hydrographs have a peak of nearly 20 m³/s. Figure 3.25 shows the terms for both Wilket and Ganatsekaigon Creek floods. Contrary to expectations that the rural creek would have relatively low error, in fact the error is of similar magnitude to that in the urban creek based on the terms in Saint-Venant equations, though the patterns and the relative magnitudes of the terms through the flood are different. In this figure the terms values reach a negative value due to the shape of the hydrograph.

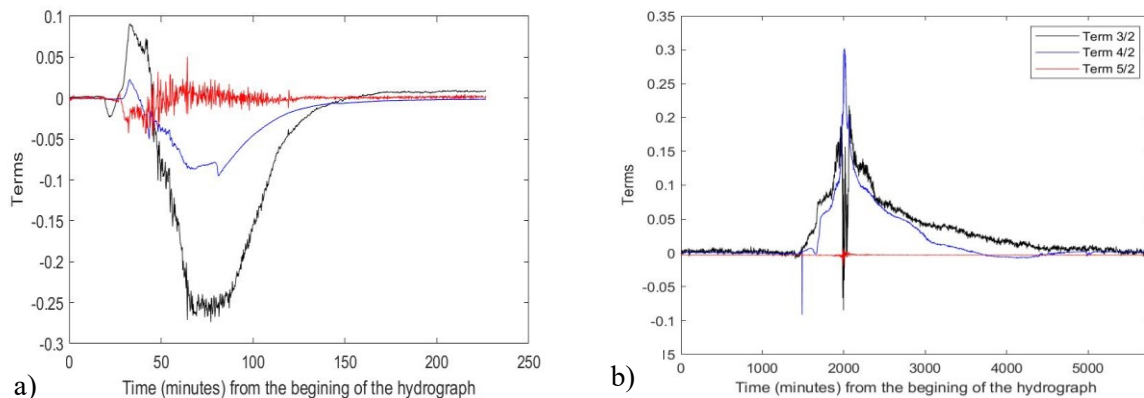


Figure 3.25 a) Terms for the hydrograph observed on July 13th in Wilket Creek b) The hydrograph observed in Ganatsekaigon Creek.

A comparison of water surface slopes and elevations (Figure 3.26) shows that the hydrograph observed at Wilket Creek shows a looped hysteresis, while the one observed in Ganatsekaigon Creek does not show the same trend. However, T3 stays high over most of the hydrograph duration and T4 goes to 30% of term 2, indicating that it represents a significant error. Although such high flows are observed

less frequently in the rural catchment compared to that in urban catchment, it can be concluded that the two-gauge method will lead to significant reduction in error for both urban and rural catchments.

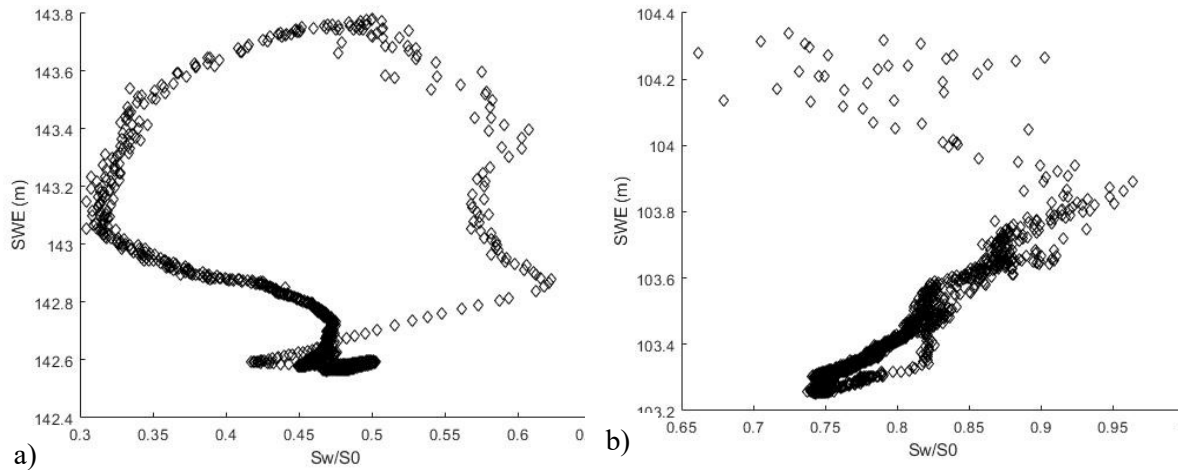


Figure 3.26 The relationship between observed surface water elevation versus observed surface water slope a) in Wilket Creek and b) in Ganatsekaigon Creek

3.4.5 Sediment transport

A key motivation for the current study was to investigate the impact of unsteady flows on sediment transport. Our results show that the steady flow assumption underestimates shear stress on the rising limb but overestimates it in the falling limb of large floods. This work supports previous work by Phillips, Hill, & Paola, (2016), who showed a strong hysteresis effect in a series of laboratory experiments such that the modelled critical threshold for sediment entrainment was different on the rising and falling limbs of the hydrographs. A re-interpretation of that work based on the current analysis is that the shear stress on the limbs are different rather than the threshold. Previous work had been able to fit data on the displacement distance of sediment tracers to the excess stream power (i.e. the stream power above a sediment transport threshold) integrated over time (Papangelakis et al., 2022). This approach used steady flow assumptions and was successful at collapsing results from Wilket Creek (urban) and Ganatsekaigon (rural) creeks in addition to a third site and other sites from the literature with presumably different rising and falling limb slopes. These results suggest that the modification of the threshold by the flow unsteadiness is a second order effect, a hypothesis that is also supported by the laboratory experiments of Phillips et al. (2016) and others. Still the complexity of the hysteresis response of sediment transport to floods shows that this effect is not insignificant (Gunsolus & Binns,

2018; Plumb et al. 2020). Future work should with unsteady flows should consider the unsteady nature of the flow in addition to the condition of the bed to explain the variability of sediment transport during floods.

3.5 Conclusions

The following conclusions can be drawn from the results and discussion of this chapter:

1. Urban rivers are characterized by rapidly rising and falling hydrographs that lead to a sawtooth pattern of discharge, where floods are typically characterized by a series of rapid rising and falling limbs that can have significant unsteady flow signatures that affect discharge, shear stress, and stream power estimates;
2. Most gauging data is gathered for stream gauging based on the assumption that steady flow estimates are sufficiently accurate during floods. The low sampling rates and single gauge strategy means that such data are not suitable for characterizing the variability of discharge and shear stress relative to this assumption over the hydrograph. Such a limitation means that errors of 10-30% in discharge and shear stress estimates are likely to be common in urban channels and in large floods in other watersheds;
3. A two-gauge system with both gauges recording simultaneously at frequencies between 1 and 5 minutes is recommended to allow users to apply a diffusion routing method and assess the significance of the third and fourth terms in the St-Venant equations, which represent the streamwise gradient in water depth and velocity, respectively. Including these two terms will allow better characterize of discharge and shear stress during rising and falling limbs of floods characterized by a strong hysteresis of the water surface slope;
4. Rural channels may suffer from similar weaknesses in gauging data during extreme events despite their more gradual rising limbs, indicating that two-gauge systems could be widely implemented for estimating discharge and shear stress in rivers.

Chapter 4

Sediment Transport under Unsteady Flows in a scaled model of Wilket Creek

4.1 Introduction

Natural channels are composed of fractional grain size sediments, where the channel material is made of different sediment sizes. Flow unsteadiness is known to affect the transport of such graded sediment distributions, where during flood events, sediment transport is high (Berta & Bianco, 2010). Some studies have shown that flow discharge alteration can affect sediment transport rate (Plumb et al., 2017). Discharge alteration can happen frequently due to natural or anthropogenic factors such as rainfall, land use change, and dam constructions (Bombar et al., 2011; Gregory, 2006), which in turn impacts channel stability and morphology. Poff et al., (1997) characterized natural flow regimes with five parameters: magnitude, frequency, duration, timing, and unsteadiness or flashiness. Each of these parameters can affect sediment transport. Since measuring sediment transport under high flows in rivers is extremely difficult, scientists have done laboratory experiments to track bedload sediments under unsteady flows (Guney, Bombar, & Aksoy, 2013; Xiao, Hu, & Yang, 2020).

Previous experiments of sediment transport in unsteady flow have often focused on the question of transport hysteresis in unsteady flow. Clockwise hysteresis, where more transport occurs on the rising limb, and counterclockwise hysteresis have been observed (Mrokowska & Rowinski, 2019; Ben D. Plumb, Juez, Annable, McKie, & Franca, 2019). Lee et al., (2004) showed that there is a temporal lag between peak flow and peak of sediment export. (Hassan, Egozi, & Parker, 2006) showed a gradual coarsening of bedload over a rising limb and a bed fining over a falling limb, which shows a clockwise hysteresis effect. The same article showed that by increasing hydrograph duration a coarsening is also observed in the falling limb. Therefore, it can be concluded that the shape of the hydrograph can affect the transport of mixed size sediments, with the protrusion and hiding effect modeled by Wilcock and Crowe (2003), among others, potentially varying with flow stage. Mao, (2012) showed an opposite trend with counterclockwise hysteresis and coarser bed particles moved during the falling limb. Wang et al., (2015) found that bed surface grain size distribution did not change during hydrographs with longer rising limbs but was finer where rising limbs were shorter. Plumb et al., (2019) investigated flow magnitude and duration by tracking differences in exported sediment when running a series of

hydrographs, showing that shorter duration floods with the same peak discharge tended to result in larger changes in bed variability and more sediment export on the rising limb, which they attributed to differences in bed armoring, with shorter floods leading to greater disturbance of the bed and reduced armoring.

Previous studies have necessarily simplified some aspects of sedimentology and river morphology that may be relevant for understanding field scale dynamics. For example, many studies used only one sediment size. Recent studies have shown that different particle sizes act differently under identical flows (Wang et al., 2015). The impact of a meandering planform or changes in channel width have also rarely been considered. The recent laboratory study by Pierce et al (2021) did consider these elements in a study on the effects of irregular meanders on sediment transport during a steady flow, noting a number of sediment size-dependant effects occurred. While a net equilibrium was achieved between the feed and export of sediment at different feed rates, the size breakdown of exported sediment showed that different size fractions achieved equilibrium at different times. The coarse particles tended to move faster through the empty channel, forming a set of skeleton bars that were then covered over by finer sediments, the latter of which built into thick deposits that sometimes exceeded the initial flow depth in the channel. Due to the sequential building of these bars, a downstream coarsening effect was observed, with the coarsest sediment exported first from the channel. Preliminary videos suggested that different size fractions moved through the channel in different locations, but Piece et al. (2021) did not design their sampling regime to assess spatial differences in sediment transport and did not consider unsteady flow.

This research is part of a larger project to develop restoration techniques that employ nature-based solutions using the power of the river to move sediment and rebuild morphology. This work follows directly from the steady flow experiments of Pierce et al. (2021), which was in turn based on the characterization of Wilket Creek by Bevan et al (2018) and the sediment tracking work of Papangelakis et al. (2019). Wilket Creek was also one of the sites described in Chapters 2 and 3 for the investigation of urban hydrographs. The novelty of the current work is that the laboratory study uses unsteady flow hydrographs with different shapes and uses painted sediment tracers with a novel video technique to describe the spatial variability of sediment transport during a flood. The objectives are to: 1) develop an image-based technique to trace the movement of painted particles during an experiment, and 2) assess the impact of unsteady flow on sediment transport and bar formation in the irregularly

meandering channel. The experiments help to better understand sediment transport in unsteady flows and contribute to the development of nature-based solutions for river restoration and management.

4.2 Methods

4.2.1 Experimental Setup

4.2.1.1 Flume

Physical experiments were performed in 13.3 m long, 2 m wide tiltable flume located at the University of Waterloo. (Figure 4.1). The experimental setup is largely the same as described by (Peirce, MacVicar, Papangelakis, Vervynck, & Ashmore, 2021). Flow is pumped to the head tank from a reservoir under the floor and returned to the reservoir via the return channel below the floor. Flow exiting the channel passes through a mesh basket, which removes sediment particles that are transported out of the channel. The camera cage is a cart that moves on the outer rails of the flume and has a steel frame designed to hold cameras, lights and plastic curtains up to 2.5 m above the bed of the flume. The bridge is a second cart that moves on the inner rails of the flume and is used to facilitate the installation of the cameras and the distribution of sediment.

4.2.1.2 Flow Control

Flow to the head tank pumped from a below-grade reservoir was controlled with a proportional valve with positioner (ASCO series 290). The position of this valve is designed to be controlled via a computer, and a custom control software was written to allow different hydrographs to be run for the experiments. Hydrographs are described by time and discharge pairs in a .txt file that control when the pump setting is changed. The hydrographs can have any shape, but the time step must be constant for the entire hydrograph. The signals were calibrated using bucket and stopwatch tests over a range of 0.35 – 3.7 L/s, with a mean error of ± 0.053 or 1.4% of the total range. The absolute error did not show any trends with discharge above 0.50 L/s, but below this level the error could be as high as 20%.

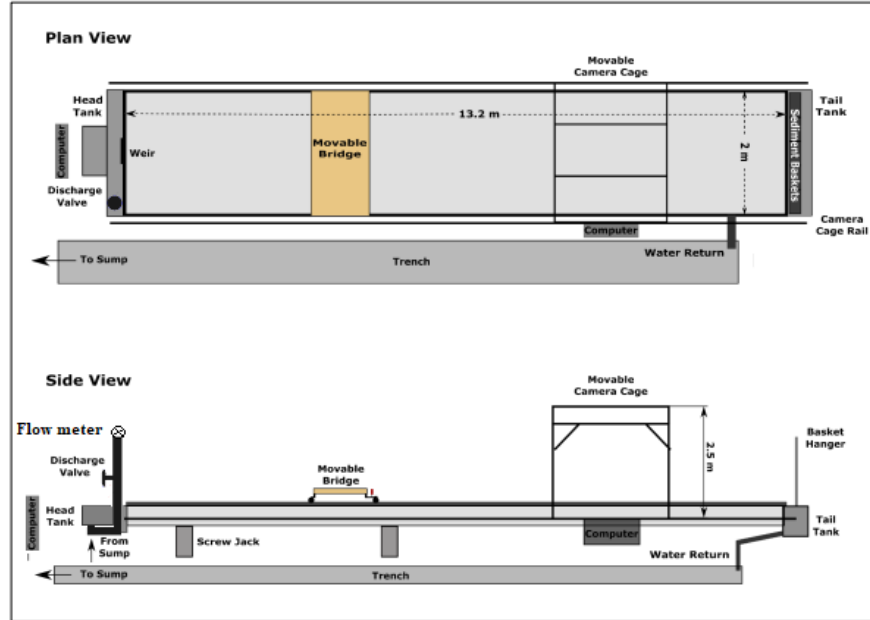


Figure 4.1 Schematic plan view and side view of the flume used for the current research

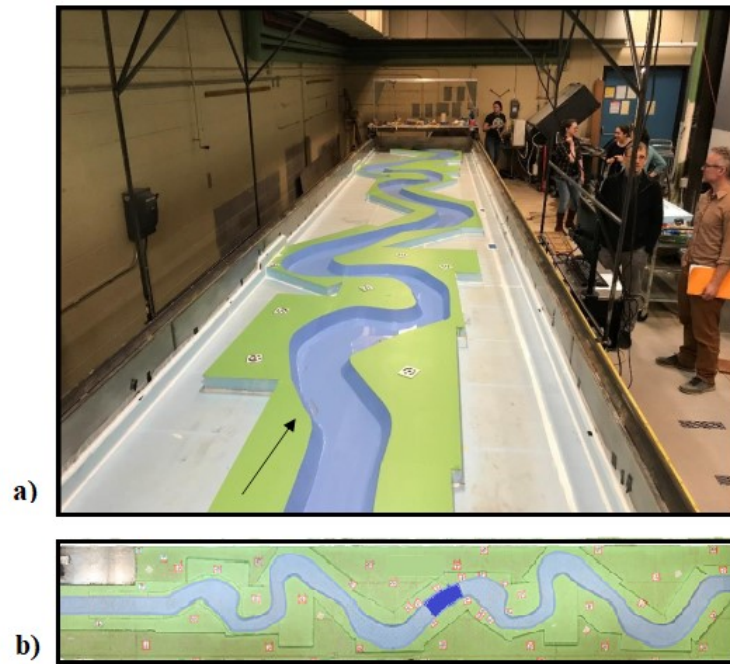


Figure 4.2 a) pattern of the channel b) plan view of the flume used for the experiments of the current study. The dark blue part is the region of interest for our current study.

An ultrasonic flow meter (Omega model M5221) with FDT-40 series transducers was installed on the pipe that connects the sump pump to the upstream tank. The readings of this flow meter are shown on a display that is attached to the pipe near the entrance to the upstream tank. The custom software can read the signals from the flow meter and plot the readings over time.

4.2.1.3 Experimental model and scaling

The experiments used an irregularly meandering channel that was constructed within the flume to create a 1:40 length scale of a section of Wilket Creek located in Toronto, ON (Peirce et al., 2021). The slope of the flume was set at 1.5% to match the previous study, with the sinuosity ($\Omega = 1.35$) and channel slope ($S_0 = 1.1\%$) set to match the field site in Wilket Creek of Papangelakis et al. (2019).

Froude scaling was used to scale the discharge for the experiment following the recommendations of Julien, (2002). For this method the scaling equations are shown below.

$$Q_{sc} = Z_{sc}^{2.5} = 40^{2.5} = 10100 \quad (4.1)$$

where Z_{sc} is length scale (= 40 for our study) and Q_{sc} is the scaling parameter for discharge. This means the lab discharges are 1/10119.3 times the discharges observed in the field. We applied this scaling parameter to one observed flood peak to come up with the peak value for our triangular hydrographs. The total duration of the experiment was also scaled from field hydrographs. m is the parameter shown in the Julien book which was used for scaling in this research.

$$m = \frac{1}{\ln(12.2 \frac{y}{d_s})} \cong 0 \quad (4.2)$$

where y is flow depth and d_s is particle grain size. As Julien, (2002) states if the parameter m is close, then we can use the equation below for scaling the duration of the hydrograph. Given that this parameter was close to zero, the time scale (T_{sc}) was calculated as:

$$T_{sc} = Z_{sc}^{0.5} = 40^{0.5} = 6.32 \quad (4.3)$$

Based on the above scaling, the duration of the lab hydrograph was 1/6.32 times the duration of the hydrograph in the field.

4.2.1.4 Sediment preparation

All experiments were begun with a prepared channel bed with well mixed sediments spread over the channel to a depth of 1 cm (Figure 4.3). Sediment size classes were weighed separately to ensure that the sediment mixture was consistent for the different experiments. The total mass of sediment added to the channel was also recorded to ensure consistency. The experiments were run without any sediment supply beyond what was already in the channel.

Sieve sizes used for this soil mixture are 0.3, 0.6, 1.18, 2.36, 4.0, and 5.6 mm. To assist with color threshold classification and in-flood video tracking of sediment in transport, a set of tracer particles were painted with fluorescent colors so that they appeared bright under ultraviolet lights while the rest of the sediment and water was dark (Figure 4.4). Preliminary assessments were completed using the MATLAB Color Thresholder application to determine which colors were distinguishable from water and the channel bed. Based on this preliminary assessment, tracers in the largest size fraction (4 to 5.6 mm) were painted pink, next size fraction (2.36 to 4 mm) were painted green, and a third size fraction (1.18 to 2.36 mm) were painted orange. Particles that were less than 1.18 mm were not painted for these experiments due to image resolution limits. The particles were painted with fluorescent color with a ratio of three drops of paint for a volume of sediments that fit in a cup. The thin layer of paint did not affect the particles characteristics such as size and surface roughness (Figure 4.4a). Based on visual observations of the final mixture and the desire to not overwhelm the accuracy of the particle tracking software, it was decided to paint 50% of the 4 to 5.6 mm class, 25% of the 2.36 to 4 mm, and 10% of the 1.18 to 2.36 mm. The particle colors were also visible under natural light as shown for the final dry mixture in Figure 4.4b.

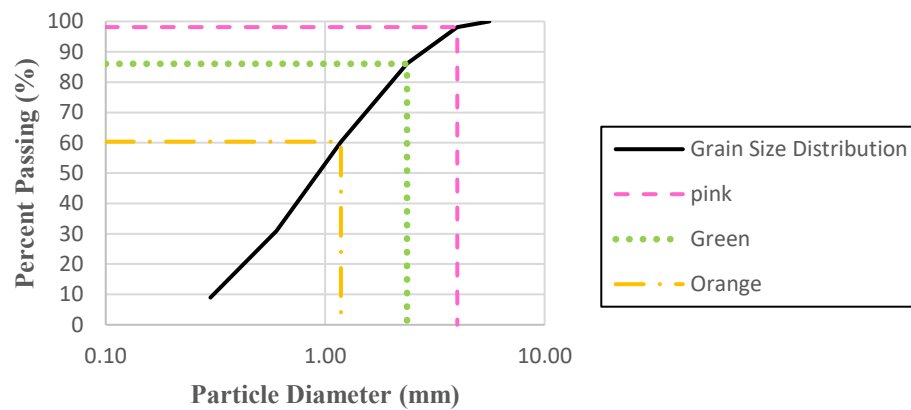


Figure 4.3 Grain size distribution for the particles used in the current study



Figure 4.4 a) The three sizes of painted tracers used for this study under regular light. b) The final sediment mixture used for the current study under regular light

The Froude scaling for the experiments ensures that the relative roughness (y/D) is preserved relative to the field scale, where y is the flow depth and D is a bed particle size. The dimensionless shear stress (θ) is also preserved, calculated as $\theta = \tau(\gamma_s - \gamma)^{-1}D^{-1}$, where γ and γ_s are the specific weights of water and the solid sediment, respectively, and τ is the bed shear stress calculated as $\tau = \gamma R_h S$, where R_h is the hydraulic radius (m) and S is the bed slope of the channel. As for the field scale, transport conditions were well above the threshold of motion ($\theta \sim 0.12$) and full mobility of all grain sizes was expected (Wilcock and McArde, 1993; Papangelakis et al. 2019). The Reynolds number was calculated as $Re = Vy\nu^{-1}$, where ν is the kinematic viscosity. At XS-A of the entrance area $Re = 8300$, which was sufficient to ensure turbulent flow. The boundary or ‘particle’ Reynolds number (Re_p) is relevant for scaling particle size relative to the shear boundary layer, calculated as $Re_p = D_{50}u^*\nu^{-1}$, where $u^* = \sqrt{\tau/\rho}$ and ρ is the fluid density. The small physical scale of the flume means that $Re_p =$

40 in XS-A, which is necessarily lower than the field scale but sufficient to ensure the flow is transitional rough turbulent. Given the transport conditions well above threshold, the impact of the scaling discrepancies between the field and laboratory scaling were thought to be minor.

4.2.1.5 Experimental hydrographs

Previous studies used various shapes of hydrographs to investigate the effect of unsteady flow on sediment transport. Triangular hydrographs are the most common (Lee et al., 2004; Wang et al., 2015, 2021), though some have used trapezoidal hydrographs (Bombar et al., 2011). Due to experimental limits of control, hydrographs are often run with stepped shaped and manual adjustment of valves to increase and decrease flows, though some have been able to model continuously varying hydrographs (Xiao et al., 2020). Given that the discharge peak for the modeled event is only slightly higher than an estimated discharge with a 2-year return period at the site of $19.35 \text{ m}^3/\text{s}$ (AECOM, 2011), we used this event as our design event for this urban creek. To simplify the hydrograph, we modeled it as a triangular hydrograph with linearly increasing rising limbs and linearly decreasing falling limbs.

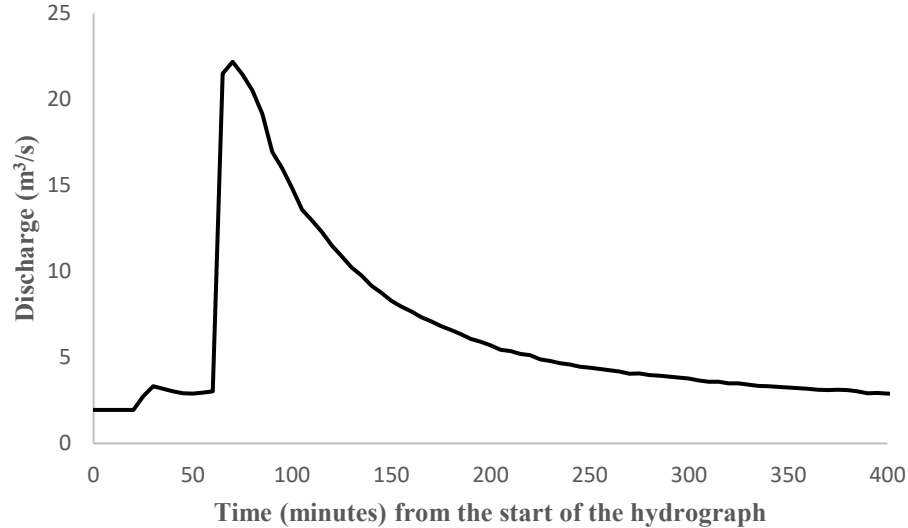


Figure 4.5 Field hydrograph recorded on June 2019 was used to design hydrographs for the laboratory experiments.

Based on a geometric scale of $Z_r = 40$, Q_r and T_r were calculated as 1.01×10^4 and 6.32, respectively, so that a peak model discharge of 2.16 L/s and an event duration of 24 minutes were set. The peak

discharge was thus just slightly higher than the steady state discharge from the Peirce et al (2021) experiments, which had been scaled to match a 2-year flood discharge at the site. To match the design hydrograph where HR (Hydrograph Ratio) = 0.064, the time of rise was set at 1.56 minutes.

To assess the effect of this rapid rising limb, we compared the results from the model event with three other hydrographs shapes with more moderate rising limb slopes. (Figure 4.6). A rising limb time of 8.6 min was selected based on a ratio $HR = 0.35$ that matched with typical floods on Ganatsekaigon Creek, which is located in the same geographical region as Wilket Creek but has a more muted response to floods because the watershed is relatively rural (Papangelakis et al, 2019; 2022, also see Chapter 2 and Chapter 3). A rising limb time of 12.0 min was chosen to represent a symmetrical hydrograph ($HR = 0.50$), while the T_r of 15.6 was chosen to represent an extreme case where the falling limb slope is steeper than the rising limb ($HR = 0.65$).

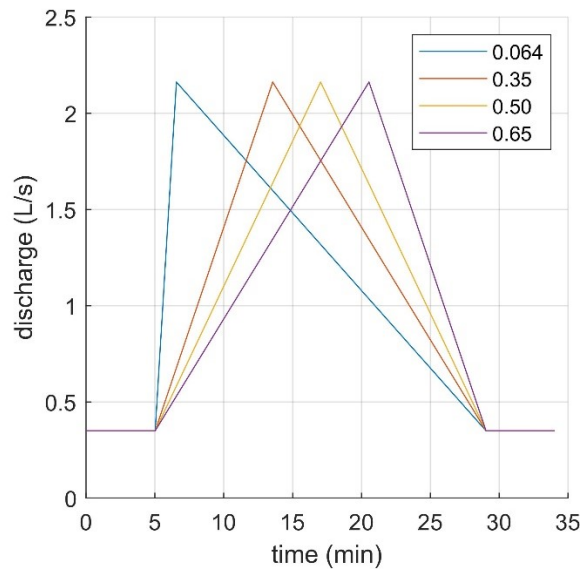


Figure 4.6 Triangular hydrographs used for the laboratory experiments. Legend shows HR values where H_r = time of rise/event duration. The total time of the hydrograph is 24 min, with 5 min of baseflow before and after the event for a total of 34 minutes.

4.2.2 Measurement and analysis

4.2.2.1 Exported sediment

Sediment output was collected in a set of removable mesh baskets at the downstream end of the flume. Baskets were switched through the experiment to allow export mass and size distribution from different

parts of the hydrograph to be compared. To standardize the weighing procedure, all baskets were allowed to drain for at least 120 seconds before measuring their wet weight (precision of 0.01 kg), and converted to a dry mass using a conversion factor for the field capacity of the sediment mixture. The conversion factor was verified by Peirce et al. (2021) and found to be 1.21. The total sediment mass was validated by removing, sieving, and weighing the sediment stored in the flume at the end of the first experiment. The sediment output was dried, weighed, and sieved for grain size analysis.

4.2.2.2 Topography

Digital elevation models (DEMs) and orthomosaic images were determined using Structure-from-Motion (SfM) photogrammetry following established techniques (Morgan et al, 2017; Leduc et al., 2019). The software package Agisoft Photoscan 1.4 was used for this purpose. A set of control points distributed throughout the flume on the outside of the channel was surveyed with a total station for image georeferencing. DEMs and orthomosaic images were exported at a grid resolution of 1 mm. The vertical accuracy of the DEMs was estimated at $2.4 \text{ mm} \pm 0.6 \text{ mm}$ based on the maximum depth of deposition, which is 5 mm.

4.2.2.3 Sediment Tracking with in-flood videography and color segmentation

Sediment mobility and transport was tracked during the flood by recording video within a region of interest in the channel, segmenting the video by color, and then applying a particle tracking software. Video could only be recorded for a relatively limited section of the flume, so a suitable area for testing was chosen where the water surface was relatively flat allowing a clear view of the bed transport during the experiment. The section is approximately straight but is located between two bends as shown on Figure 4.2b. Point bars [an accumulation of sediments inside bends of streams] were observed at both bends during the experiments and a cross-over riffle tended to form at this location, so it was thought that this location would also provide a suitable location for observing the changes to mobility and transport characteristics as a function of flood stage. The location is also far enough from the upstream and downstream ends of the channel so that it is relatively unaffected by the boundary conditions.

A Panasonic-DC-BGH1 camera was used to record this section for the entire length of the experiment. Camera settings were set to record at 60 frames per second (fps) and a resolution of 1080 x 1920 pixels. The camera was connected to a computer via Ethernet cable and controlled using the Lumix Tether for Multicam application supported by Panasonic. During video recording a black plastic sheet was used

to cover the camera cage to prevent external light pollution and the bed was illuminated with ultraviolet (UV) lights so that the painted particles were clearly visible (Figure 4.7a). Two UV lights suspended from the camera cage at elevations approximately 1 m above the channel was found to be sufficient. Lights were angled to minimize reflectance so that the water surface was nearly invisible.

To allow the tracking of sediment particles to be completed by size class, a custom MATLAB algorithm was created to allow users to extract a video segment from a longer video, generate color profiles, and apply the color profiles to the videos to create separate masked video files for each tracer color. Color profiles were generated using an interactive MATLAB tool called ColorThresholder, which allows a user to upload an image, select example areas with the desired color, interactively assess the quality of the masking, and export a function with the created color profile. Based on preliminary tests, the most effective option was to transform the RGB raw images into CIELAB ($L^*a^*b^*$) color space, where L^* , a^* , and b^* representing Lightness (0 – black, 100 – white), red/green value (large positive – red, large negative – green), and blue/yellow (large positive – yellow, large negative – green), respectively. To reduce visible errors caused by imperfect segmentation by colour, the masked images were post-processed using the image analysis algorithms called `imclose` (to remove small gaps between masked areas), `imfill` (to fill in areas that are completely enclosed by masked areas), and `bwareafilt` (to filter out small masked areas). Figure 4.7a-c shows a frame of the processed mask for each of the three tracer colors.

An open-source MATLAB based software called TracTrac (Heyman, 2019) was used to track the movement of the sediment particles. This software uses a Particle Tracking Velocimetry (PTV) approach, which is a Lagrangian technique that allows the statistics of the movements of each individual tracer to be extracted. In comparison with other PTV software, TracTrac is advantageous because it a) identifies objects with blob detection algorithms that allow for high particle densities and frame rates; b) allows for calibration of the object detection parameters for different applications and c) incorporates a motion model to predict the displacement of identified objects, which helps to reduce error, and d) has a GUI interface that it is easy to use. The input used for TracTrac are the video files where each image has been segmented for color as shown in Figure 4.7 b-d.

Five targets were attached to the side of the channel in the camera sight, which allowed the video image to be rectified by transforming pixels numbers into a local cartesian grid defined based on the flume dimensions. As for Figure 4.7, the grid origin is located at the corner of the flume near the far upstream

near right bank of the channel. To improve the observation of the movement of sediments in the videos and distinguish between areas of the bed that were covered with sediment from those that were bare, points on a 2 cm grid were painted on the bottom of the channel. The grid point color was made by mixing three colors that appeared brown color under normal light, but appeared as white under the black lights (grid points are visible in Figure 4.7a in the purple area top right and bottom left). To eliminate errors due to uneven lighting of the channel or apparent objects detected outside the channel boundary, the custom MATLAB algorithm designed to control the detection procedure allows for a region of interest to be defined.

Due to the large region of interest and large number of frames, the computational time was extremely high in early iterations of the analysis for the sediment tracking analysis. Due to the exploratory nature of the work and the need to refine the tracking methods, the procedure was only completed for seven one-minute sections of a video for one 34-minute flood with a symmetrical hydrograph ($HR = 0.50$).

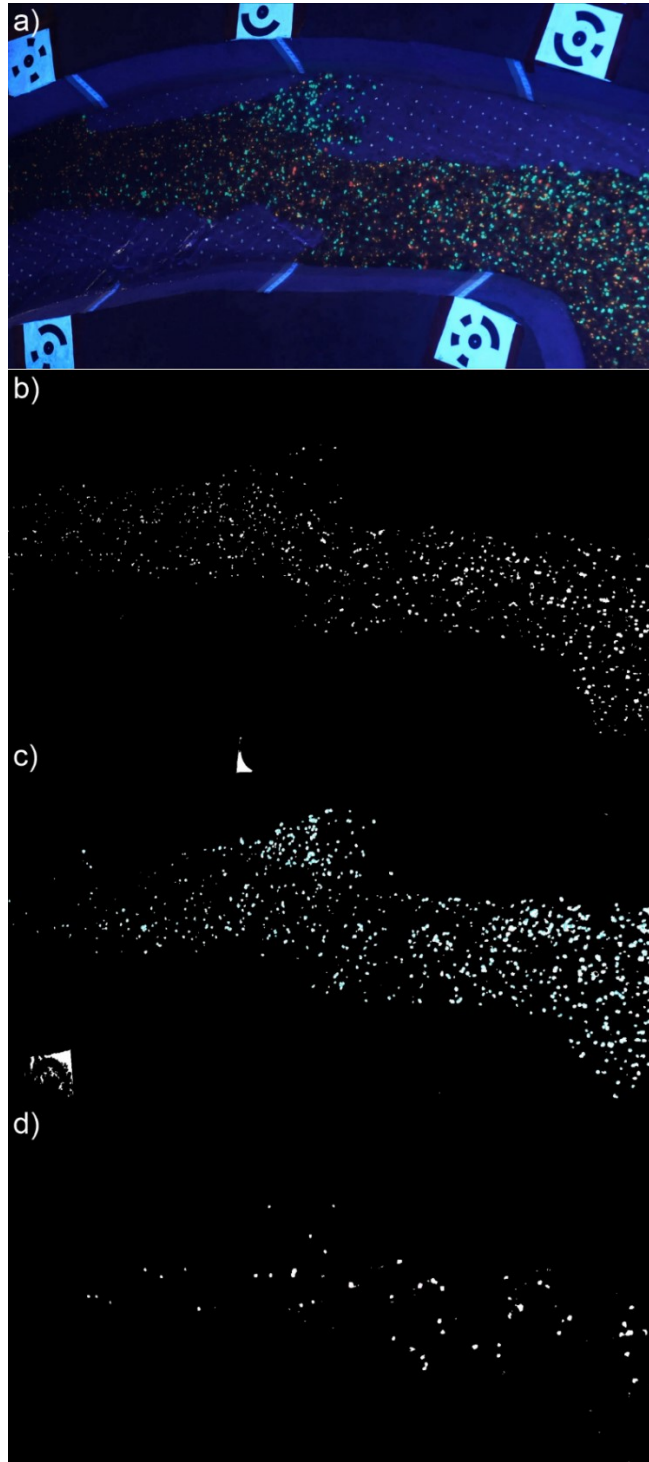


Figure 4.7 Image segmentation including a) a sample raw video frame; and segmented b) orange (1.18 to 2.36 mm); c) green (2.36 to 4 mm); and b) pink tracers (4 to 5.6 mm).

4.3 Results

4.3.1 Cumulative results

4.3.1.1 Export and cover

Total sediment export and the final cover fraction are shown for all experiments in Table 4.1. For comparison, the mean and the 95% confidence intervals (CI) are calculated for the repeated experiments with a symmetrical hydrograph (HR = 0.5). There is no trend of export mass with HR, as the lowest export was recorded for one of the HR = 0.5 experiments and the HR = 0.35 experiments. There is relatively high variability of export between identical experiments, as the HR = 0.50 tests resulted in a standard deviation equal to 16% of the mean sediment export. Similar conclusions can be drawn for the cover fraction, though the variability was less (standard deviation of 1.4% of mean value), and a possible trend was observed, with the highest value recorded for HR = 0.064. The trend is not significant however, so the general conclusion is that the hydrograph shape does not affect the exported mass or the final cover fraction.

Table 4.1 - cover fractions reported are relative to the starting cover fraction of top of bank areas painted as green in Figure 4.2b (average of 0.761 with a standard deviation of 0.010).

Experiments	HR	Export (kg)	Final/Initial Cover Fraction
PTH1_1	0.5	5.512	0.747
PTH1_2	0.5	4.330	0.754
PTH1_3	0.5	5.953	0.733
Mean $\pm$ 0.95CI for HR = 0.5	0.5	5.26 $\pm$ 1.65	0.745 $\pm$ 0.021
PTH2_1	0.064	5.480	0.766
PTH3_2	0.35	5.289	0.746
PTH4_1	0.65	5.537	0.740

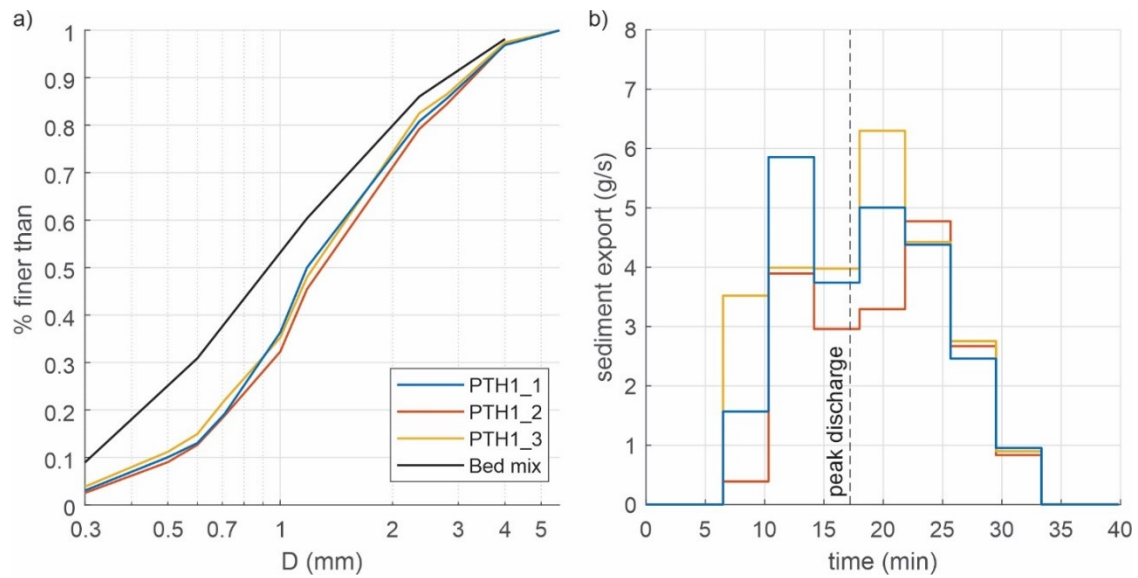


Figure 4.8 Comparison of sediment export from three repeat runs of the symmetrical hydrograph (HR = 0.5) including a) total sediment size distributions in comparison with bed mixture; and b) mass export as a function of time.

A comparison of sediment export from repeat experiments with a symmetrical hydrograph shows a relatively coarse export sediment mixture, with highly repeatable results (Figure 4.8a). Analyzed over time we can see that the mass export roughly follows the hydrograph shape, with an unsteady peak sediment export rate occurring close to the same time as the peak discharge value. Peak export rates of ~4-6 g/s correspond with the range of sediment feed rates tested by Peirce et al (2021), who also estimated that the sediment transport capacity of the channel (Q_c) was 7.9 g/s based on a value that was measured from a straight channel with a mobile equilibrium bed and then scaled to the current channel based on a Meyer-Peter Muller formula for sediment transport.

The most interesting result for a discussion of hysteresis is the high variability on the rising limb in comparison with the falling limb. Over a 4-5 minute period between 5 and 10 minutes, for example, the export rates from the three cases vary from < 0.5 g/s to ~3.5 g/s. Similar magnitudes of variability occur until after ~22 min, at which point the variability decreases to <0.5 g/s for the three cases, even while the export rate remains relatively high on the falling limb. This variability helps to explain the large differences in total export that were observed for the three cases (Table 4.1). The low total mass export for the PTH1_2 experiment is shown to have relatively low export rates on the rising limb in comparison with the other experiments with matching initial conditions and flood hydrographs. Sediment export on the falling limb, in contrast, matches closely with what was observed in the repeats.

The low transport on the rising limb in PT1_2 is not easy to explain, but the difference may be related to differences in the self-organization of the bed sediments in this particular run.

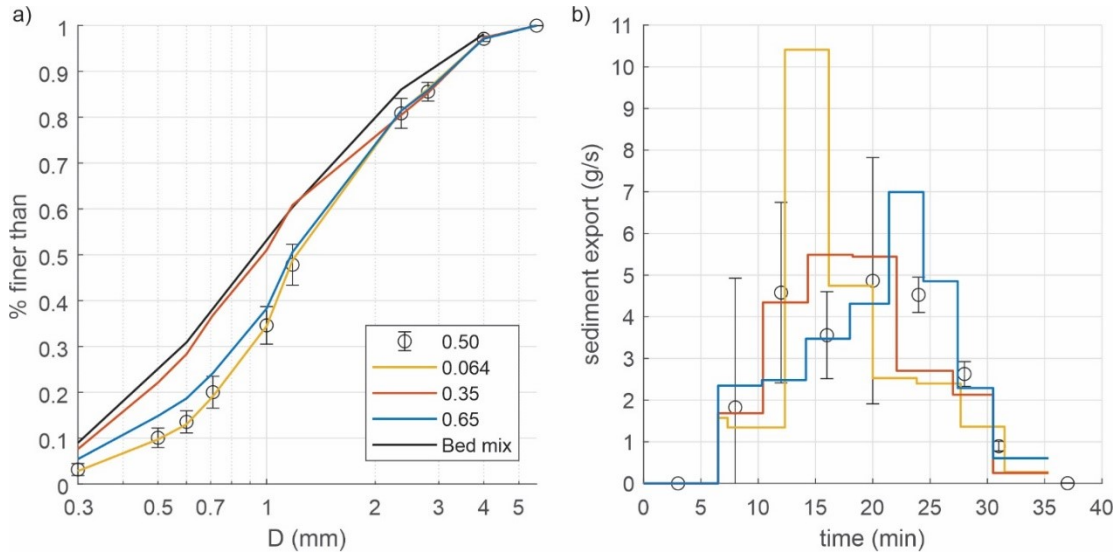


Figure 4.9 Comparison of sediment export from three runs with different peak time ratios (HR) including a) total sediment size distributions in comparison with bed mixture; and b) mass export as a function of time. Error bars are based on the mean and 95% CI from the repeated experiments with HR = 0.50.

Comparison of sediment export from experiments with variable HR show that, similar to the HR = 0.5 experiments, all exported distributions were coarser than the mixture used to form the initial bed of the channel (Figure 4.9a). Despite this similarity, however, the export size distributions were more variable for the experiments with different HR values and there is no discernable trend to the increase in variability. For example, the most extreme event, which had a very rapid rising limb (HR = 0.064), had a good match with the HR=0.5 size distribution, but an event that deviated much more mildly from the symmetrical flood (HR = 0.35) showed a significant deviation for sediment sizes < 2 mm. For both the HR = 0.35 and the HR = 0.65 experiments the exported sediment was relatively fine in comparison the HR = 0.50 experiments. The lack of a trend with HR values suggests that the repeatability observed for the repeat HR = 0.5 experiments may have been a result of the low number of repeats.

Despite the overall similarity in sediment export and size distributions, the HR = 0.064 experiment did produce the largest pulse of sediment exported from the channel (Figure 4.9b). This peak of > 10 g/s exceeded the 95% CI values of the HR = 0.50 experiments. In contrast, the peaks observed for the other hydrographs did not exceed the range of values observed in the HR 0.50.

4.3.1.2 Export by size fraction

To understand how different size fractions were responding to the flood hydrographs we calculated the output ratio of each size fraction (P_p) as

$$P_p = \frac{\text{Percent sediment out}}{\text{Percent sediment in}} \quad (4.4)$$

where the ratio shows the amount of sediment exported the channel for a fraction over the amount of sediments inputted into the channel before starting the experiment for the same sediment fraction. Results for the repeated symmetrical hydrographs show that the output sediment has a different size distribution than what was placed on the bed (Figure 4.11). The coarsest fractions are generally over-represented, particularly near the peak discharge, while the smallest fractions are underrepresented except in the very end of the experiments when the discharge returns to near baseflow conditions. The particle sizes nearest the D50 (0.6-2.36 mm) are relatively consistent through the flood, with P_p slightly higher than one, indicating a slight over-representation of the output. Though represented in Figure 4.10 in only three size fractions, which helped to simplify the presentation, the observed patterns are robust for finer size classifications as well, with the general pattern of more equal representation of all sizes at the beginning and end of the flood and separation between sizes at high flow. Variability between experimental runs is stage and size dependent, with the highest variability observed for the coarsest particles on the rising limb. Variability reaches a minimum for all size fractions at ~ 22 min, which is after the peak flow on the falling limb.

To assess the impact of the asymmetric hydrographs, P_p was calculated and compared with the 95% CI lines from the repeated HR=0.50 experiments (Figure 4.10). The HR = 0.064 experiment is notable for the high transport of coarse particles that occurs at the onset of the experiments ($P_p > 2.0$), which exceeds the CI from any time interval other than the end of the HR = 0.50 experiments (Figure 4.11a). This high coarse transport is mirrored by an extreme under representation of the finest size fractions in the exported sediment ($P_p < 0.25$). The middle size class (0.6 – 2.36 mm) is remarkable only for remaining largely within the 95% CI lines set by the symmetrical hydrographs despite the difference in hydrograph shape. The results from the HR = 0.35 experiment are unusual because P_p for all sizes < 2.36 mm generally lie outside of the 95% CI lines set by the HR = 0.50 experiments at all times. Relatively high transport of fines occurred in this experiment, even through the peak of the flood. P_p for the finest fraction even exceed that of both other size fractions at one point on the falling limb (~22 min), a phenomenon that was not observed in other experiments. The unusual behavior seen for the HR

= 0.35 experiment was not repeated for the HR = 0.65 experiment (Figure 4.11c). Instead this final experiment largely matched the results from the HR = 0.50 experiments, with only the finer sediments occasionally slightly over-represented in the output.

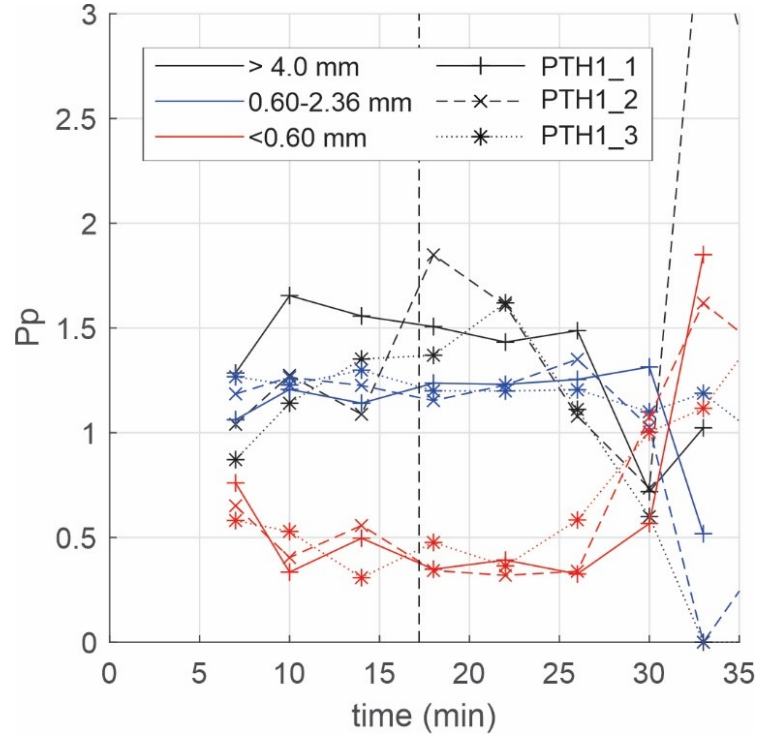


Figure 4.10 Output ratio (P_p) of three size fractions for experiments with symmetrical hydrographs. Peak flow is indicated at 17 minutes. Size fractions with $P_p > 1$ are over represented, meaning that there is a higher percentage of that size fraction leaving the flume in comparison with the initial bed cover placed in the channel, while fractions with $P_p < 1$ are relatively under-represented in comparison with the initial bed cover.

A second representation of sediment exported by size was created to better represent the mass export. For each time interval the output mass of each size fraction ($Q_{s(out)}$) was compared to the output mass that would be expected if the total mass exported during the flow followed the size distribution of supply ($Q_{s(initial)}$):

$$P_{Qs} = \frac{Q_{s(out)}}{Q_{s(initial)}} \quad (4.5)$$

Note that the cumulative values of this metric will be greater or less than 1 depending on the overall mass of sediment exported, which were shown to be coarser than the initial bed mixture. A value greater

than 1 means the amount of sediments leaving the channel is higher than the amount of sediments inputted in the channel.

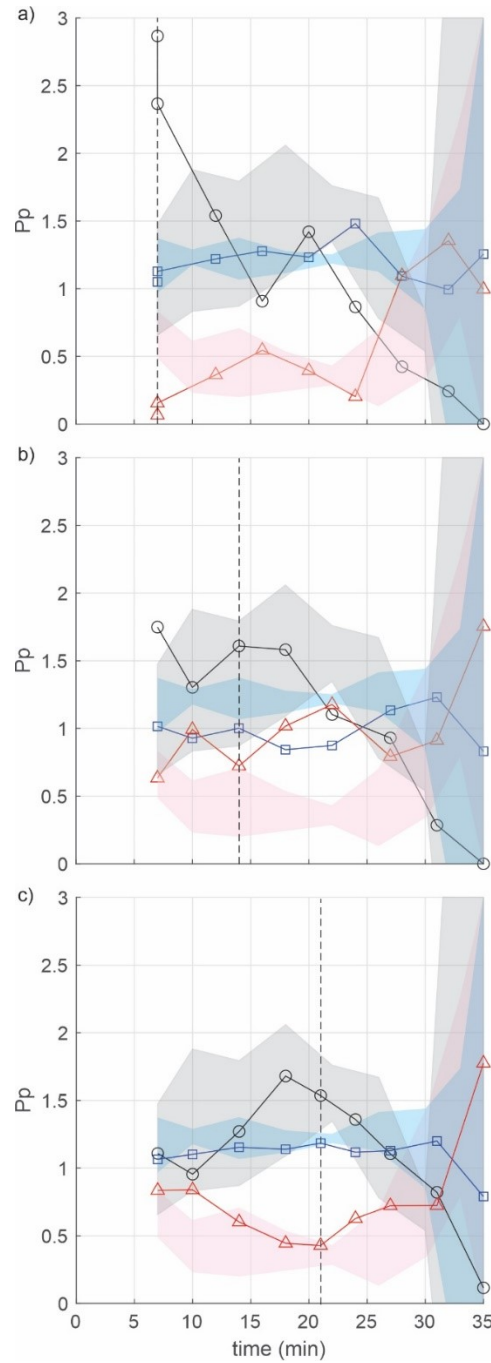


Figure 4.11 Output ratio (P_p) of three size fractions for experiments with asymmetric hydrographs: a) $HR = 0.064$, b) $HR = 0.35$, and c) $HR = 0.65$. Peak flow times are indicated with dashed lines. 95% confidence intervals from the symmetrical hydrographs are included for each of the size classes for comparison.

The key observation from this plot is that the coarseness of the sediment export at the peak floods is more evident than the earlier representation as a percentage. The importance of the peak is emphasized because most of the export occurs during the peak and the visual importance of the small amounts of material that are exported by water draining from the flume after the pump is stopped is minimized. The peak value for the largest particle size class is the same for $HR=0.36$, 0.5 , and 0.65 . However this number increases to higher than 0.8 for $HR=0.064$. For the two highest HR values, the peak of PQs occurs when peak flow reaches the downstream end of the channel. When the rising limb is short ($HR=0.064$ and 0.35), the peak in PQs occurs after the peak discharge. When the rising limb is long ($HR = 0.5$ and 0.65), the peak in PQs is coincident with the peak discharge.

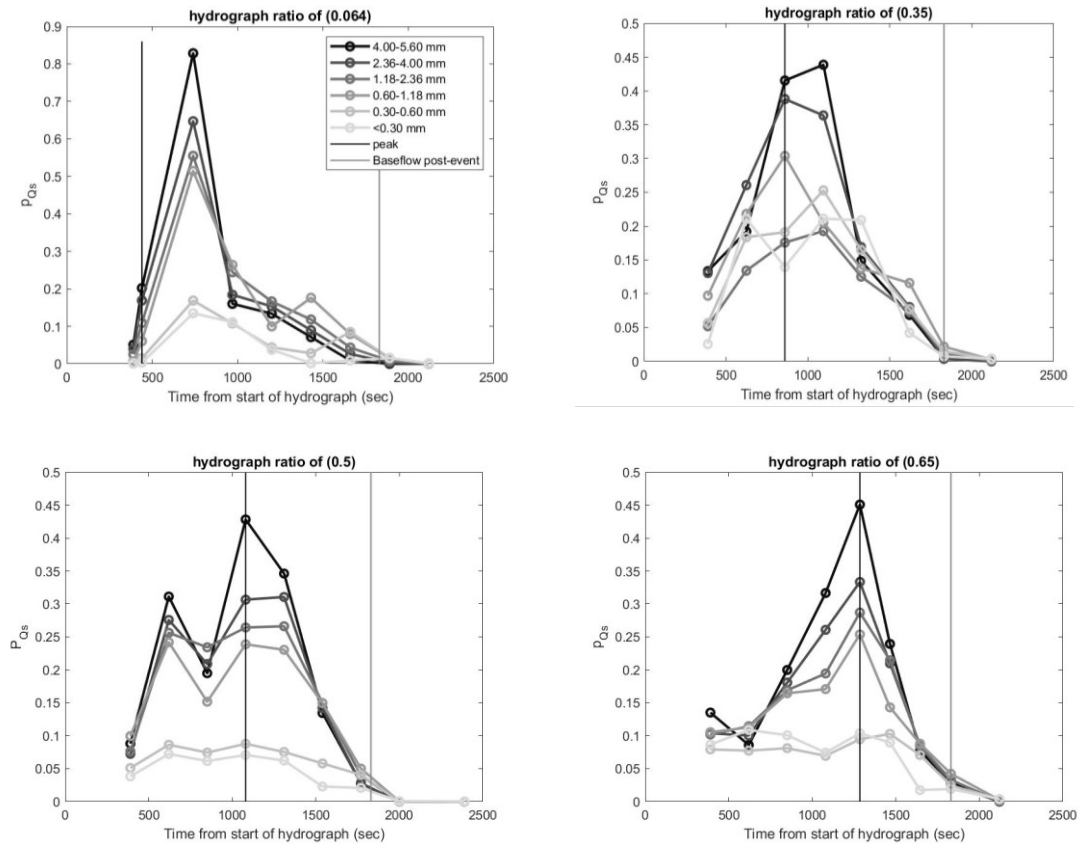


Figure 4.12 Mass output ratio (P_p) of six size fractions for experiments with symmetrical hydrographs: a) $HR = 0.064$, b) $HR = 0.35$, c) $HR = 0.50$, and c) $HR = 0.65$. Peak flow times are indicated with solid lines.

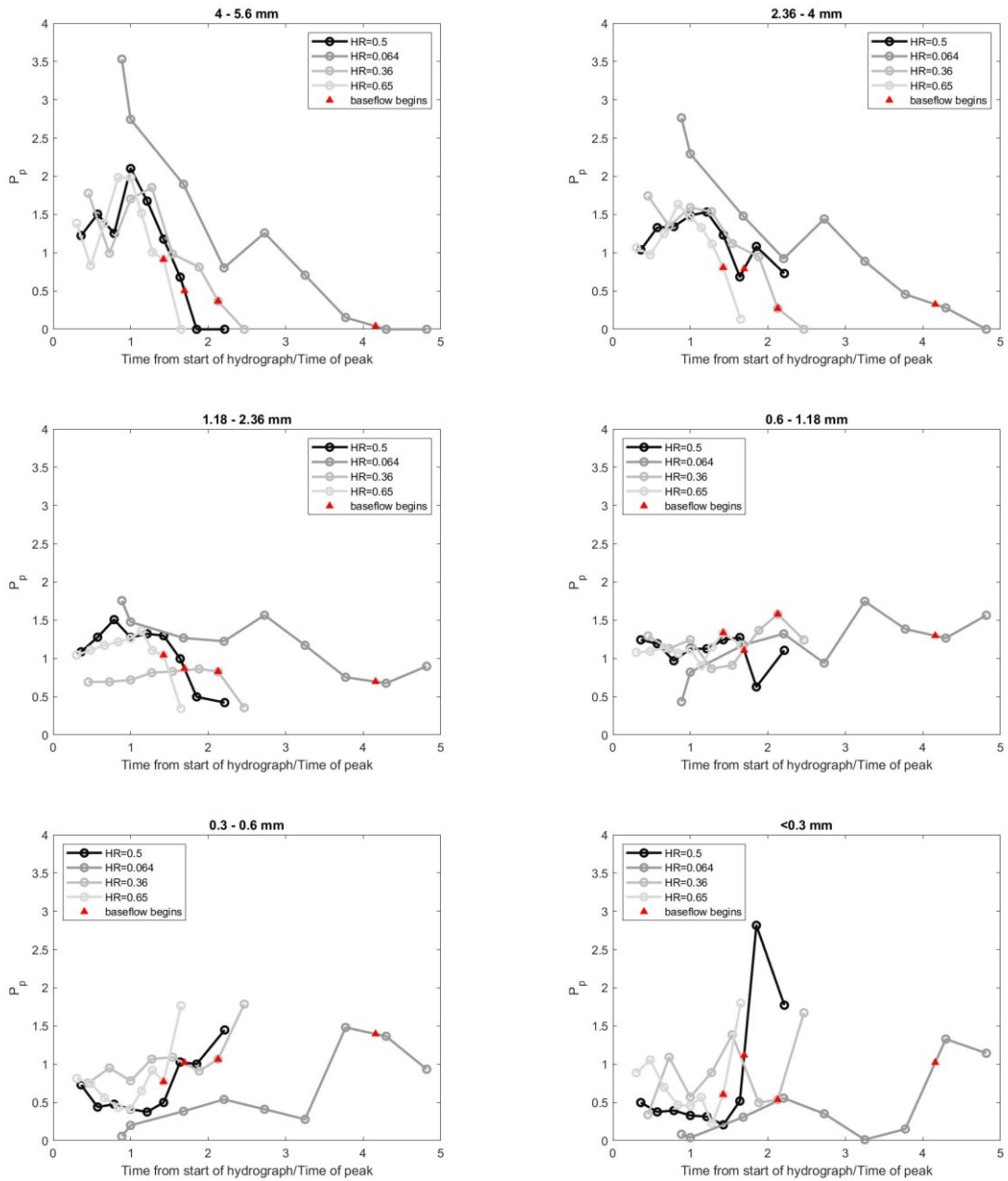


Figure 4.13 Comparison of output ratios (P_p) of six size fractions where the time axis has been normalized by the time to peak to better compare the rising and falling limbs for experiments for the different hydrographs. Where $P_p = 1$, the percentage of the particles leaving the flume is the same as the percentage of the particles placed in the channel as initial bed cover. Red triangles indicate the times when baseflow begins for the respective hydrographs.

When the horizontal axis is normalized by the time to peak to better compare the rising and falling limbs, it is evident that the hydrograph with a ratio of 0.064 is distinct from the others hydrographs, with coarse particles generally over represented on the falling limb. For the hydrograph ratio of 0.064 the percentage of large particles are much higher than for the rest of the hydrographs. The percent out/percent in for this hydrograph exceeds 3.5 for the largest particles (Figure 4.13). The higher the rising limb slope, the faster time to peak and the higher the volume of larger particles leave the channel. This shows that increasing the rising limb slope leads into having more large particles leave the channel and more smaller particles stay in the channel. This means that the remaining particles at the end of each experiment should have different sediment grain size distribution.

4.3.2 Cover

Image analysis was used to assess changes in cover and distribution of coarse sediments between the different experiments. To extract the relevant data, the area within the channel was divided into non-overlapping ‘pizza-slices’ using the algorithm developed by Peirce et al. (2021). Within each pizza slice we compared the number of pixels covered at the end of the experiment $((NP_s)_F)$ to that at the beginning $((NP_s)_I)$, and normalized by the total number of pixels in each pizza slice (NP) :

$$P_{Ns} = \frac{(NP_s)_F - (NP_s)_I}{NP} * 100 \quad (4.6)$$

In these figures positive values show deposition and negative values show erosion. When the values become zero, it means that the final bed cover is that same as the initial bed cover. An image from the final bed of the PTH_1 experiment is shown in Figure 4.14a. Guidelines A-K have been added at the upstream and downstream limit of each bend to assist with the description of the results. The channel bed was colored blue and the blue parts in the channel shows the washed out parts of the channel. Looking at the figure it can be seen that we have highest value of erosion between D and E and between H and J, which are repeated bends, with slightly more erosion for the downstream bend (Figure 4.14). Overall, however, the bed remained relatively covered ($< 20\%$ change) and there was little difference between the runs with the different hydrograph shapes. In the bend between D and E, it can be seen that the experiments with highest erosion are the experiments with $HR=0.65$ and $HR=0.5$. The experiment with least erosion in this part of the channel is the experiment with $HR=0.064$. This shows that lower rising limb slope leads into lower erosion in this part of the channel. Between H and I it can

be seen that erosion in the channel is highest for the experiment with $HR=0.65$. This shows that decreasing the rising limb slope leads to higher erosion.

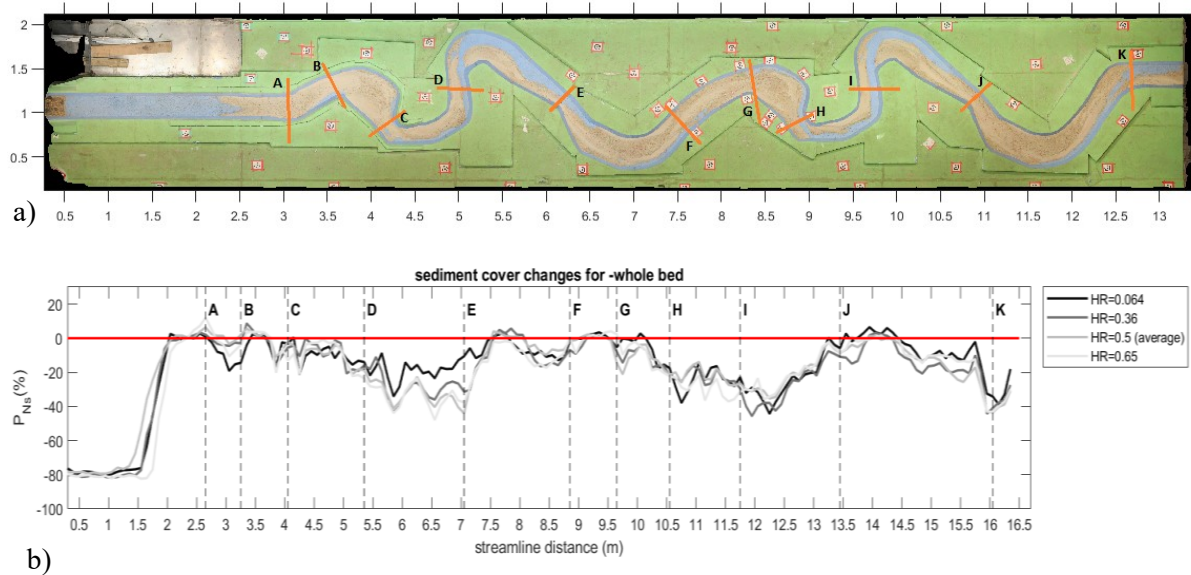


Figure 4.14 a) Image from the final bed of the PTH_1 experiment. Guidelines A-K have been added at the upstream and downstream limit of each bend to assist with the description of the results. B) sediment cover change for the channel bed (P_N s) comparing runs

Due to perceived differences in bar construction on the two sides, the pizza slice algorithm was run a second time and the data was plotted separately for each side of the channel relative to the centerline (Figure 4.15). This presentation shows that some of the areas without much of a change on the previous plot are in fact areas of erosion and deposition. Between C and D, for example, an area that was slightly erosional taken as a whole has accumulation on the left inner bank of the bend and erosion on the outer bank. Between D and E the opposite occurs due to the inversion of the inner and outer banks. The bend between D and E also shows some of the most significant differences as a function of hydrograph shape. For hydrograph with $HR=0.064$, the left side experiences the least erosion but the right side experiences the most (Figure 4.15). This means that the inner bar is smallest for this case with a rapid rising hydrograph. This pattern is repeated between sections I and J. We have the least percentage of bed cover for this experiment, which means that the bar created on the right side is smaller.

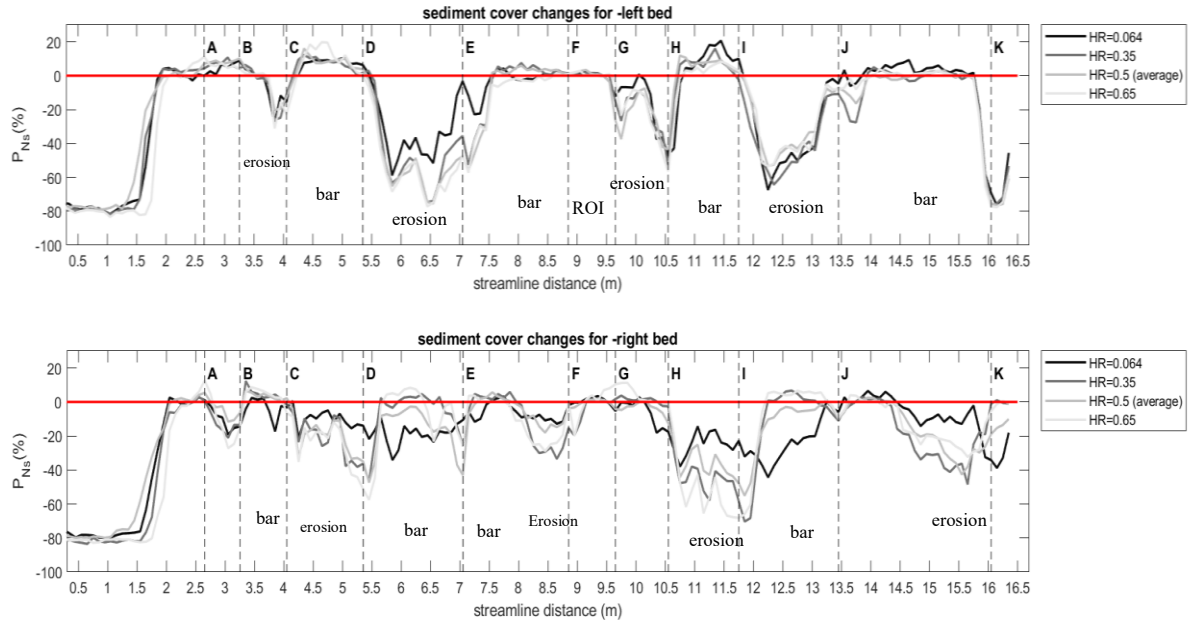


Figure 4.15 Sediment cover change (PNs) as a function of streamwise distance on the a) left and b) right side of the channel. Lines A-K indicate sections defined on the orthoimage of the channel (Figure 4.14a) that were added at the upstream and downstream limit of each bend to assist with the description of the results. Areas of bar growth and erosion are marked based on the visual impression of bar development from the orthoimage.

The pizza slice algorithm was also used to find concentrations of coarse particles. Analyzed size classes included the pink (4- 5.6 mm) tracers and the green (2.36-4 mm) tracers. Plotted over the entire bed, the highest number of pink particles at the surface were found in the BC, EF, GH, and JK bends (Figure 4.16). These bends are the widest in the channel, with BC - GH and EF – JK as repeated bends in the planform. In the BC-GH bends, more pink particles accumulated on the left side of the channel compared to the right side. This bend has a tight inner radius and the flow is concentrated on the left side. In contrast the right side is characterized by flow separation, and fine particles tend to accumulate in this area and cover the coarse sediments. In the EF-JK bends there is not a clear flow separation zone and pink particles tend to occur on both the left side and on the right side.

There are only a few clear differences between the experiments with different hydrographs. One unusual result is the large number of particles in JK bend for HR=0.5. Similar size particles tend to accumulate in this area in all runs, but for this particular run, approximately twice as many pink particles were found in this area. The bend CD is also interesting because a high number of pink particles

accumulated on the right side of this channel, but only in runs $HR = 0.50$ and $HR = 0.65$. The same trend does not occur in the repeating HI bend.

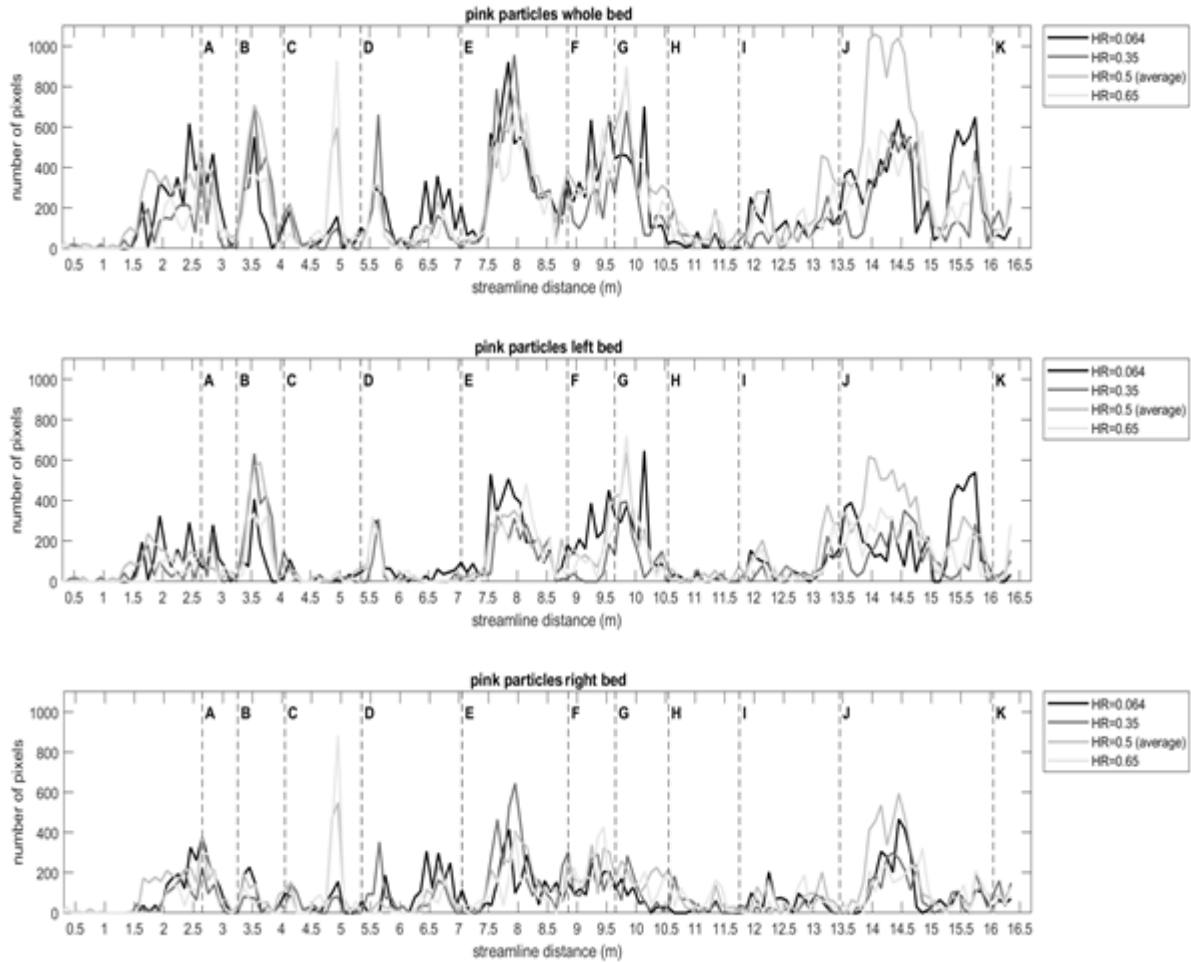


Figure 4.16 Pixels occupied by pink particles (4- 5.6 mm) as a function of streamwise distance for the a) whole bed, b) left side, and c) right side of the channel. Lines A-K indicate sections defined on the orthoimage of the channel (Figure 4.14a) that were added at the upstream and downstream limit of each bend to assist with the description of the results.

Similar trends are seen for the coarse green particles as were noted for the pink particles. One notable difference occurs in bend J-K, where fewer green particles were found except close to section K, where these particles tended to accumulate on the left bank for $HR = 0.064$ (Figure 4.17). A second point to note is that the accumulation on the right side of bend CD contains many green particles and that a clear trend emerges

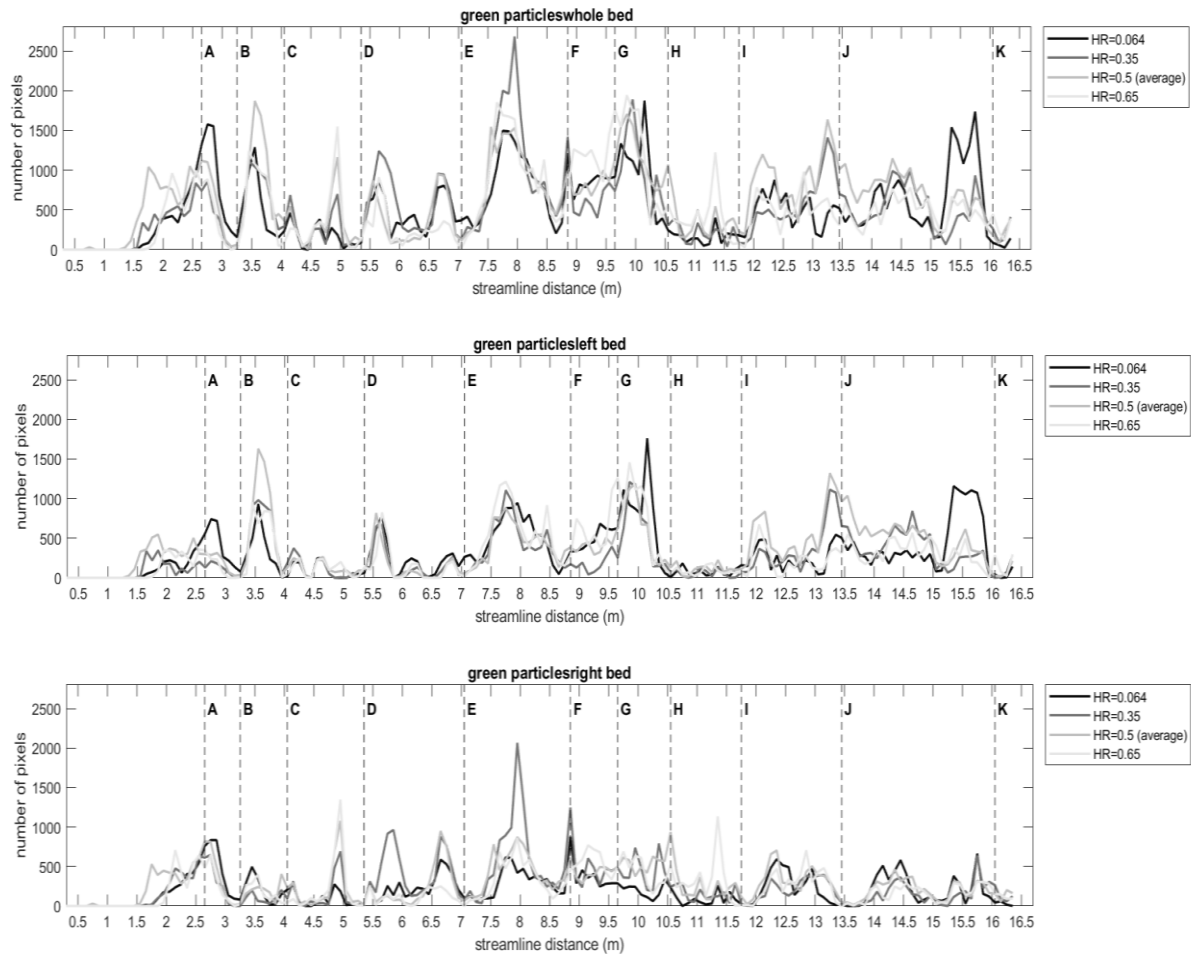


Figure 4.17 Pixels occupied by green particles (2.36–4 mm) as a function of streamwise distance for the a) whole bed, b) left side, and c) right side of the channel. Lines A-K indicate sections defined on the orthoimage of the channel (Figure 4.14a) that were added at the upstream and downstream limit of each bend to assist with the description of the results.

4.3.3 Topographic change

The Structure from Motion (SfM) technique is able to produce high quality measurements of channel topography. Based on a comparison between the beginning and end of the runs, we were also able to create highly detailed DEM of Difference plots, an example of which is shown in Figure 4.18. Plotted at the full scale it is difficult to see much difference between the runs, which is why only one plot is shown. The bars are clearly visible as yellow orange and red, with the highest bars occurring in bends

BC and GH where the channel is wider and the bend is relatively tight. In many areas a blue thalweg is visible, which indicates that most or all of the sediment was eroded and the underlying fixed bed of the channel was exposed. In some areas, no blue channel is noted, which indicates that the bed remained covered across the entire width of the channel. The experiment with the longest rising limb was also notable because of sharp dropoff edges for some bars in comparison with the plotted run (HR=0.064), where the bars were smoother in shape.

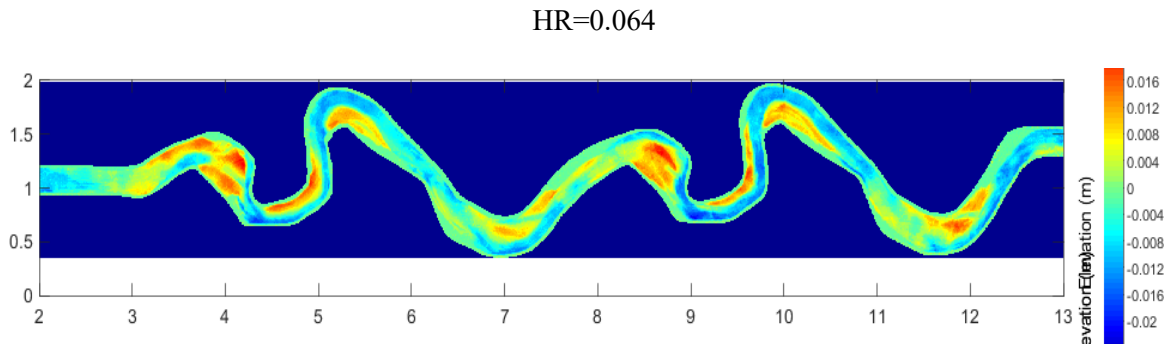


Figure 4.18 DEM of Difference (DOD) for run HR = 0.064.

To assess the overall final bed elevations, the hypsometry or depth of sediment for the final topography was calculated relative to a bare bed (Figure 4.19). In this representation the percentage of the bed (PI) with any relative depth of sediment (l/y) greater than or equal to the x axis value. The percent cover is the intercept of the line with the y-axis. The results appear highly repeatable because the 95% CI error bars for the repeated HR = 0.50 runs are relatively small, and no significant difference from the results with different hydrograph shapes are shown. CI bounds are largest at sediment depths that are less than the estimated DEM error. The vertical line on Figure 4.19 shows the initial bed that occurs when $l/y = 0.50$. The error bars are also somewhat larger close to $l/y = 0.50$, which corresponds with the initial depth of sediment for the experiments. Inflection points occur above and below $l/y = 0.50$ so that two distinct segments of the hypsometric curves are visible. For $l/y < 0.50$ these depths are erosional whereas depths where $l/y > 0.5$ would have to have been formed as a result of deposition. For comparison, the models fit by Peirce et al. (2021) for equilibrium beds formed as a result of deposition at different sediment feed rates are shown. Based on this comparison, the segment below $l/y = 0.50$ is not significantly different from the 4 gs^{-1} model, while the segment above $l/y = 0.50$ fits between the 2 gs^{-1} and the 4 gs^{-1} models (Figure 4.19).

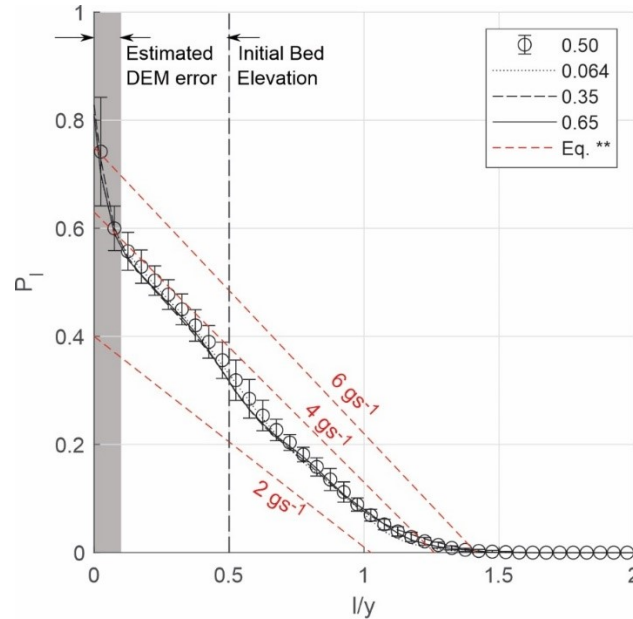


Figure 4.19 Hypsometry of bed relative to the initial flow depth in a bare bed channel ($y = 0.020$ m) as determined by Peirce et al 2021. Error bars represent the mean and 95% CI of the PTH1 experiments where $HR = 0.50$. Models shown are for equilibrium beds with different sediment feed rates (Equation 2) from Peirce et al. 2021. The estimated DEM error and the initial bed depth of sediment (~ 0.010 m) are also shown for comparison.

Despite a lack of global differences on the hypsometric plot, local differences were noted between the run in topography and in the distribution of coarse particle sizes (Figure 4.16 and Figure 4.17). To see the difference in greater detail, DOD and cover classification plots of particular bends were created to compare different runs. Bend CD was earlier noted as one that showed differences in the accumulations on the right side or outer side of the meander depending on the duration of the rising limb. As shown in Figure 4.20a, there is a continuous eroded channel on the right side of the channel for the case with the rapid rising limb ($HR = 0.064$) and less net erosion at the apex of the bend for the case with the longer rising limb. Differences are also visible on the inner bank of the bend, where less net deposition occurs for the longer rising limb. The cover plots show that the inner bank is composed of fines as there is a large area without any particles > 2.4 mm, while the larger particles are concentrated close to the apex of the bend (Figure 4.20c and d). For the $HR = 0.064$ run, however, there is a continuous length of the channel without cover in this area (Figure 4.20c). In contrast, there is an accumulation of particles > 2.4 mm at the apex of the bend for the $HR = 0.65$ run (Figure 4.20d). This accumulation reaches the base of the bank slope so that mobile particles have to pass over it in order to continue downstream.

DOD and cover classification plots were also created for bend GH to compare the differences between the bars formed in this wide area downstream of a sharp turn on the inner bank and a significant narrowing downstream (Figure 4.21). Significant differences were observed between the erosion and bars formed in the different runs in this section. In the run with the rapid rising limb ($HR = 0.064$), for example, the bar was characterized by two main lobes, with the most deposition occurring in a teardrop shaped bar in the middle of the channel. The bar deposited relatively close to the area where erosion led to the formation of a clear channel thalweg (Figure 4.21a). A second bar formed at the toe of the slope on the inner bend and is highest close to the channel wall. Between these two bars is a large area of channel deposition that is somewhat lower than the two main lobes. In the run with the milder rising limb slope ($HR = 0.65$) the bar is dominated by a large central bar with a steep drop-off slope. The area downstream of the drop off experienced no deposition or erosion during the run and so appears as smooth and green in this plot. A smaller lobe near the inner bank is also observed for this run. The different character of the bars is shown by the image segmentation plots, which shows that no coarse particles are visible at the surface of the bar lobes that are attached to the inner banks (Figure 4.21c and d). The fine texture of the inner bank bars contrasts with the coarse central bars, which have many pink and green particles at the bed surface at the end of the run. Careful comparison between the plots shows that the areas of the central bars with significant deposition match closely with the highest areas of the bars. This is particularly true for the upstream sides of the central bars, which are very coarse. Lenses of fine sediment occur in the areas between the highest points of the bar and the thalweg. Accumulations of coarse sediments are also intermittently seen right at the edge of the cover/bar bed interface. A larger accumulation can be seen at $x = 8.9$ and $y = 1.3$ m in run $HR = 0.064$. This accumulation touches the toe of the outside bank, similar to what was observed for run $HR = 0.65$ in bend CD. As was seen in the previous example, the deposit tends to form where the bend radius is smaller, i.e. at the tightest part of the outer bend.

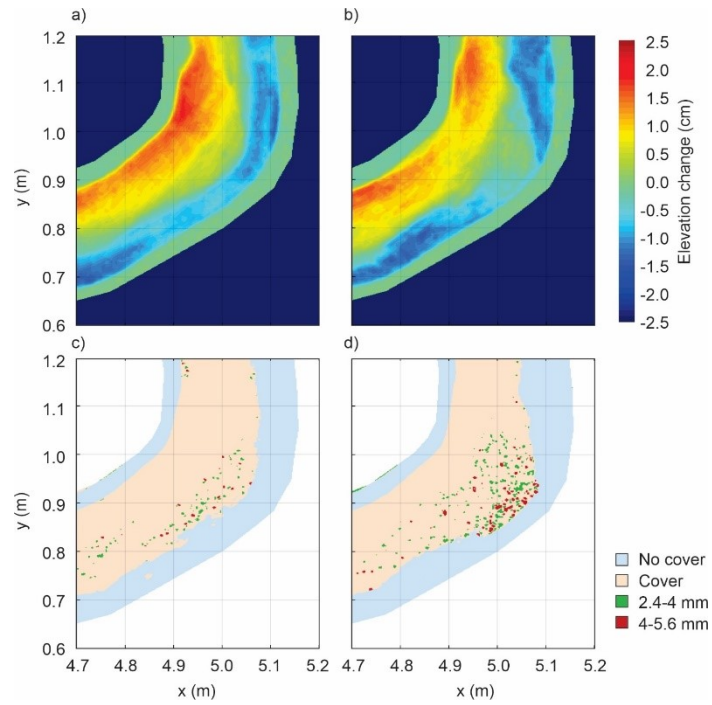


Figure 4.20 Bend CD showing a) DOD for run HR 0.064; b)DOD for run HR 0.65; c) cover plot for HR 0.064; and d) cover plot for HR 0.65.

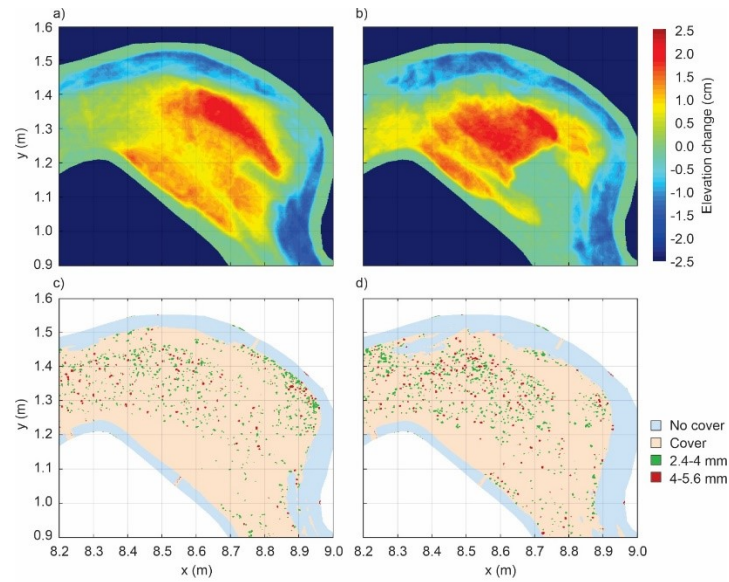


Figure 4.21 Bend GH showing a) DOD for run HR 0.064; b)DOD for run HR 0.65; c) cover plot for HR 0.064; and d) cover plot for HR 0.65.

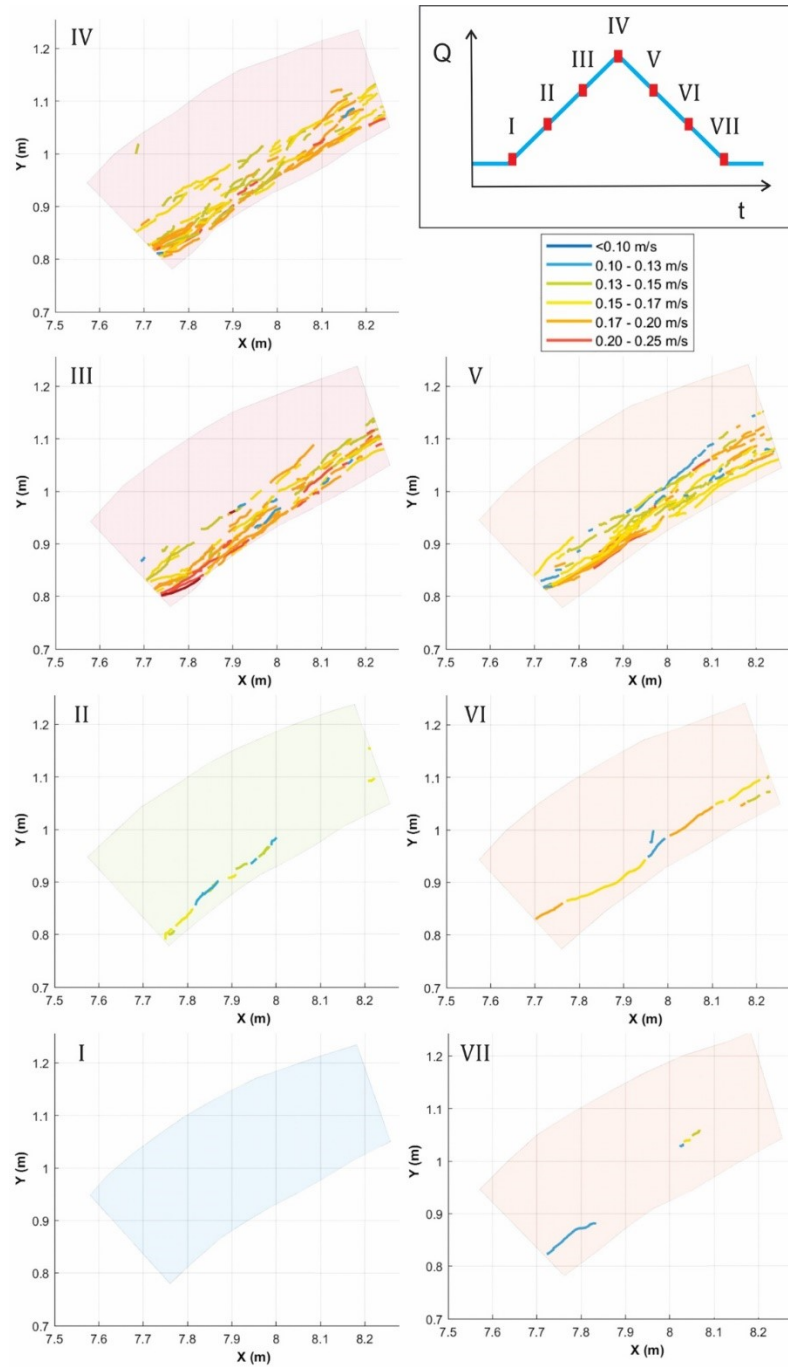


Figure 4.22 Coarse (4.0 – 5.6 mm size particles) pink tracer tracks and velocities for 30 s intervals at different points in the symmetrical hydrograph starting from beginning of the rising limb (time interval I) to peak flow (IV) and the end of the falling limb (VII). The inset shows the time intervals for each of the track plots. Flow is from left to right.

4.3.4 Tracking pink particles

The tracking analysis was only applied to the pink particles due to some challenges with the method. Nevertheless, the results clearly show that the sediment transport is highly variable in space and over the hydrograph (Figure 4.22). No movement occurs during time interval I at the beginning of the rising limb. During time interval II, transport is rare, consisting of a series of short tracks with relatively low velocity that terminate within the ROI. The tracks may represent the motion of multiple grains, or they could be the continuous motion of a single particle that the algorithm is extracting as a series. This uncertainty was the main reason the analysis was not applied to the other particle sizes. It appears that the particle motion model used in the TracTrac algorithm is not suitable for tracing the particle movements in the videos obtained for the current study. Further investigation of these issues is warranted to refine the color segmentation, identify suitable ranges for parameters in the TracTrac algorithm, and assess the need for post-processing of the results. As flows increase through time interval III, the tracers move more frequently and at higher velocities, but the transport is still highly localized to the right side of the channel where flow is concentrated. Some lateral spreading over the bar is visible, but the area of active transport is still restricted to the right side of the channel where the water flow is laterally concentrated due to the effect of the upstream channel bend. In the upper half of the channel the large particles that began in that area remained undisturbed. These trends continue through the peak of the flood (time interval IV), where only isolated motions occur in the upper half of the channel. There is no clear difference between time intervals III and IV, with the possible exception that velocities appear somewhat faster at the lower discharge in III. The falling limb of the flood appears relatively similar to the rising limb, with again only a visual impression that the velocities are slower on the falling limb (for e.g. by comparing ti III and V). At lower discharges, some differences are noted, but the number of tracer particles is too low to assess whether these differences are significant.

A DOD and cover plot were made for the final topography for run PTH1_1 to compare with the particle mobility results. As expected, the lower region of the ROI is an area of significant erosion (Figure 4.23a). The deepest erosion occurs in the left-hand corner where transport occurred first and last. In the final time interval (VII of Figure 4.22), a single particle came to rest in this area, indicating that it was an area of deposition at the end of the flood. The cartoon plot showing the image color segmentation confirms that many coarse particles remained in this area where transport was most common and net erosion occurred. These particles become more concentrated at the end of the ROI and continuing onto

the front face of the downstream bar where net deposition was observed. Above the area of net erosion, a band of net deposition can be observed (Figure 4.23a), with the area closest to the left bank showing no erosion or deposition. The left bank area with no disturbance corresponds with the area where few tracers were observed to move (Figure 4.22), but has many coarse particles at the end of the run (Figure 4.23b), which are therefore likely residual from their initial placement on the bed. The area of net deposition in the centre of the channel, is different because no coarse particles were deposited in this area, indicating that it is relatively fine sediment depositing beside the main channel thalweg.

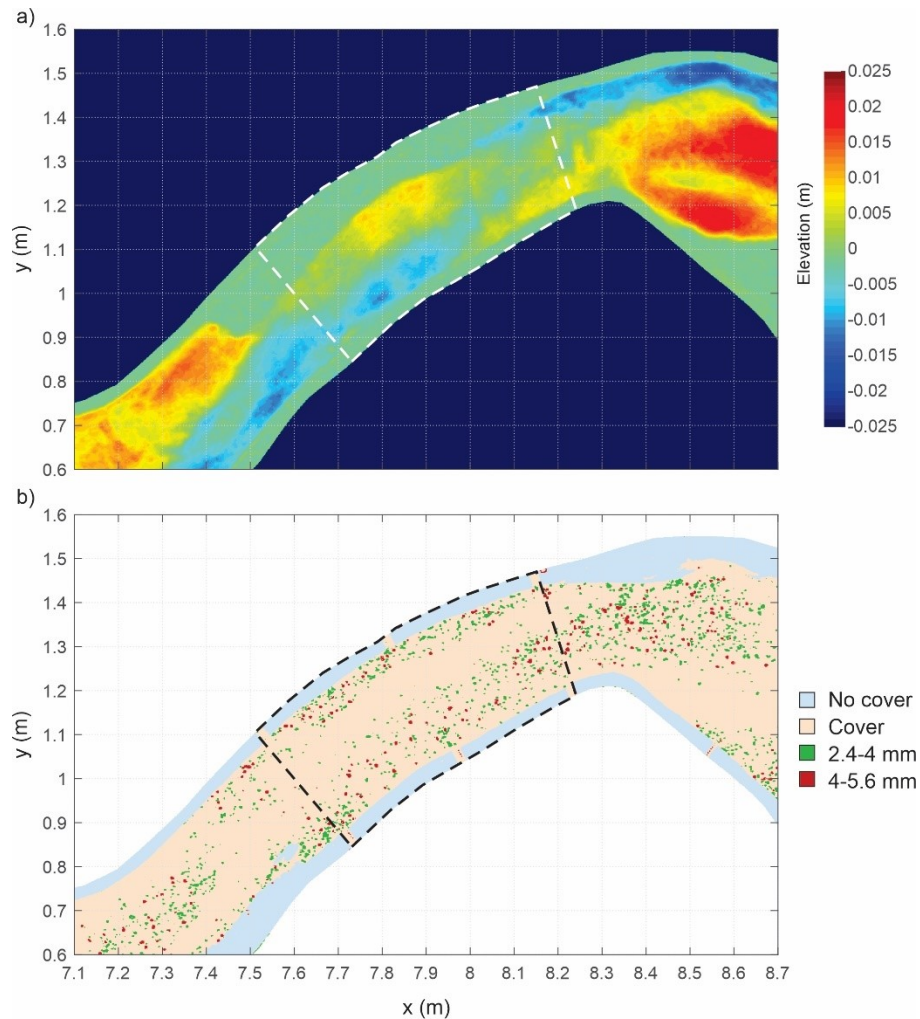


Figure 4.23 Final bed topography crossover section (bend FG) for run PTH1_1 (HR = 0.50) including a) DOD and b) cover plot. Flow is from left to right.

4.4 Discussion

The objectives of this research were to develop an image-based technique to trace the movement of painted particles during an experiment and assess the impact of unsteady flow on sediment transport and bar formation in the irregularly meandering channel. For the first objective the method successfully identified areas of active sediment transport within the flume and demonstrated that such an approach is viable for experiments in shallow flumes. However, the available algorithm for tracing sediment paths was not particularly developed for sediment in motion and may require some modification to be more reliable. For the second objective, the results show that the shape of the hydrograph is of secondary importance to the overall strength of the flood for sediment export, and that within flood spatial variability of bar dynamics may vary between repeated experiments. Such variability appears to have a significant impact on mass outputs. In this discussion we assess the significance of trends in the results related to the shape of the hydrograph and examine some of the dynamics in greater detail so as to guide future experiments. We also comment on limitations in the methods and provide recommendations for future analysis and experimentation.

The mass export results show that there is no significant difference in the sediment export mass or size distribution between the runs with different hydrographs. There are some outlier results, for example the total mass exported during run PTH1_2 (HR = 0.50) and size distribution during run PTH 3_2 (HR = 0.35). The lack of trends in the max exported is also reflected in the lack of difference in the final topography of the channel. The hypsometric plot (Figure 4.19), for example, shows that the final topographic distribution of elevations relative to the bare bed is remarkably consistent between the runs. These two results are consistent in that the mass exported should be reflected in the total volume of sediment exported – so it is not surprising that the median sediment depth is similar between the experiments, but it is remarkable that even the tails of the distributions match for the different experiments so that the peaks of erosion and deposition are also consistent between the runs.

Outlier results in mass export may be the result of inherent variability in the experiments or methodological errors. For example, it is possible though not probable that some mass was missed when the output sediment was weighed for run PTH1_2. It is also possible though not probable that the wrong sediment size distribution was used when the sediment was mixed for run PTH3_2. The idea that differences in rising and falling limb slopes are not significant for modelling the total export is consistent with the model developed by Papangelakis et al. (2022) to predict sediment displacement as

a function of the channel hydrograph. The general picture that emerges is thus that the cumulative discharge or stream power above a transport threshold can be used to predict the sediment flux. The degree of hysteresis appears to be not significant for the overall mass export based on the current study. Given the limited number of runs in the current study, additional experiments are recommended to test this result and compare with other studies.

Despite the lack of a general trend in mass export as a function of channel hysteresis, it is clear that different size fractions respond to the floods differently. As was found by Peirce et al. (2021), the larger size particles tended to be over-represented in the exported sediment. In most experiments it can be seen that P_p stays > 1 for larger particles and < 1 for smaller particles, which means that the sediment remaining in the channel becomes relatively finer during the experiment. For the case with the highest rising slope, an even higher proportion of coarse sediment was exported, which indicates that the random nature of the initial bed may result in more transport of the coarse material. This is unlikely to be a micro-scale result because the particles were well mixed and distributed, but is thought to be related to the macro-form organization. When time to rise is longer, it is possible that particles are able to find locations from which they are unlikely to be transported in a way that does not happen in a rapid rising limb. More work is necessary to better understand the fate of the different particle sizes for different rising limb slopes.

Another interesting result is that the P_p value for the median size particles (d_{50}) is close to 1 for the entire hydrograph duration no matter the shape of the hydrograph. The proportion of median size particles leaving the flume thus tends to match the sediment remaining in the flume even as the mass of exported sediment rises and falls with the flood. The channel is not in equilibrium because the initial bed is an even layer of mixed sediment, so it would be interesting to compare the current results with those in which a bed develops due to a distribution of response to floods with a bed that was water worked. It would be useful to know if a bed in equilibrium would show a similar effect of flow hysteresis on sediment export or not as most natural channels will not have a randomly distributed sediment but will have important spatial variability in the sediment size distribution that could help it maintain a more balanced response to a flood. It was also observed that rapid rising limbs result in a lag between the peak flow and the peak sediment flux such that the peak value for PQ_s occurs after peak for Q in for most sediment sizes. Therefore, for the hydrographs with ($HR < 0.5$), the highest mass of particles leave the channel during falling limb.

Despite the overall consistency of the hypsometry, significant differences were observed in the shape of individual bars, where two examples were shown. The fast-rising limb seemed to result in a great deal of deposition close to the channel thalweg. Previous work (Peirce et al., 2021) showed that in a bare bed, the initial deposits occurred close to the channel thalweg rather than close to the bank and that the near bank bars developed later in the simulation once the skeleton of the bars had formed, largely made of coarser particles. The current results show that a similar process occurs despite the presence of the uniform initial sediment depth. In a rapid rising limb, the coarse bar forms a prominent central feature. The near inner bank bar likely forms in the long falling limb, which may explain why the bar is larger for the experiments with the rapid rising limb. Output suggesting the peaks of the floods are dominated by coarse material transport, which indicates that these central particles are mobile in the center of the flood. If we compare with the model developed by Pyrcie and Ashmore (2005), the lower elevation bars likely represent material deposited in the falling limb after the high bars have been developed. The fine material tends to collect on the inner bank relatively close to the thalweg. Coarse material still appears to be transported at this stage, but now along the cover/bare bed line as was observed by Peirce et al., 2021. These deposits are likely to be material deposited near to the thalweg in higher flows that is reworked at lower stages. Accumulations of coarser sediment that cover the thalweg appear in both the run with the long and short rising limbs (Figure 4.21), which suggests that their appearance is somewhat random rather than being a feature of a particular hydrograph shape. More bars from the different runs will likely help to discern whether there are clear trends with bar formation.

The method used for tracking single particles in the channel was able to capture and show particle paths. However, based on a visual assessment of the recordings, it could be seen that many particle paths that visually appeared to be a single particle movement were subdivided into multiple segments. The particles never disappear along their path in the ROI. The problem occurs when particles change their shape on their way downstream, which can happen due to the rotation of the particles and because of water surface waves, which can change the refraction of light in the flow depth and cause particles to shrink or even disappear in terms of the number of pixels in one or more frames. Overlapping particles can also occur such that a mobile particle may appear to merge with one or more underlying particles. The next step will be to confirm the sources of errors that lead to the breaking of particle paths into multiple segment. A program to manually identify particle paths will be developed to try to

‘ground-truth’ the results. Another step will be to check if the particle path terminus and initial point are so close together in time and space that they can be practically considered as a single path.

4.5 Conclusions

The following conclusions can be drawn from the results and discussion of this chapter:

1. A novel image-based method for tracking sediment during floods in laboratory flume experiments was developed. The novelty of the approach is that painted tracers are used with UV lights to create a set of high contrast videos that allow different size classes to be tracked simultaneously. This approach will allow the better characterization of sediment dynamics and bar formation in mixed-size sand and gravel transport.
2. More work is needed to verify the results from the tracking method and assess the need for post-processing the results from an available tracking algorithm developed for modelling the motion of tracers transported by water. The mechanics of transport of bed load material means that velocities and directions of transport can change over short distances, meaning that it may be necessary to develop a tracking approach that is specifically adapted to this problem.
3. The overall quantity of mass export suggests that the shape of the hydrograph is of secondary importance to the overall strength of the flood. In particular, the high variability of sediment export on the rising limb of floods indicate that the random variability of bedload transport through an irregularly meandering channel may result in different trajectories of sediment export and bed evolution. Further experiments are warranted to assess the repeatability of results between seemingly identical experiments.

Chapter 5

Discussion

This thesis was done to explore the hydraulics of floods in urban and rural channels in addition to their effects on sediment transport. In this section we summarize the major findings from the research, discuss real-world implications of this research and list limitations of the research, which leads to possible future studies that can be done further to the presented work.

5.1 Major findings

The overall findings from this research highlight the unusual aspects of flood hydrographs in urban areas and contribute to unravelling the complex sediment transport response to these events. The most significant conclusions from the work include the idea that floods in urban areas without stormwater management are characterized by a sawtooth pattern of rapid rises and falls within a single event due to the fast response time in the watershed. Finding a storm with a single flow peak is relatively unusual, and high-frequency measurements were used to measure these rates in new detail. The observed differences in shear stress may help to explain sediment transport results observed by others. Sediment transport was investigated in a physical model that reproduced the high slope of the rising limb from the field study. A significant advance from this work was the detail at which sediment transport could be observed using a novel tracking system with UV lights. The results did not indicate that the rising limb slope was a critical determinant of sediment export from the channel, but some interesting dynamics in bar morphology and spatial variability in the active areas of transport point to a need for more research before definitive conclusions can be made. Findings from individual chapters are detailed in the following paragraphs.

The major contribution from chapters Chapter 2 and Chapter 3 is the demonstration that steady-state assumptions are not valid for the studied urban watersheds and even for the rural watershed during extreme flood events. The high degree of unsteadiness was quantified by the unsteady terms in the Saint Venant equations, which has rarely been done for small urban watersheds with similar properties to those studied here. The failure of a steady-state assumption is less surprising in the case of the urban watershed without any moderation of the hydrograph with stormwater management (SWM), as other studies have used different methods to show that flow unsteadiness may be important in steep watersheds with rapid runoff, but the significance of flow unsteadiness was more surprising for rural watersheds and those with major stormwater management infrastructure. Stormwater management does

not completely solve the problem of the flashy hydrograph, despite the fact that the hydrologic moderation of the flood peak is achieved. Rural systems may also have a relatively high degree of unsteadiness at peak floods, and this needs to be better understood for flood and sediment transport modelling. There are few studies of flow hydraulics of urban systems with SWM, and the variety of types and scales of SWM practices mean that we should expect a high variation. One important point is that the hydrologic results do counter the results from the study of Bell et al. (2016), who indicated that the impact of SWM measures was hard to detect at a watershed level. For the Morningside system in the current study, the type and degree of change that results from the SWM measures is more than enough to reduce the peak floods and shear stress to at least the level of the rural control site.

Chapter 3 further examines floods in the urban watershed with a unique field sampling strategy that used multiple gauges and high sampling rate in an effort to fully capture the water surface slope and temporal depth changes in the river. The results confirm that the unsteady flow signatures affect discharge, shear stress, and stream power estimates. The error in flood rating curves was shown to be largely eliminated through the use of multiple gauges recording at a sufficient frequency to accurately characterize the changes in water surface slope that result from the rapid rising and falling limbs. Multiple gauges allow these spatial gradients in depth and velocity to be included in rating curves for at a station measurement of flow hydraulics.

Chapter 4 was designed to test the significance of the rate of the rising limb on sediment dynamics in an irregularly meandering channel. For simplicity the bed was assumed to be a uniform bed of well mixed sediment. This allows a comparison of sediment export and morphologic changes. Despite the simplicity of the experiment, however, the complexity of the transport of mixed grain sizes in a channel confounded the simple interpretation of results. It was found, for example, that the variation of the export mass between three experiments with the same hydrograph was greater than the variation between experiments with different hydrographs. This surprising result indicated that the microscale adjustments of the bed were significant for controlling export from the channel. Further research was initiated to compare the sediment export by size fraction and examine the morphologic change. This work is largely exploratory in nature but revealed some interesting hysteresis, with coarse sediment responding positively to peaks in the hydrograph and finer sediments tended to be less represented in the output mass during flood peaks. Sediment tracking was initiated and novel methodological advances made to better understand the sediment dynamics that lead to the high variation between floods with the same initial conditions and flood hydrograph.

5.2 Implications

The first major implication of this thesis is that the two-gauge method should be advanced as a method to calculate discharge without the need of velocity measurements. The benefits of using this method is that it eliminates the cost and need for having velocity measurements either for low flows or high flows. As stated in the introduction, the most popular method for developing rating curve is that velocity and depths are measured for different conditions and a rating curve is plotted for those measured depths. However, since measuring velocity at higher flows are not possible, rating curves are typically extrapolated for higher flows, which might not be accurate. Despite the traditional method of developing rating curves the two-gauge method we used in this research provides the opportunity to calculate discharge not only for low flows, but also for high flows. For this method we only need two pressure gauges installed at two adjacent surveyed riffle sections.

The other major implication of this thesis is that the method presented in Chapter 4, which is a novel method presented by Montakhab et al., 2022 , can be used to track sediment particles using videotaping, a PTV method, and painted particles. This tracking method can be used to better understand the complexity of bedload transport in flumes under steady or unsteady flows. In particular, the technique should be used to better understand the impact of the organization of sedimentary bedforms on the downstream export of sediment. The results of Chapter 4 can help researchers to better understand the effects of hydrograph shapes on the movement of sediments during unsteady flows and in natural shaped channels. Sediment transport under unsteady flows are rarely examined in the literature and the few that exist use relatively straight channels. Therefore, our research is a unique opportunity to observe the interaction between bedload transport, bar morphology, and flood hydrographs.

5.3 Limitations

This thesis involved field work and laboratory experiments, most of the limitations are related to the measurement methods. The first limitation that I faced was that the pressure gauges cannot give us reliable measurements during winter. In winter due to ice the pressure gauges might not work accurately. The other challenge that we faced is that we used several methods to find the most appropriate Manning's n . However, finding the right Manning's n is always challenging. It could be beneficial if there was a more robust way to find the appropriate Manning's n .

The next limitation is related to the velocity measurements during high flows. For velocity measurements we used two methods of PIV and a radar gun. In the PIV methods it is very important

that the coordinates and the distance measurement in the camera footage accurate in order to deduce the velocity of the water surface and might be inaccurately set. The software that we used could only find velocity for where there is high turbulence on the surface of water. The reason is that the turbulence was obvious and detectable on the top of surface and the software could track the bubbles appearing on the water surface. For the times that there was less turbulence, the PIV method would not work well and it was not reliable. For the other method we used, the challenge we had was that when there was rainfall, the radar gun would capture the velocity of rain particles instead of water surface velocity. Therefore, the velocity measurements were not reliable during rainfall. Therefore, although these two methods were able to measure velocity during high flows, we faced several challenges during our measurements.

For the results of the last chapter, we faced many challenges. The first challenge was that when two sediments with the same color would pass on top of each other, it was difficult to detect which one was moving. The other challenge was when a painted particle would be buried under the unpainted particles. These particles would not be obvious in the camera recordings after they're buried. Therefore, it is difficult to indicate the time when they stop and start moving. The next challenge is that the pump that was installed to provide the flows was not synced with our flow measurements. The flow meter that was installed in the pipe upstream of the channel, sometimes would not exactly match with the flow that was supposed to come out of the pump. It would be very beneficial to find a pump that can be controlled more efficiently.

5.4 Future Work

This research can be continued in many ways since it investigates both hydraulics of flows in urban and rural catchments and it also looks at sediment transport under flashy flows. Future work should investigate the effects of flow unsteadiness on other rural and urban watersheds with different characteristics to be able to have a better understanding of unsteady flows in both types of watersheds. Having longer period of data can also support our observations and the results of ChaptersChapter 2 and Chapter 3.

For the results of Chapter 4 future research should attempt to replicate the results from the current chapter and investigate more hydrographs with different shapes other than simple triangles. Hydrographs observed in Wilket Creek can be scaled into the flume to see how sediments move under more realistic hydrographs. In addition, a similar study should be initiated to assess the results under

the same flow hydrographs but with a water-worked bed that is built up with a steady discharge or with a series of hydrographs. The methods that were developed for the current study should help with understanding the effects of the pre-prepared channel bed, and investigate the effects of sinuosity in channels with different planform shapes.

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Appendix A

Studied Hydrographs

A.1. Hydrograph October 27th 2019

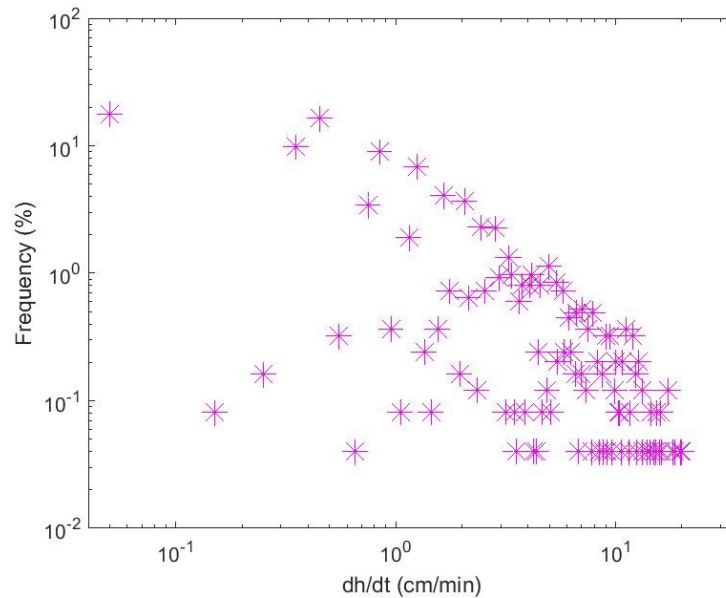


Figure 5.1 changes of flow depth over time for the hydrograph

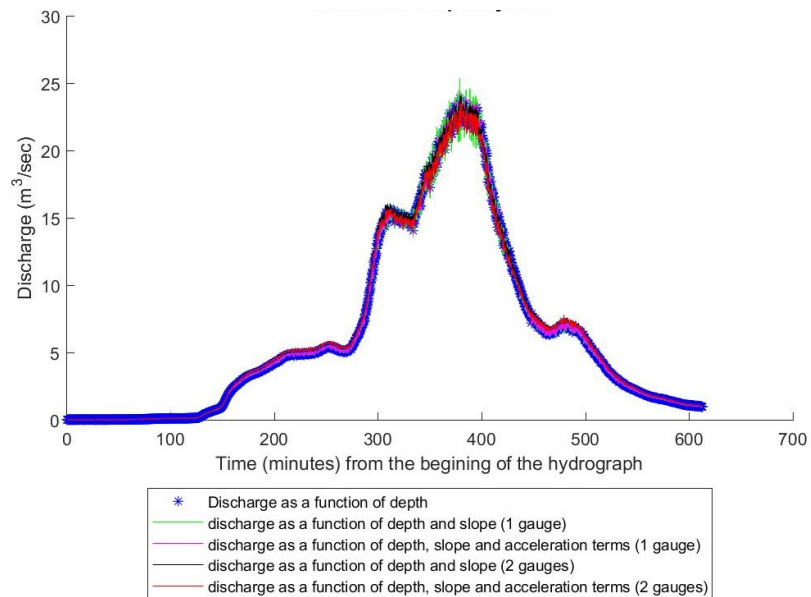


Figure 5.2 Flow Hydrograph for the flow occurred on October 27th 2019

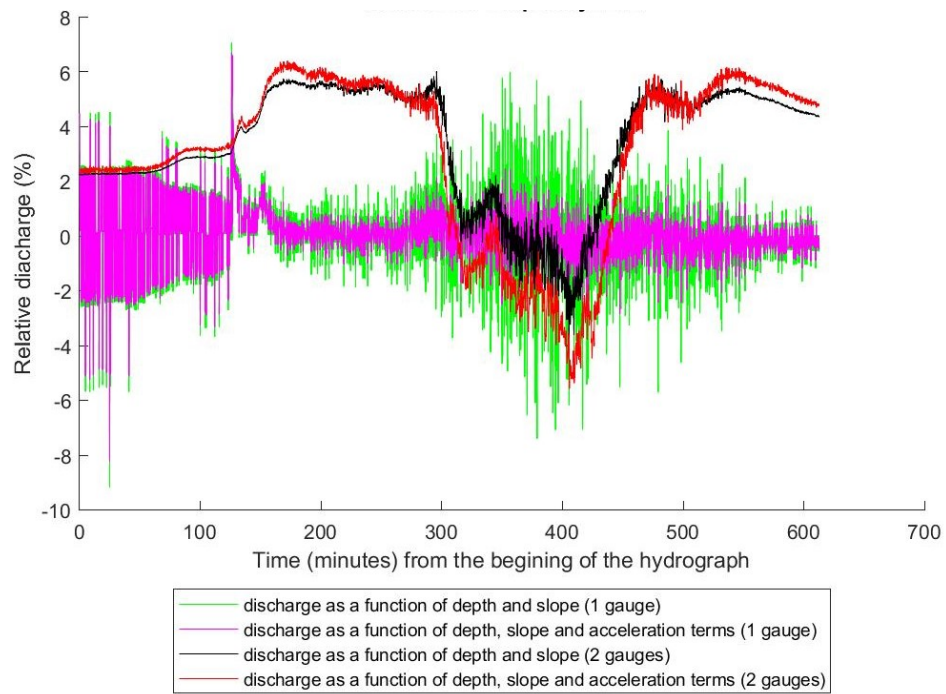


Figure 5.3 The relative discharge graph for the hydrograph observed on October 27th 2019

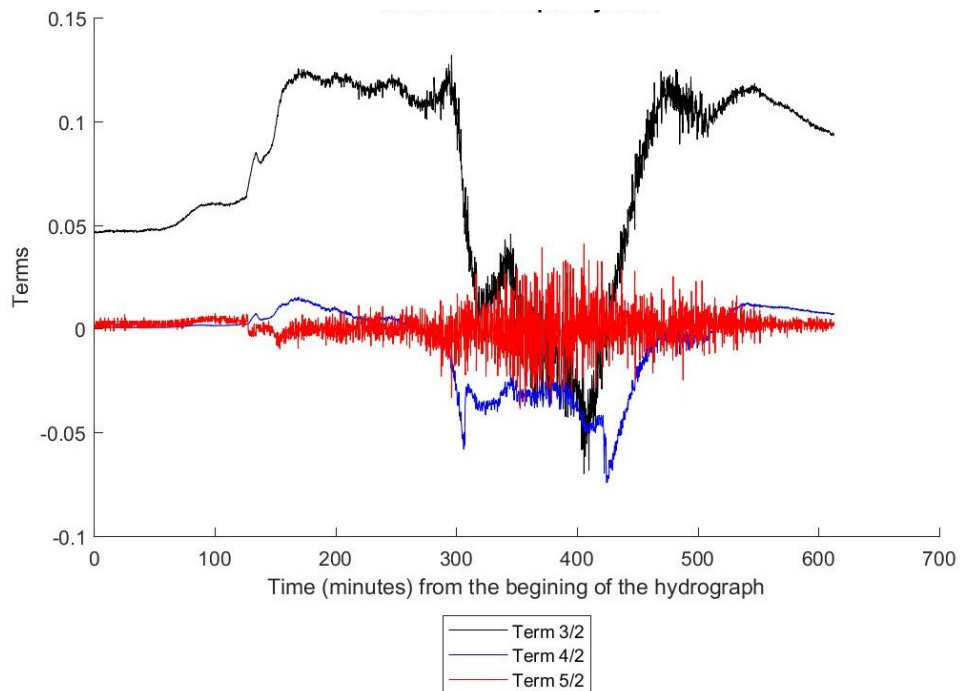


Figure 5.4 The three terms of Saint-Venant equation over time for the hydrograph happening on October 27th 2019

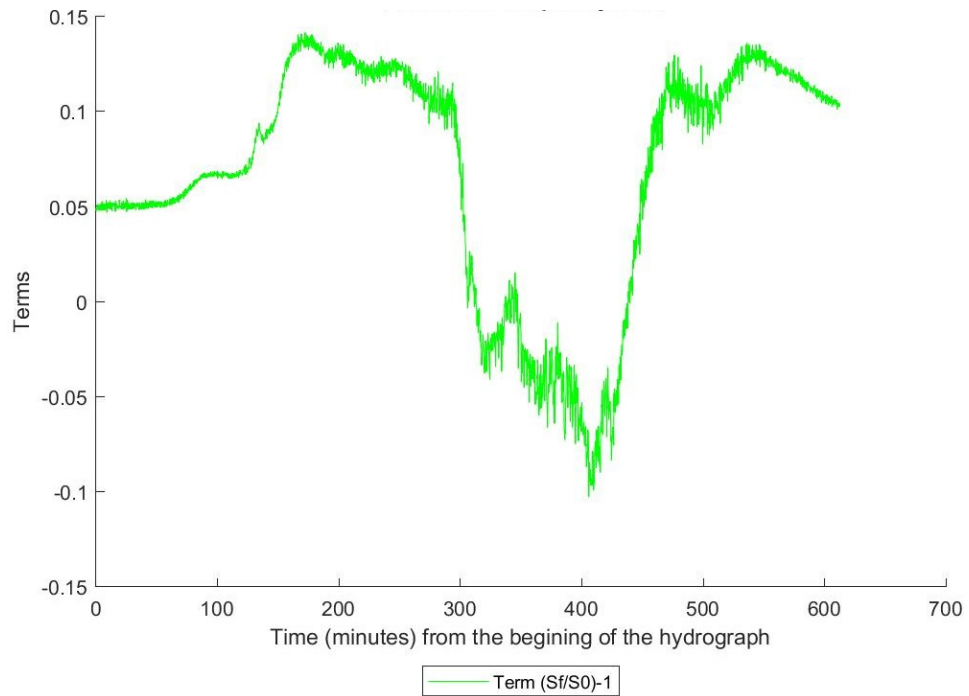


Figure 5.5 The sum of all the three terms of Saint-Venant equation ($Sf/S0-1$) for the hydrograph occurred on October 27th 2019.

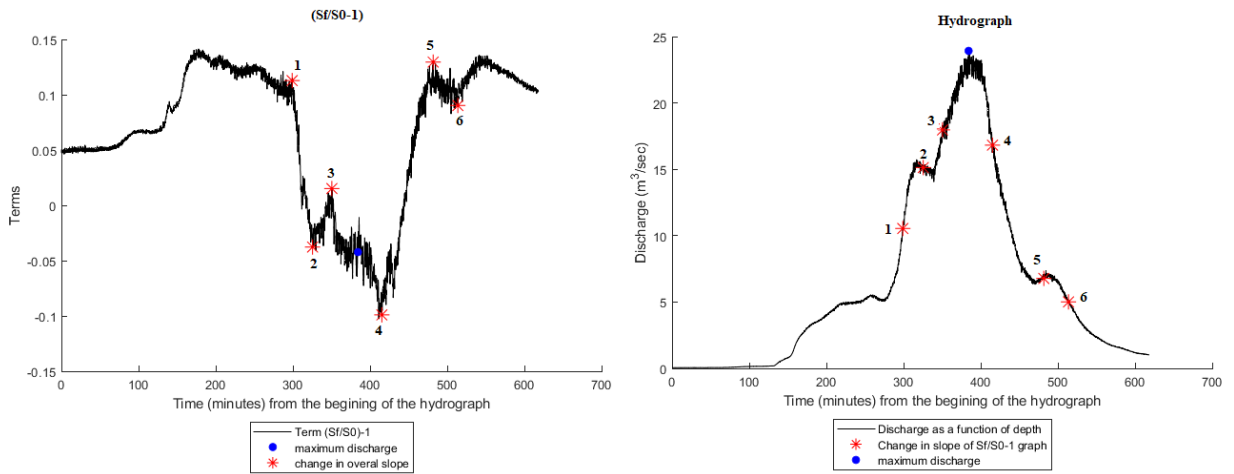


Figure 5.6 The comparison of $Sf/S0-1$ with the hydrograph occurred on October 27th 2019

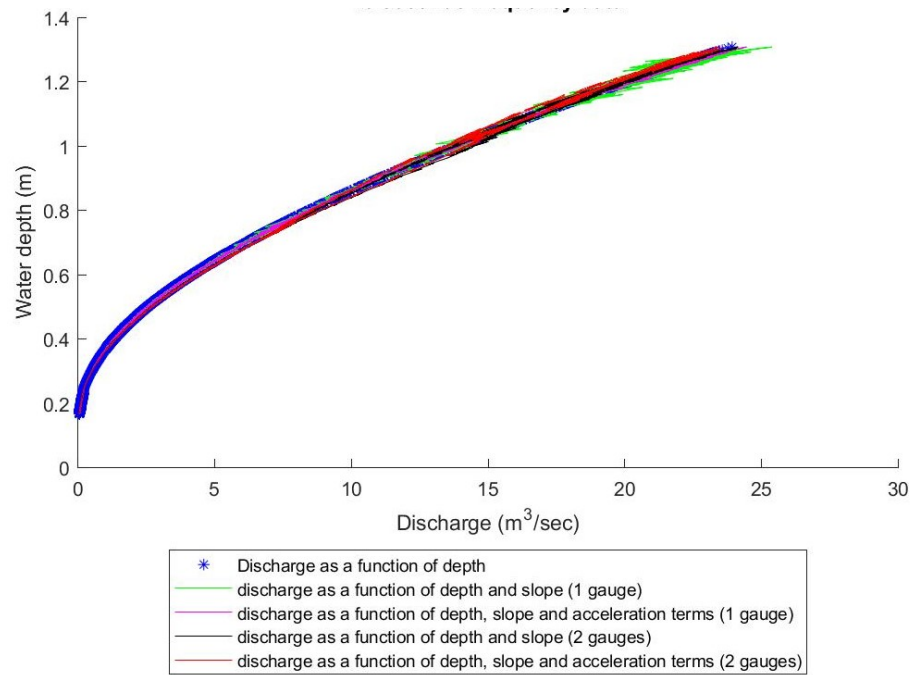


Figure 5.7 Rating curve for the hydrograph happened on October 27th 2019

A.2. Hydrograph July 17th 2019

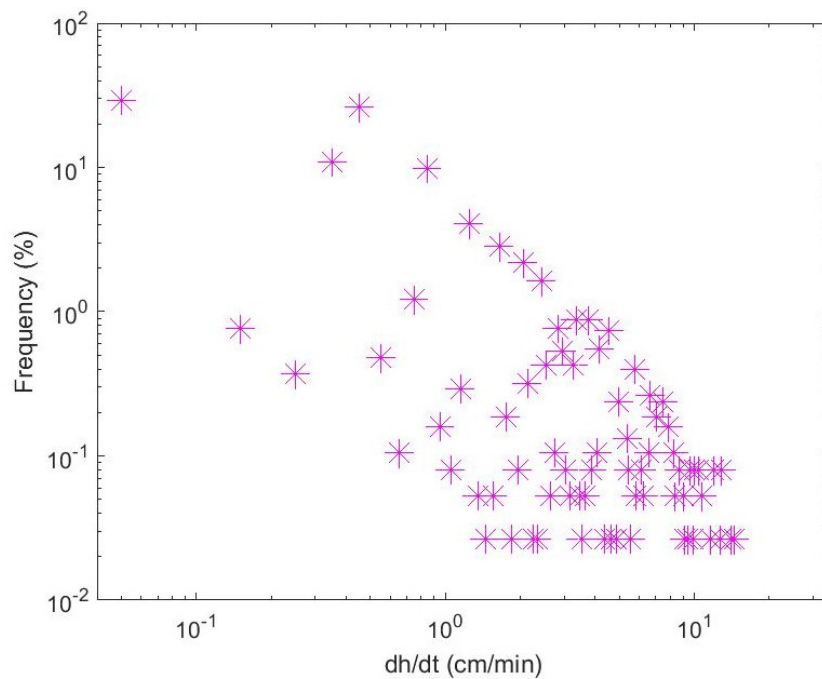


Figure 5.8 changes of flow depth over time for the hydrograph observed on July 17th 2019

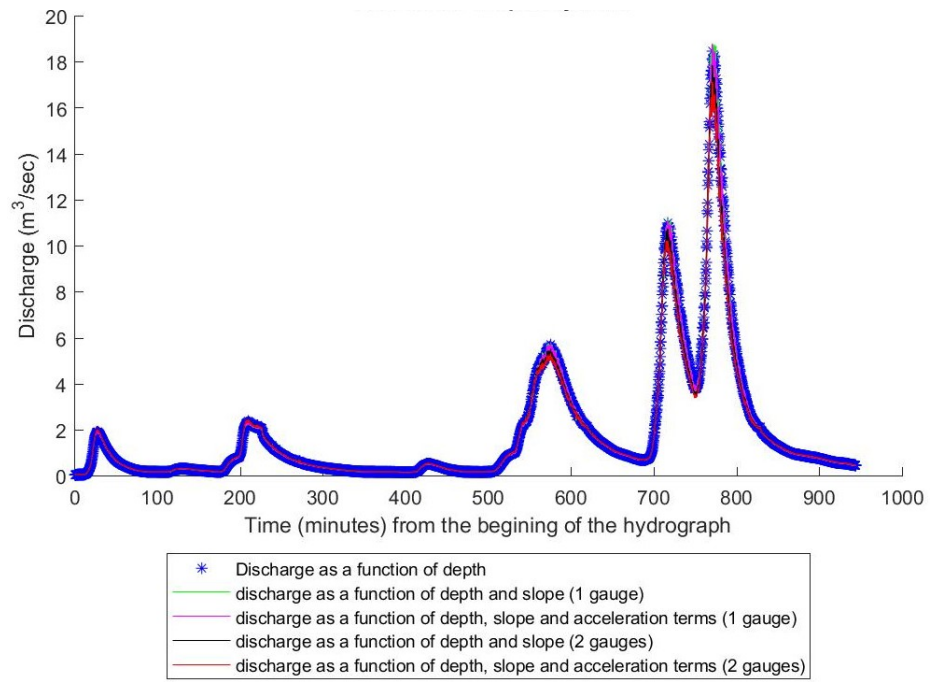


Figure 5.9 Hydrograph for the flow occurred on July 17th 2019

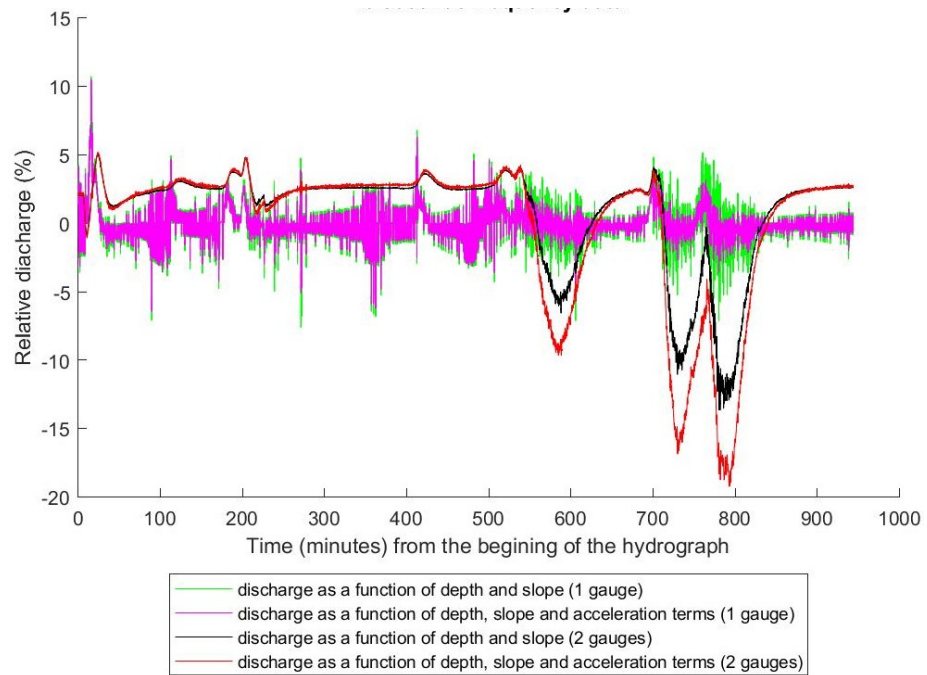


Figure 5.10 The relative discharge graph for the hydrograph observed on July 17th 2019

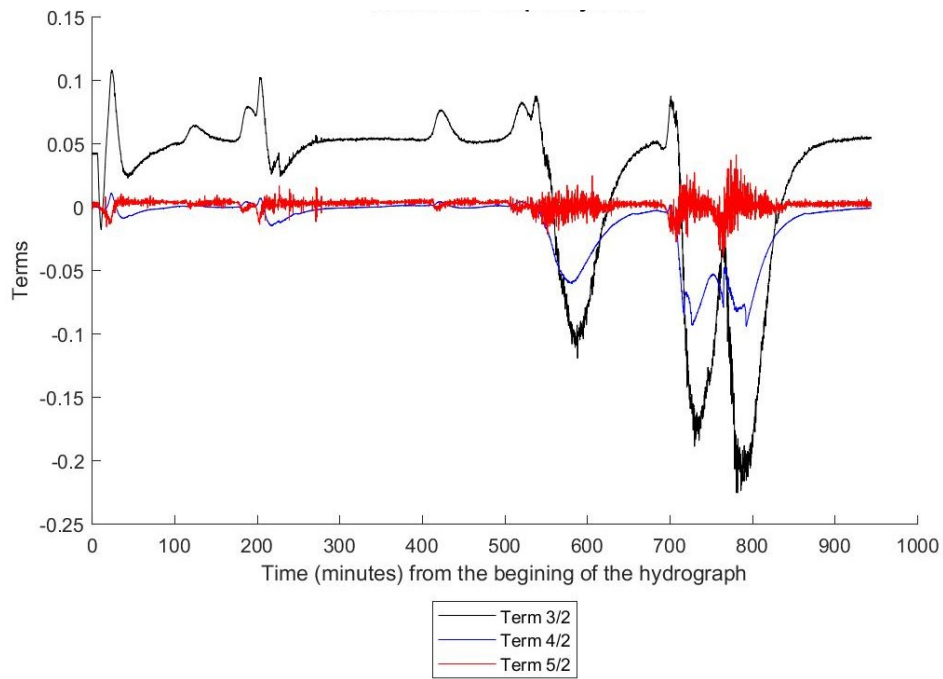


Figure 5.11 The three terms of Saint-Venant equation over time for the hydrograph happening on July 17th 2019

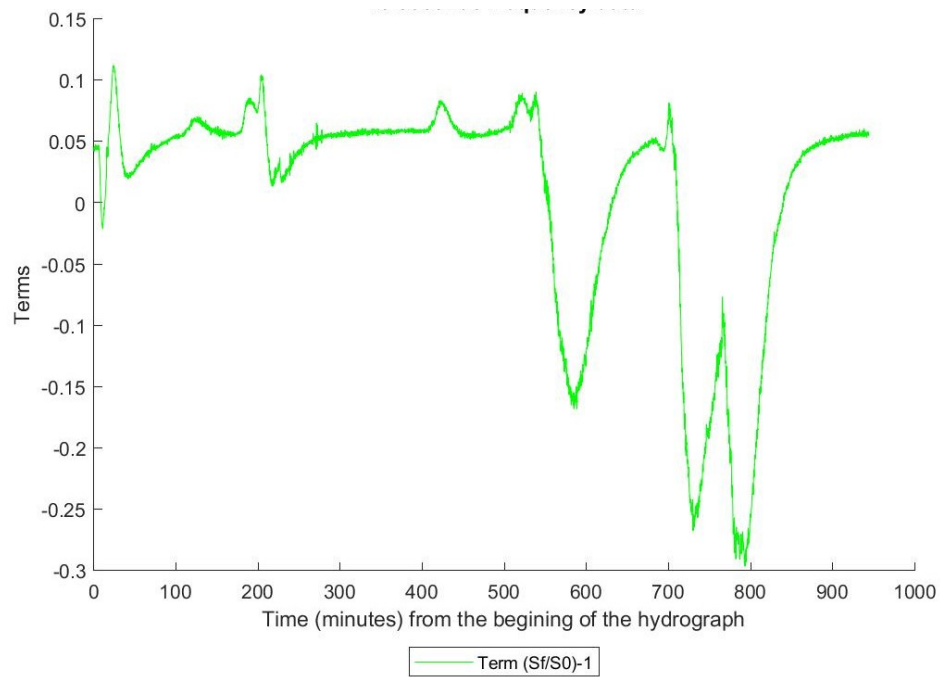


Figure 5.12 The sum of all the three terms of Saint-Venant equation (Sf/S_0-1) for the hydrograph occurred on July 17th 2019

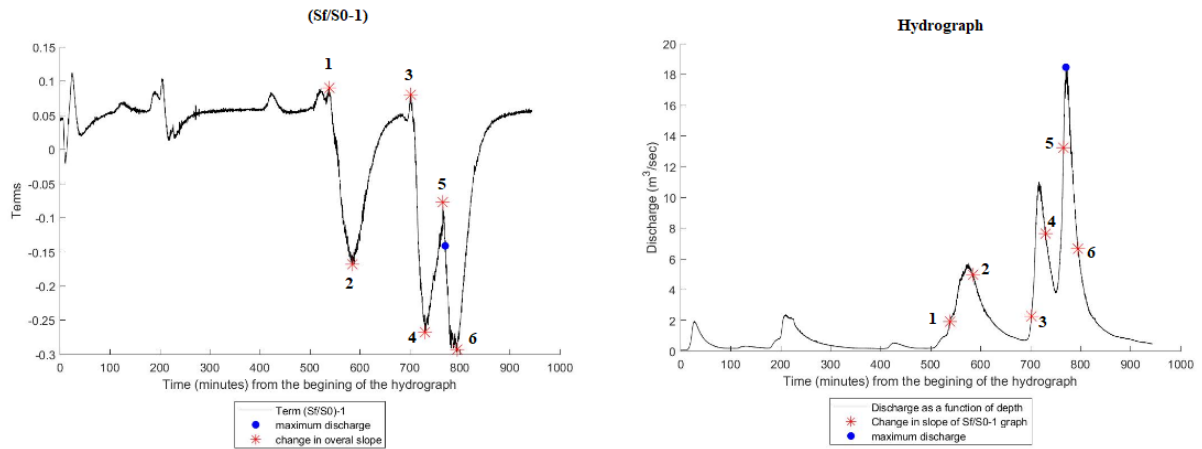


Figure 5.13 The comparison of Sf/S0-1 with the hydrograph occurred on July 17th 2019

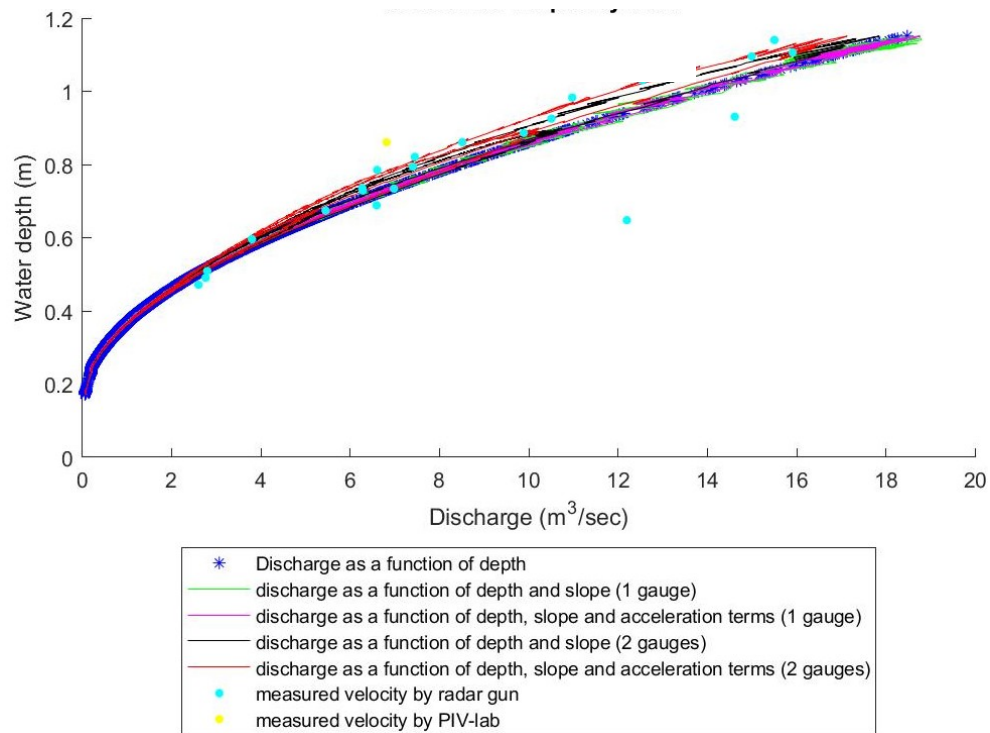


Figure 5.14 Rating curve for the hydrograph happened on July 17th 2019

A.3. Hydrograph October 31st 2019

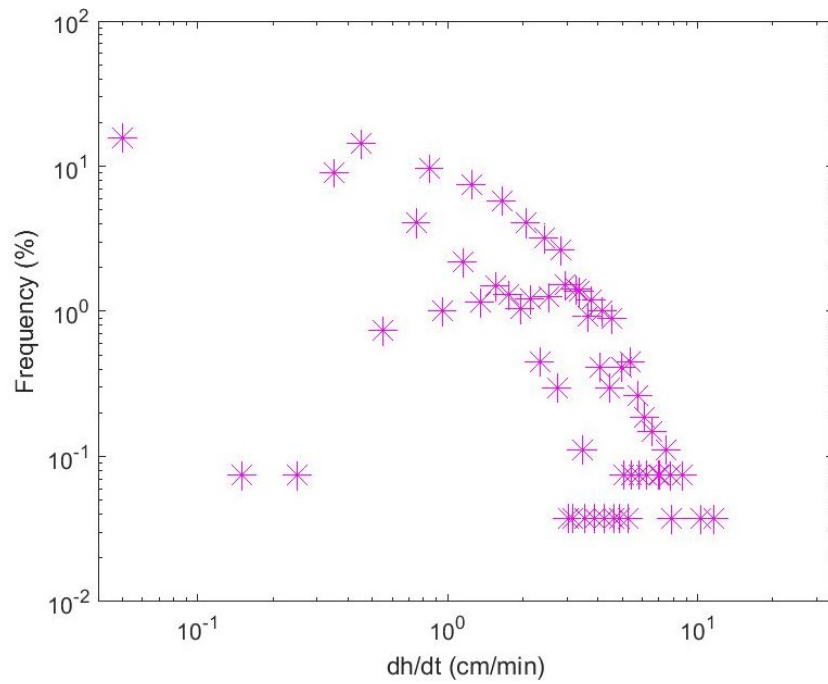


Figure 5.15 changes of flow depth over time for the hydrograph observed on October 31st 2019

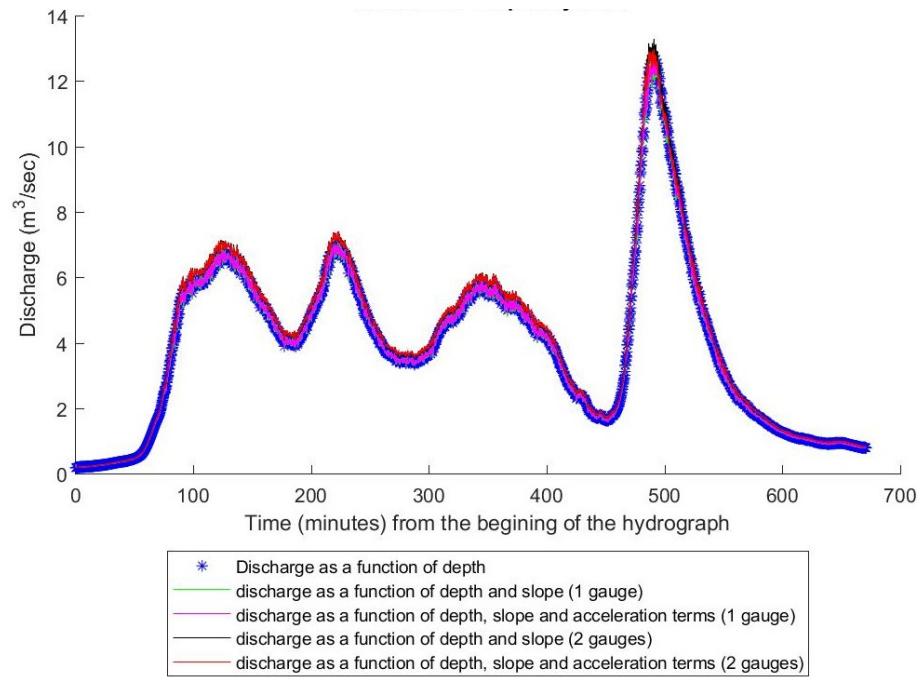


Figure 5.16 Hydrograph for the flow occurred on October 31st 2019

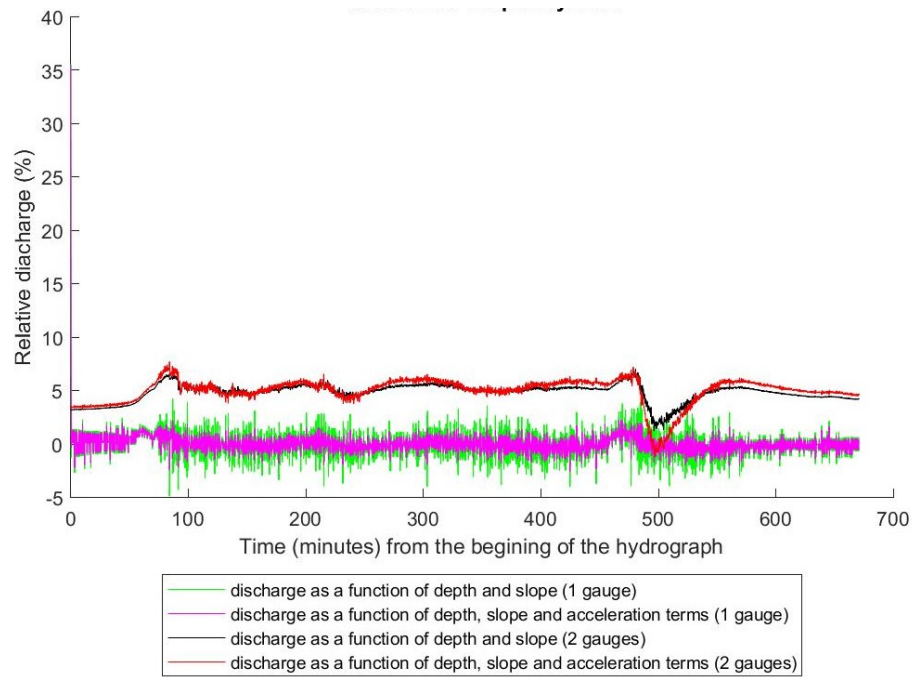


Figure 5.17 The relative discharge graph for the hydrograph observed on October 31th 2019

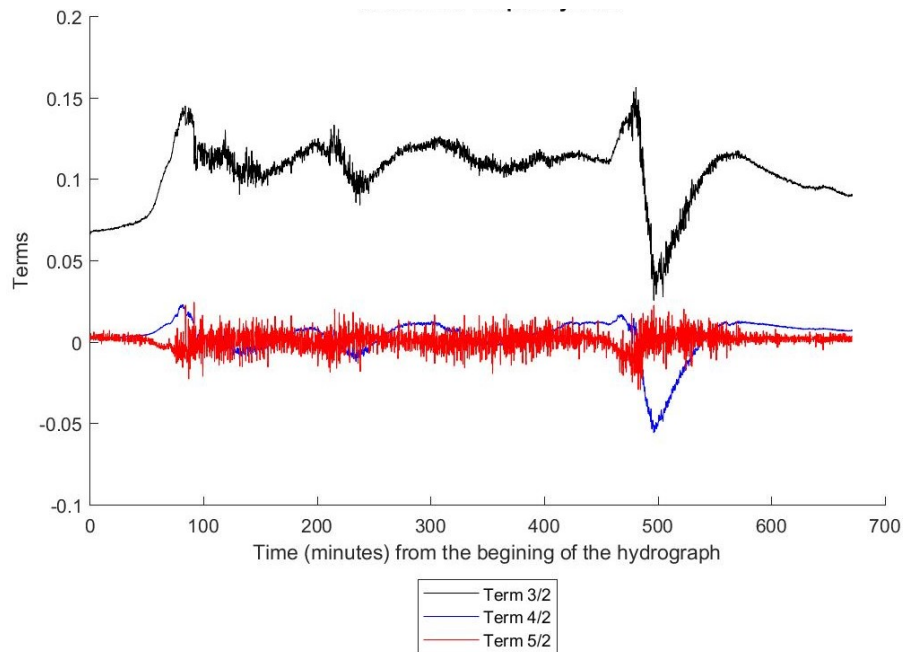


Figure 5.18 The three terms of Saint-Venant equation over time for the hydrograph happening on October 31th 2019

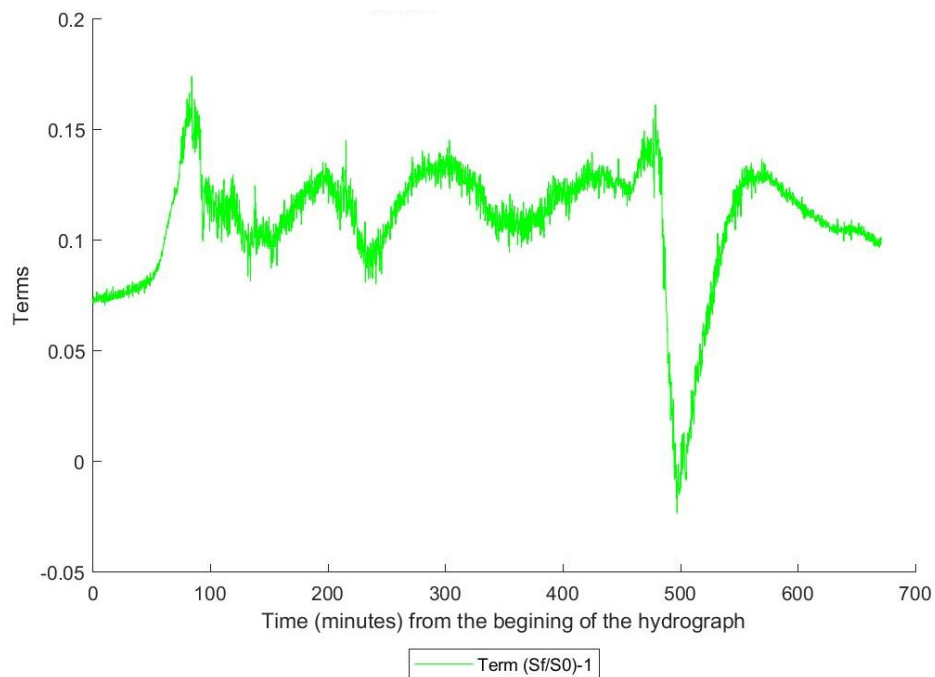


Figure 5.19 The sum of all the three terms of Saint-Venant equation (Sf/S_0-1) for the hydrograph occurred on October 31th 2019.

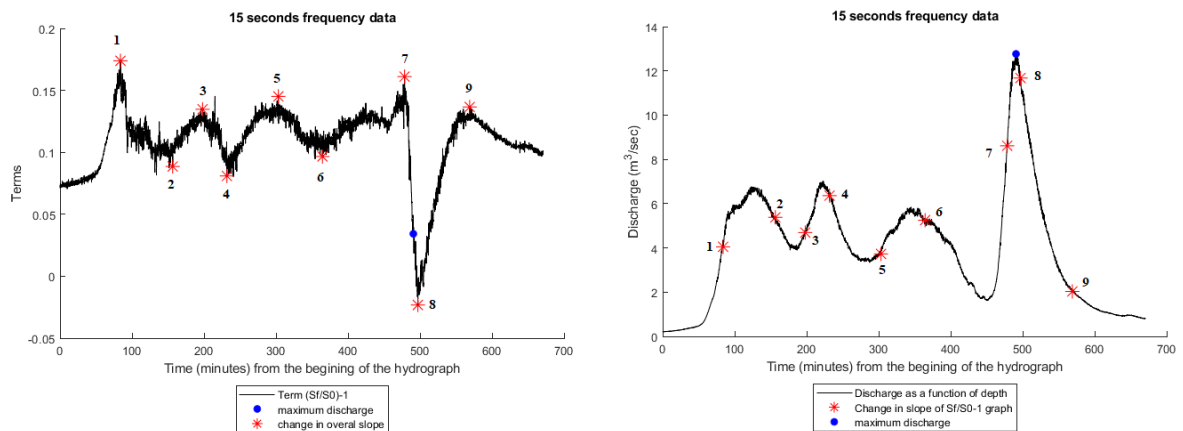


Figure 5.20 The comparison of Sf/S_0-1 with the hydrograph occurred on October 31th 2019

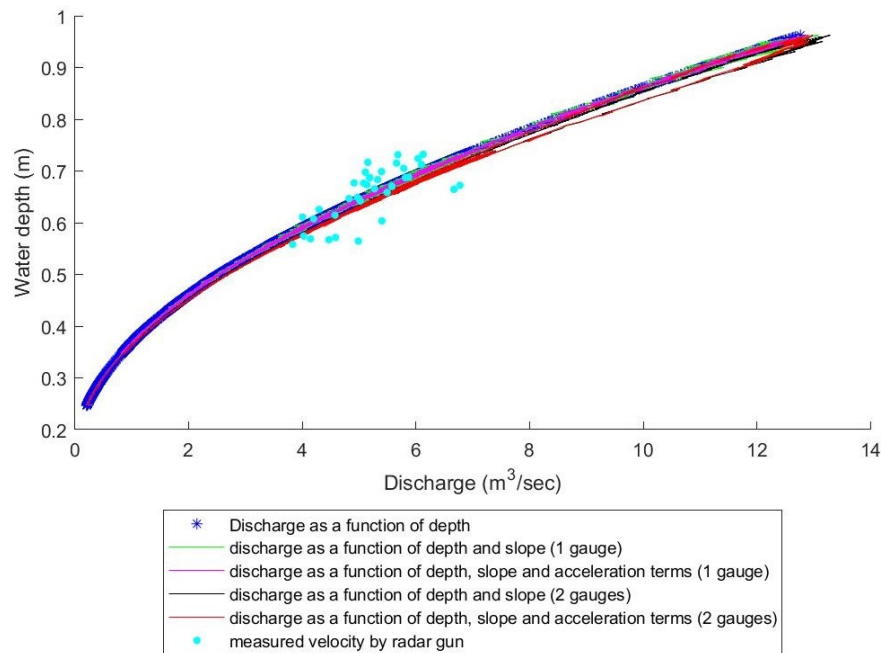


Figure 5.21 Rating curve for the hydrograph happened on October 31th 2019

A.4. Hydrograph May 9th 2019

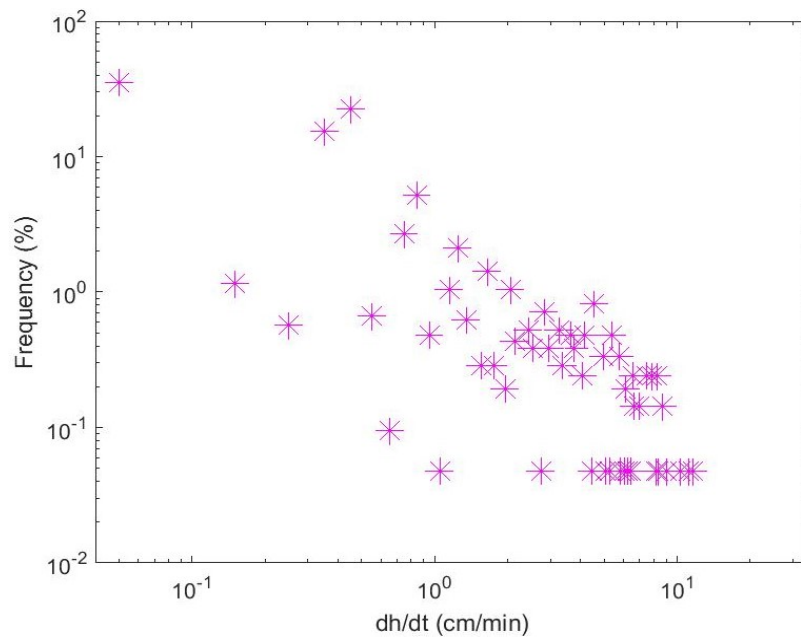


Figure 5.22 changes of flow depth over time for the hydrograph observed on May 9th 2019

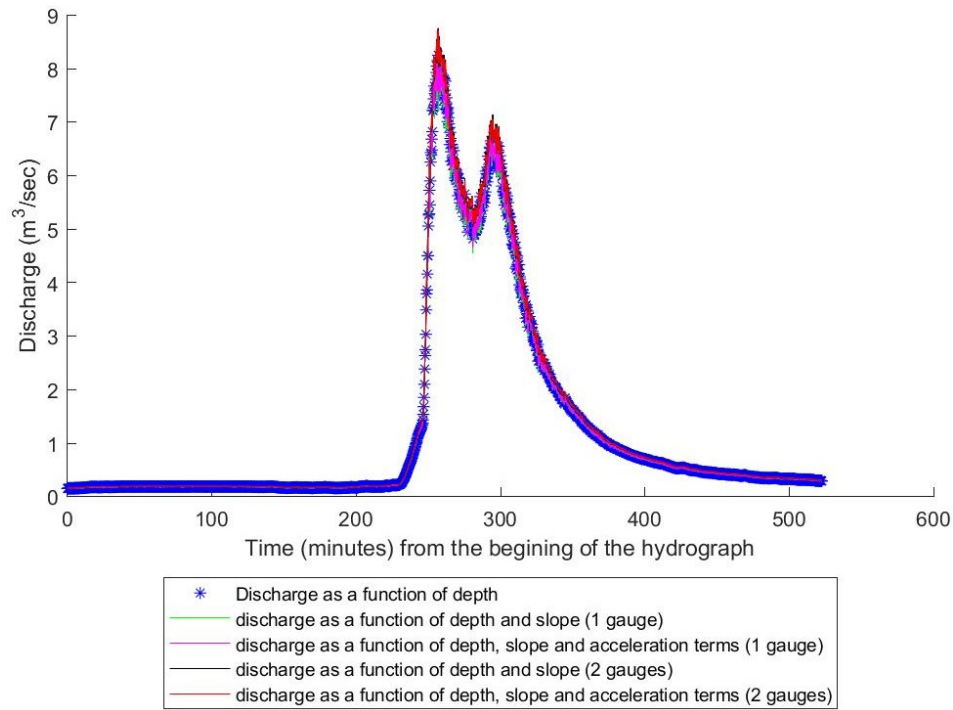


Figure 5.23 Hydrograph for the flow occurred on May 9th 2019

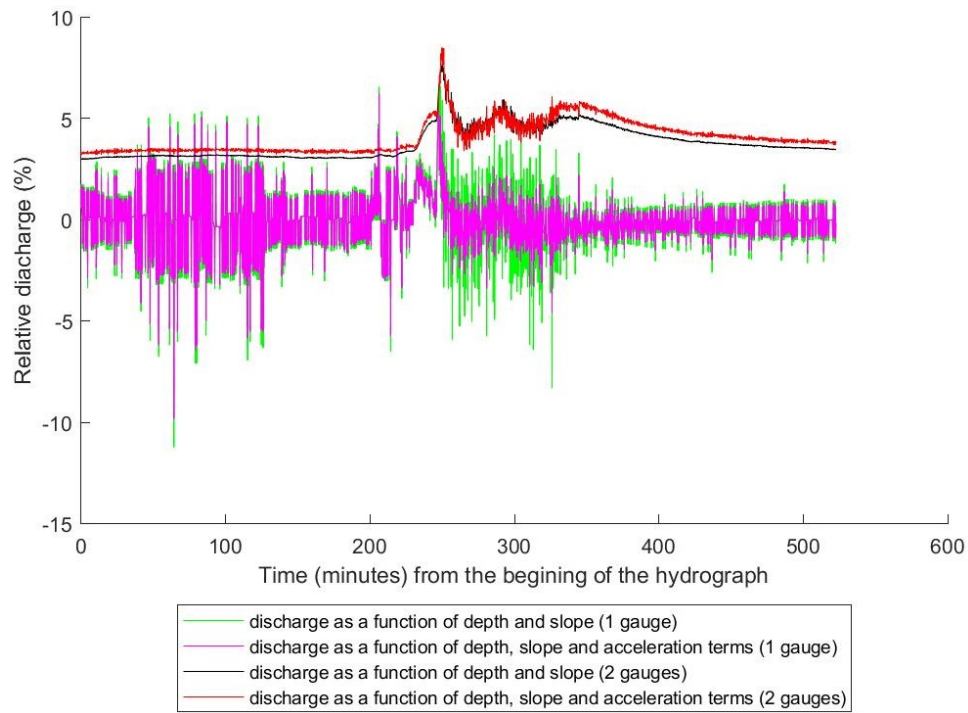


Figure 5.24 The relative discharge graph for the hydrograph observed on May 9th 2019

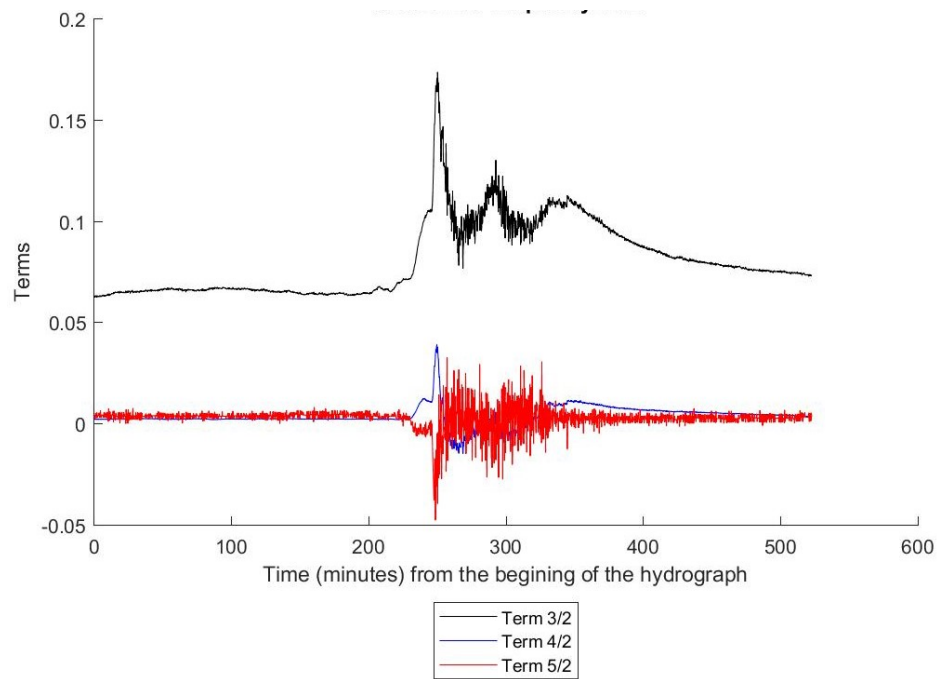


Figure 5.25 The three terms of Saint-Venant equation over time for the hydrograph happening on May 9th 2019

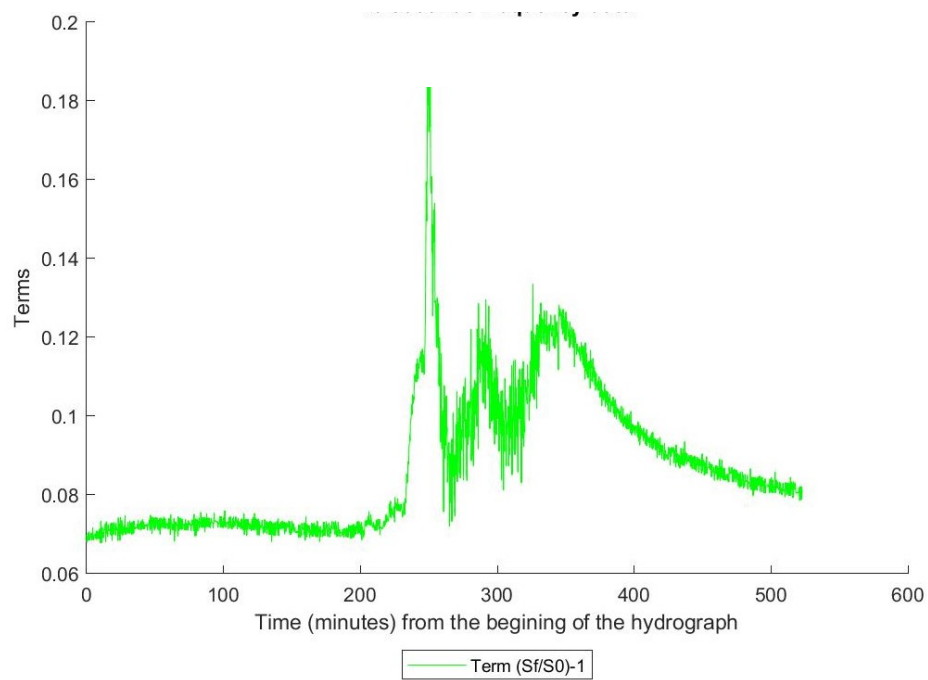


Figure 5.26 The sum of all the three terms of Saint-Venant equation (Sf/S_0-1) for the hydrograph occurred on May 9th 2019

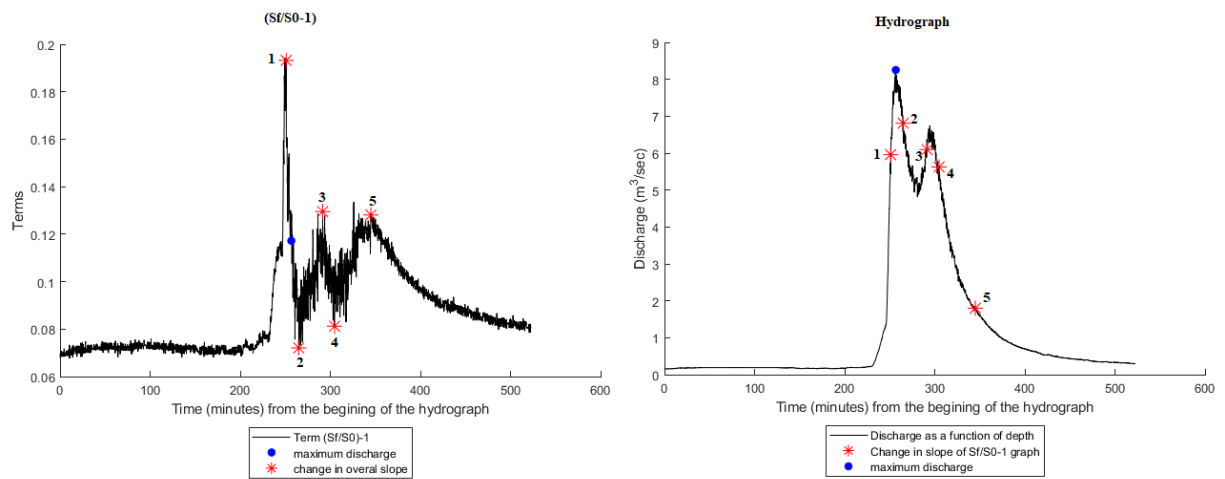


Figure 5.27 The comparison of Sf/S0-1 with the hydrograph occurred on May 9th 2019

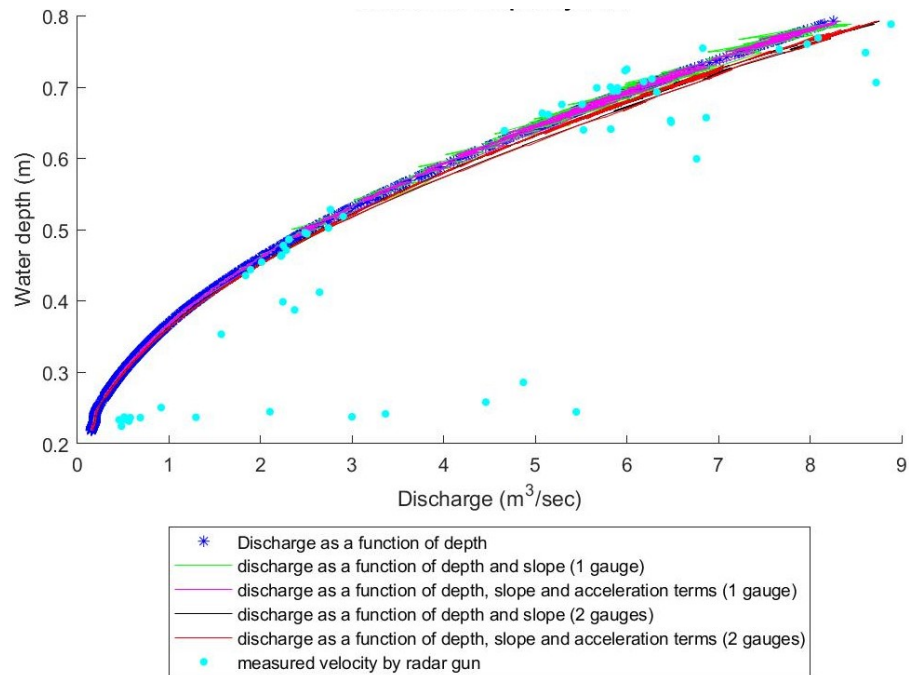


Figure 5.28 Rating curve for the hydrograph happened on May 9th 2019

A.5. Hydrograph June 13th 2019

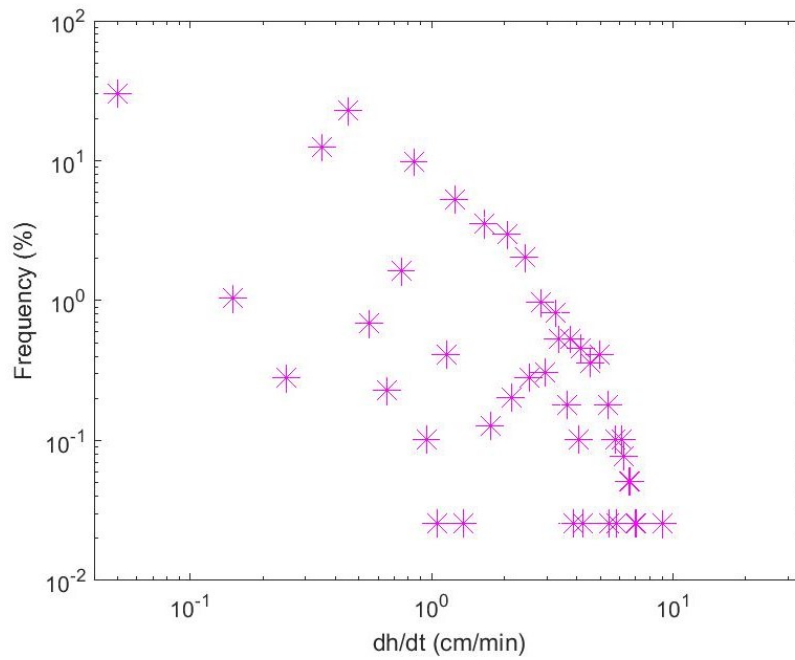


Figure 5.29 changes of flow depth over time for the hydrograph observed on June 13th 2019

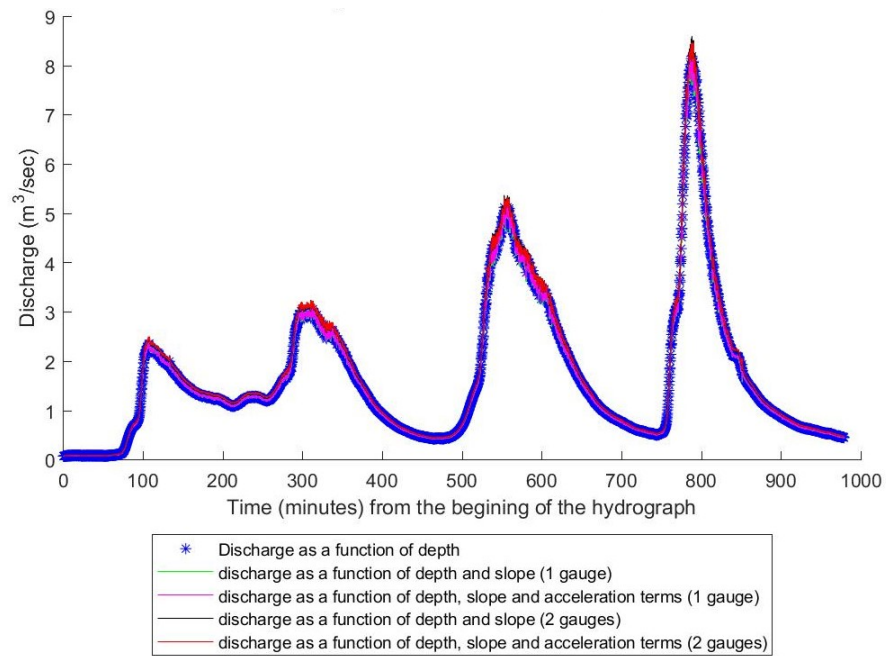


Figure 5.30 Hydrograph for the flow occurred on June 13th 2019

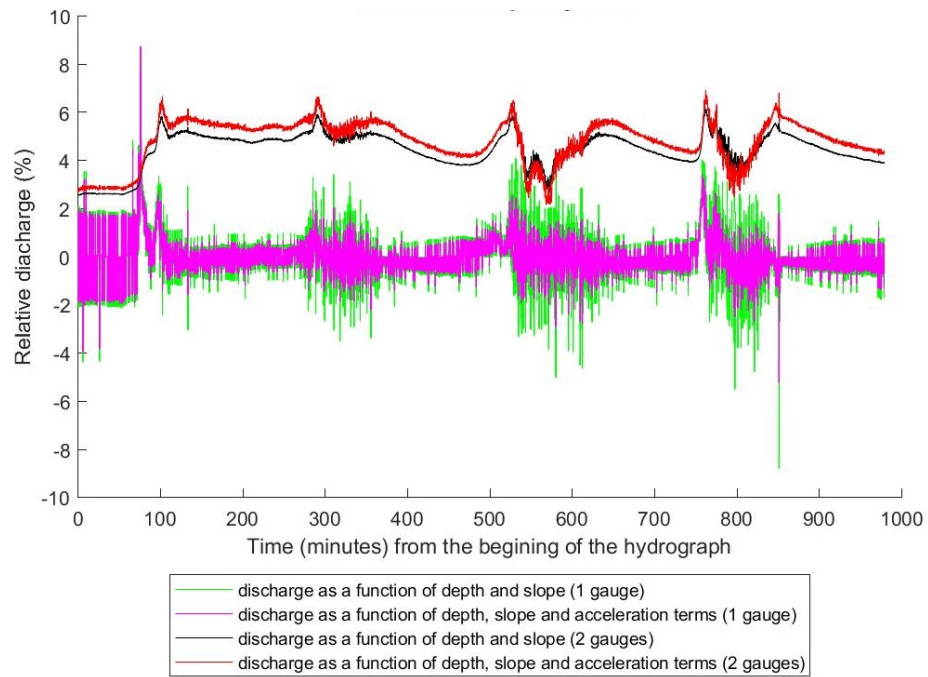


Figure 5.31 The relative discharge graph for the hydrograph observed on June 13th 2019

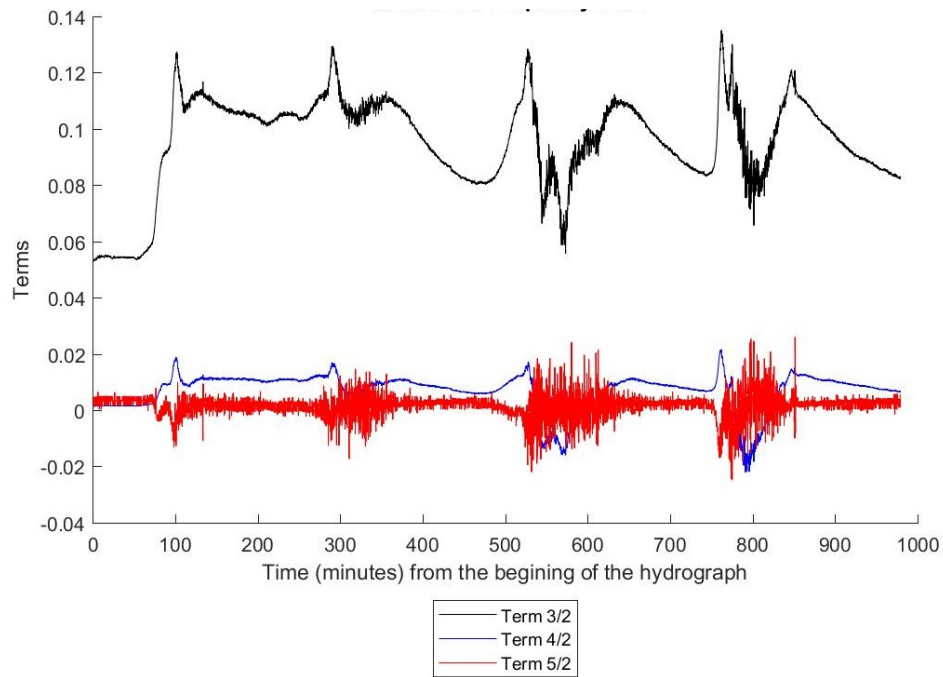


Figure 5.32 The three terms of Saint-Venant equation over time for the hydrograph happening on June 13th 2019

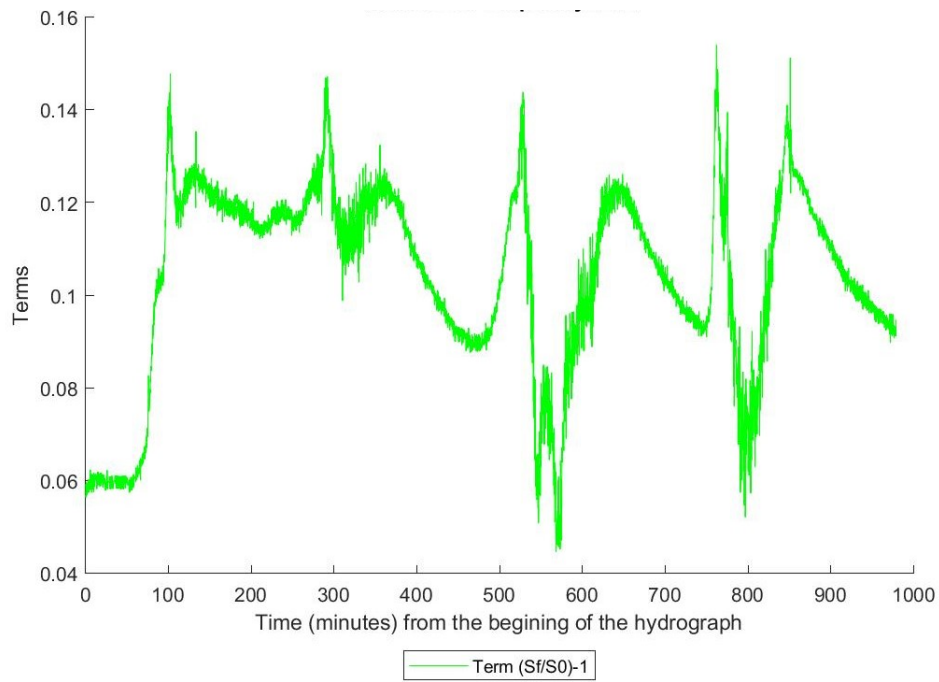


Figure 5.33 The sum of all the three terms of Saint-Venant equation (Sf/S_0-1) for the hydrograph occurred on June 13th 2019

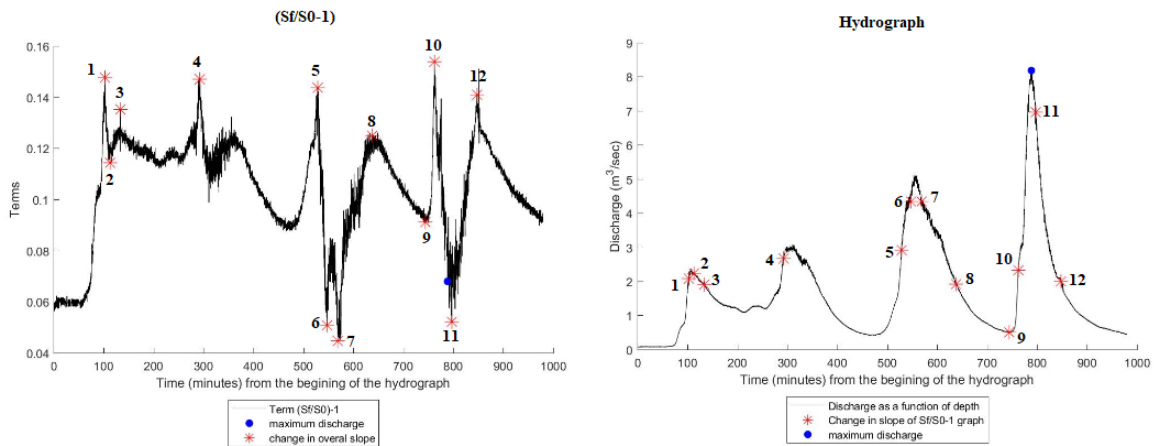


Figure 5.34 The comparison of Sf/S_0-1 with the hydrograph occurred on June 13th 2019

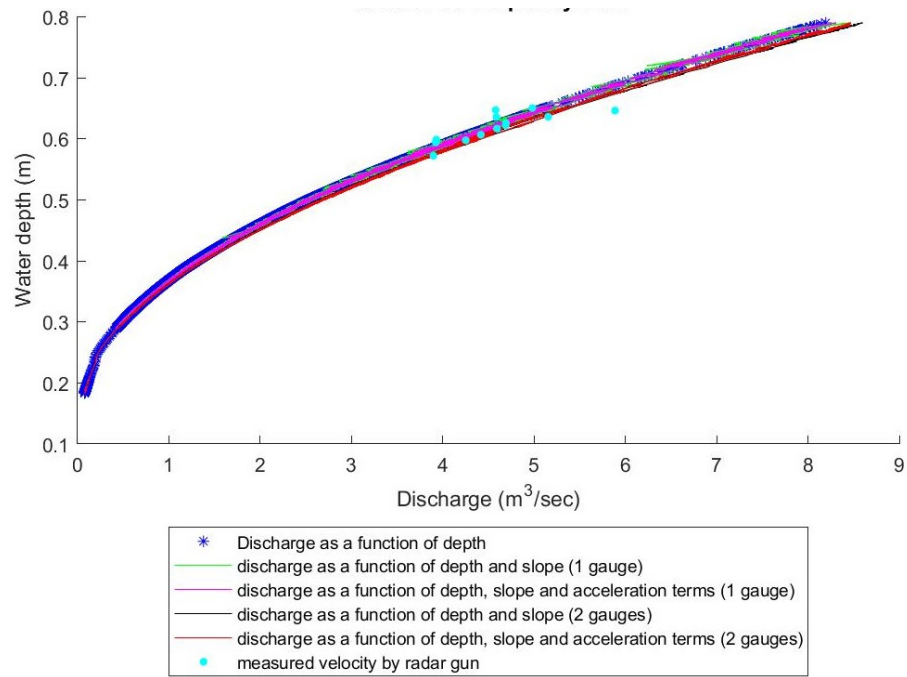


Figure 5.35 Rating curve for the hydrograph happened on June 13th 2019

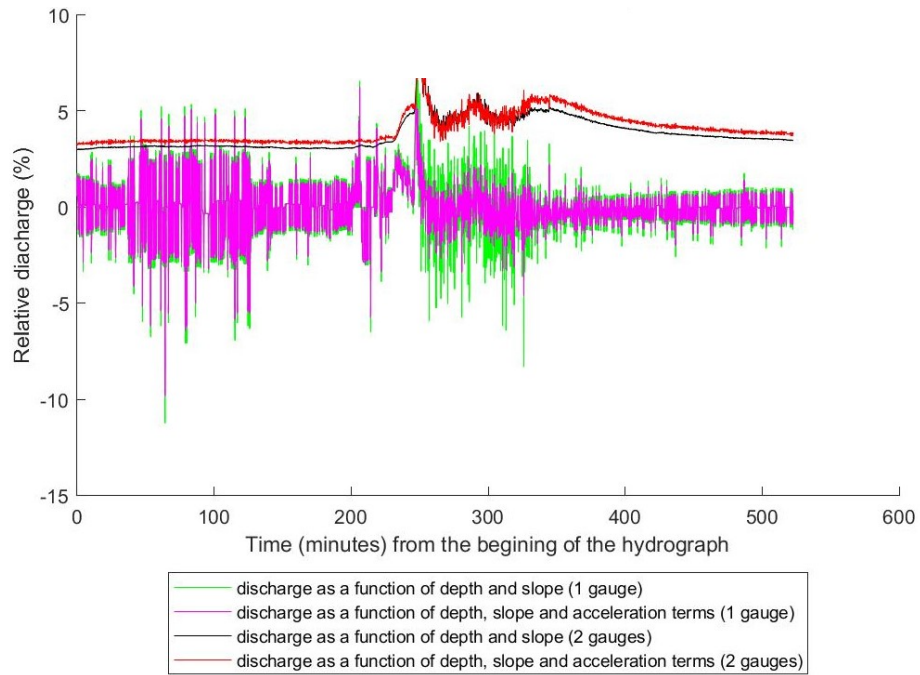


Figure 5.36 Relative discharge graph for the hydrograph observed on June 13th 2019

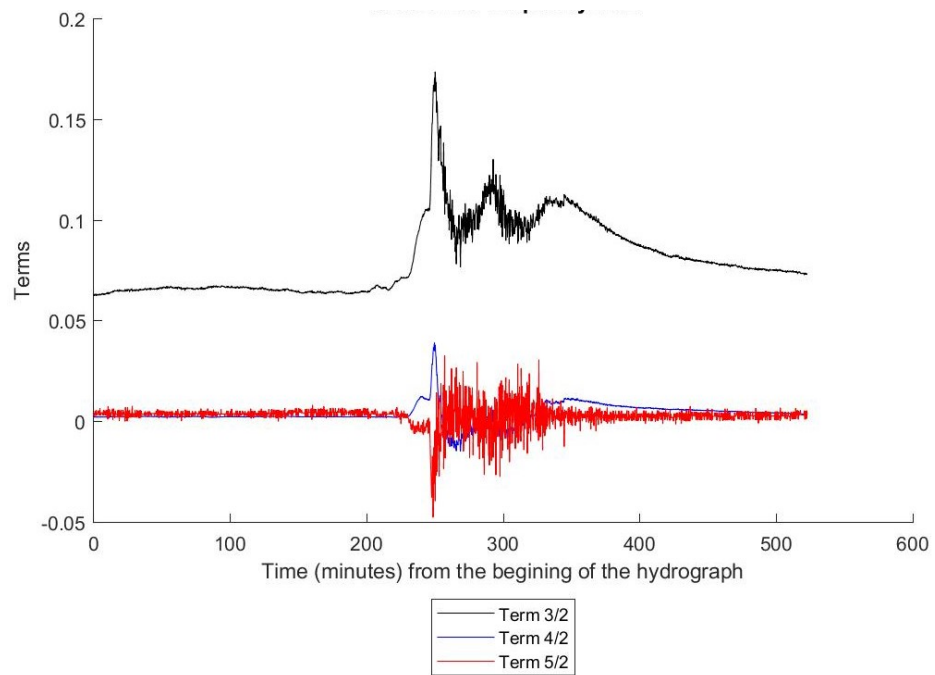


Figure 5.37 Terms of Saint-Venant momentum equation for the hydrograph observed on June 13th 2019

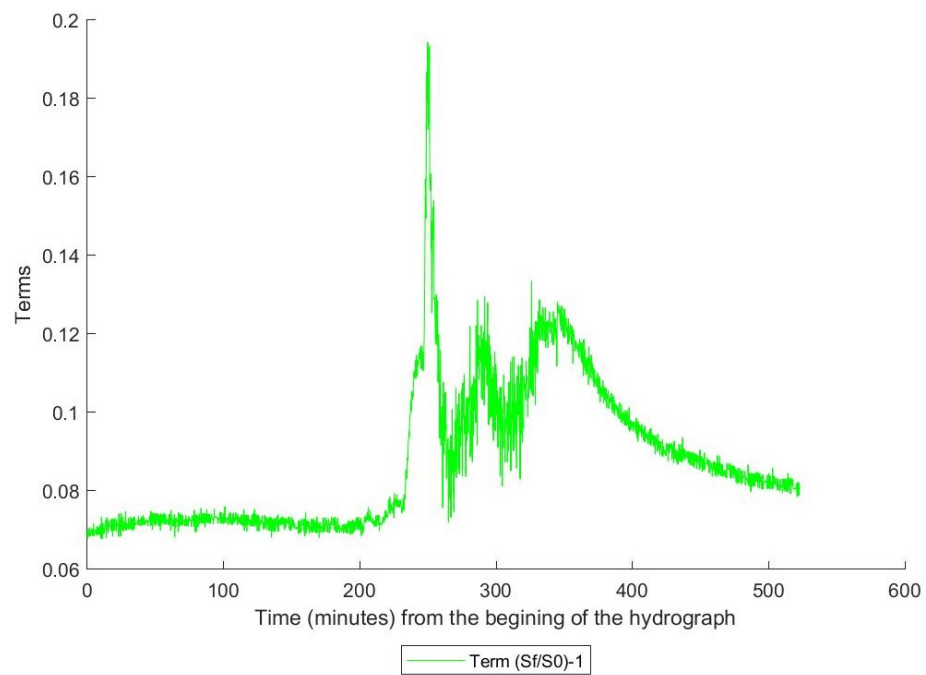


Figure 5.38 The magnitude of the term $Sf/S_0 - 1$ for the hydrograph observed on June 13th 2019

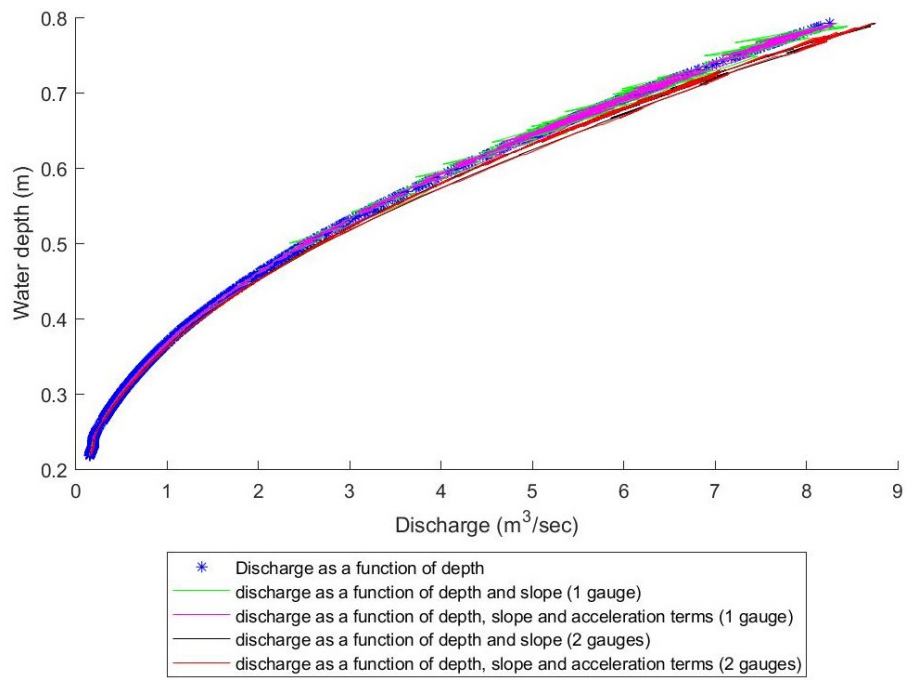


Figure 5.39 Rating curve for all the 5 calculated discharges for hydrograph observed on June 13th 2019

Appendix B

Hydrographs with Various Frequencies

In this section, the results are provided for six frequencies of data similar to the last section for the last presented hydrograph. **Figure 5.40** shows the beginning of the hydrographs for 15 seconds and 15 minutes data frequencies. Similar to the last hydrograph it can be seen that the beginning of the hydrograph is different for these two data frequencies.

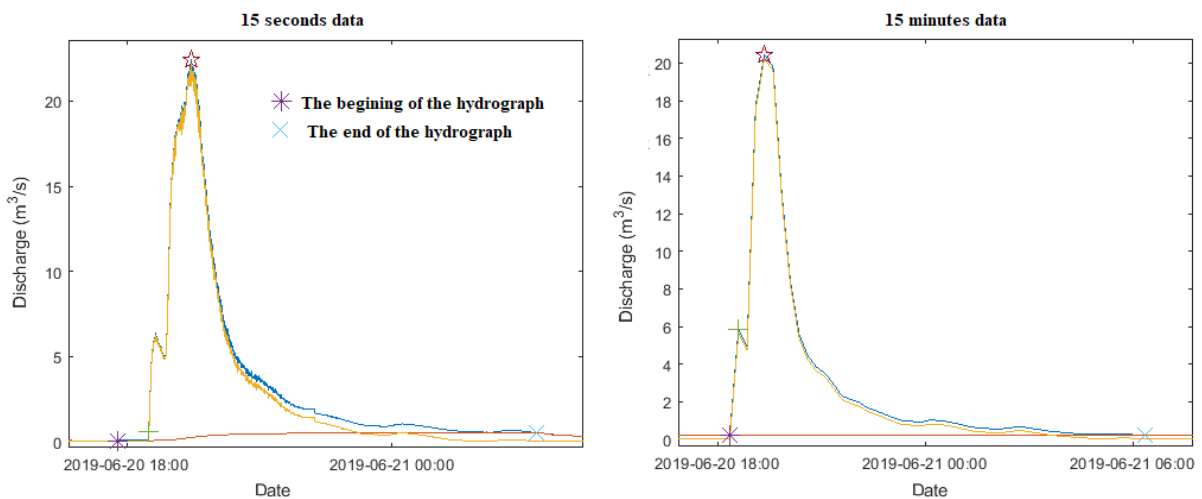


Figure 5.40 The start, end and the peak value of the hydrograph for 15 seconds and 15 minutes data frequencies

Figure 5.41 shows the hydrographs for all the six frequencies. It can be seen that the peak discharge decreases and the shape of the hydrograph changes by decreasing the data frequency. The second small bump that happens near peak discharge is disappearing from the hydrograph with 15 minutes data frequency. The shape of the hydrograph stays unchanged even for the data with frequency of 2 minutes.

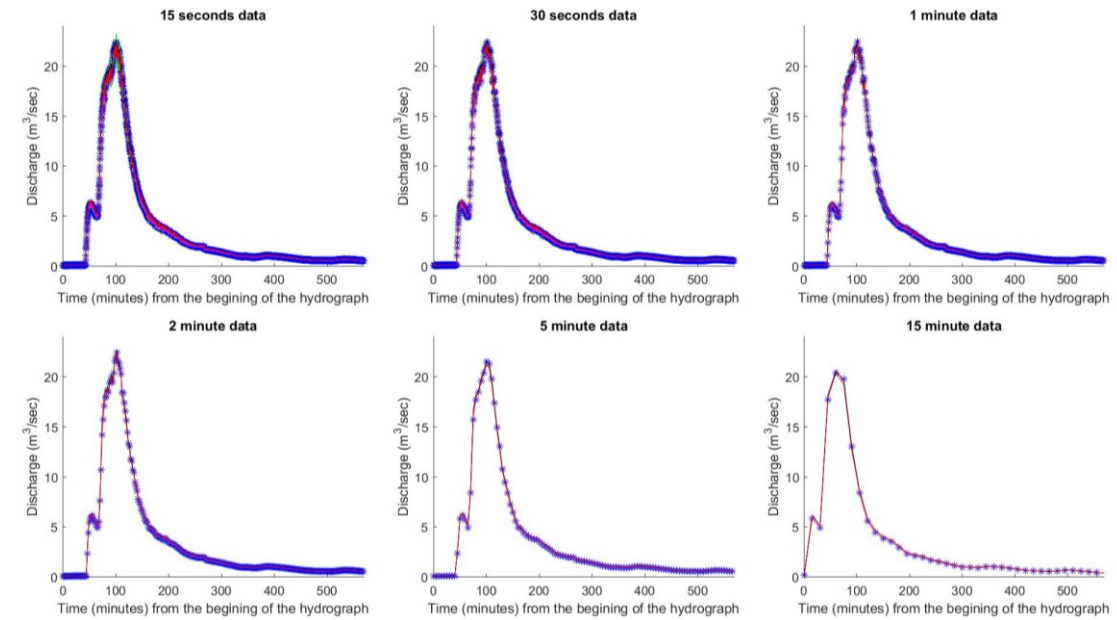


Figure 5.41 Hydrograph for six different frequencies of the surface water elevation data

The relative discharges for all the six frequencies are shown in Figure 5.42. It can be seen in this figure that the maximum value for all four relative discharges decreases by decreasing the data frequency. The minimum values do not change significantly. By decreasing the data frequency, the fluctuation of the J1 and F1 are vanishing.

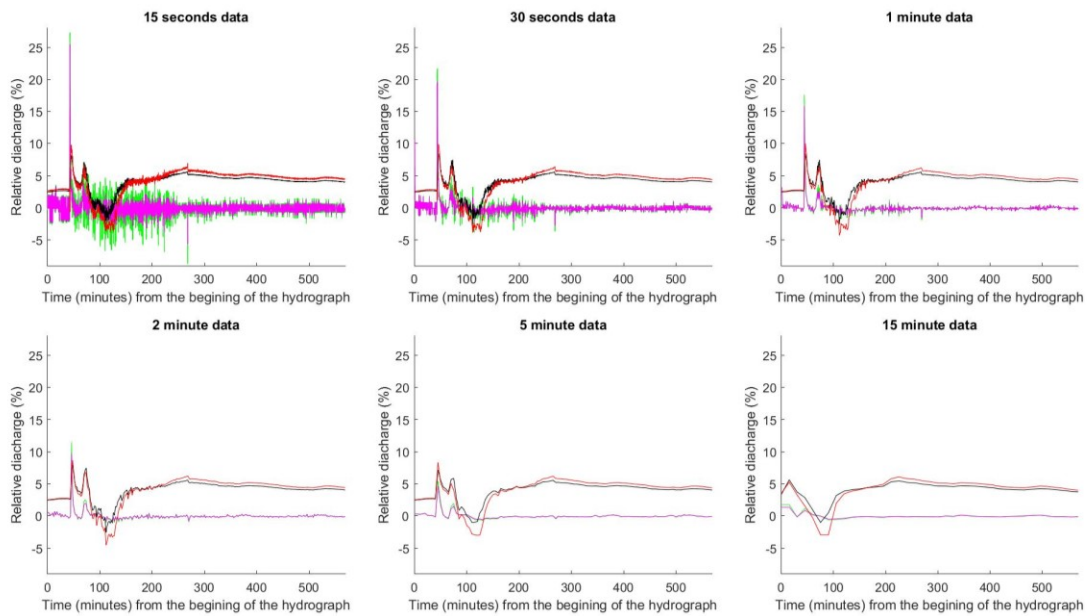


Figure 5.42 Relative discharge for all the six frequencies of water surface elevation data

Figure 5.43 shows the three terms of Saint-Venant equation with respect to the channel bottom slope for all the six data frequencies. This figure also shows that the fluctuation of term5 is dampening, the maximum value for term3 is decreasing, and the rise of term4 during the rising limb is vanishing by decreasing the data frequency. Decreasing data frequency does not affect the unsteadiness of falling limb discharges, while it affects the rising limb discharges significantly.

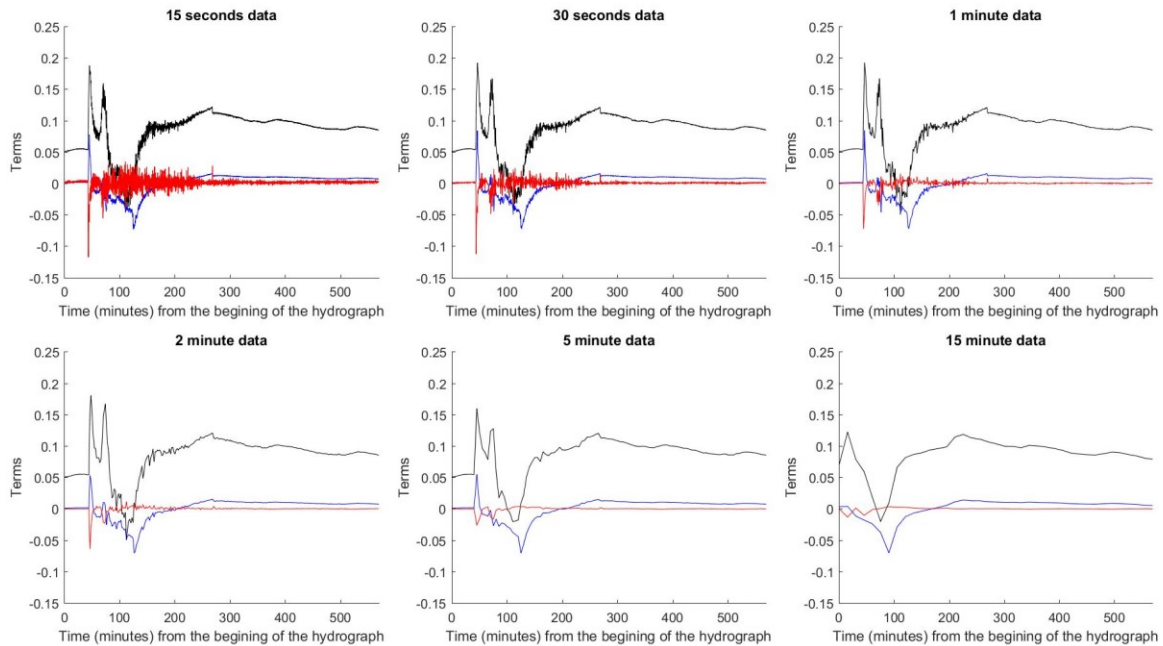


Figure 5.43 The three terms of Saint-Venant equations with respect to channel bottom slope for all the six data frequencies

In Figure 5.44 it can be seen that the maximum value drops to 0.1 for the 15-minute frequency data. This maximum value stays the same for the first three data frequencies (the three top figures). Comparing all five data frequencies with the 15-second data frequency, it can be seen that both figures with 30-seconds and 1-minute data frequencies follow the similar shape of the graph, with the same maximum and minimum values. This shows that 1 minutes data frequency is the highest that gives us the best prediction of unsteadiness for this hydrograph in Wilket Creek.

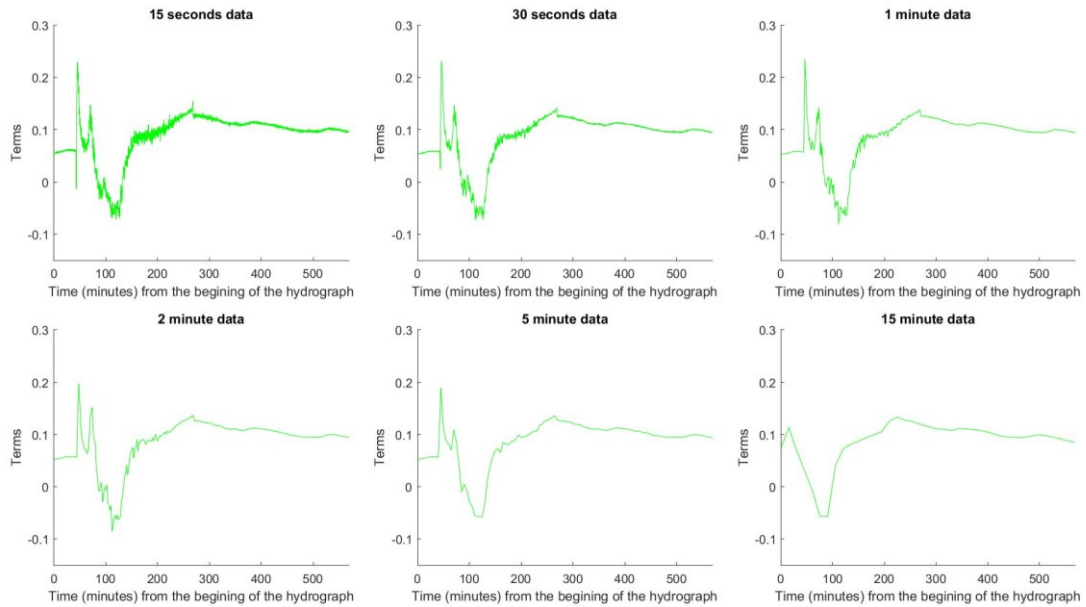


Figure 5.44 Sf/S0-1 for all the six data frequencies

The rating curves are shown in Figure 5.45 for all the six data frequencies. As it was expected, the loop becomes smaller as the data frequency decreases. In this figure it can be seen that the unsteady discharges for the rising limb stay on the right side of the graph and the unsteady discharges for the falling limb stay on the left side of the steady flow graph.

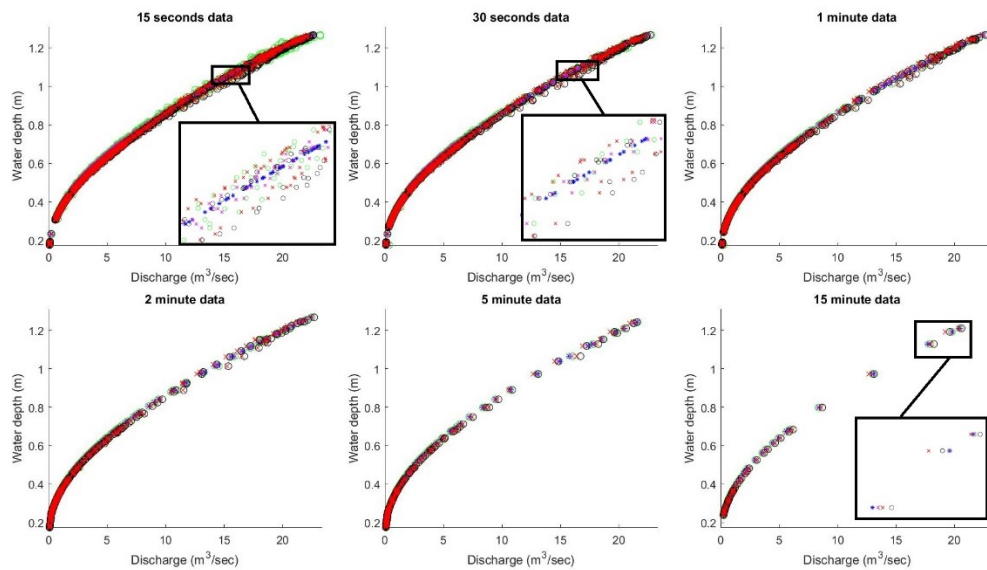


Figure 5.45 Rating curves for all the six data frequencies for the hydrograph observed on June 20th 2019

In Figure 5.46 it can be seen that the maximum value of change in depth over time decreases by decreasing the data frequency. In this figure it can be seen that even by decreasing the frequency from 15 seconds to 30 seconds the maximum value is reduced from 12.75 cm/min to 7.2 cm/min. This is a significant change in dh/dt , which consequently can affect the unsteadiness of the flow and can lead to calculating incorrect unsteady flow discharges.

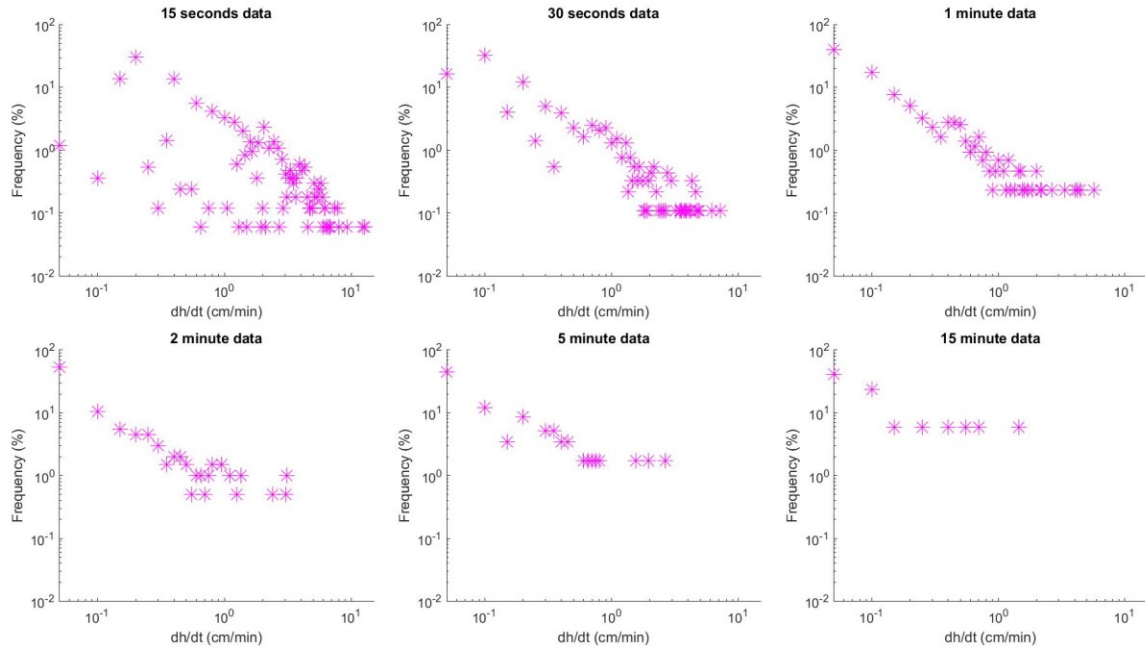


Figure 5.46 changes of flow depth over time (cm/min) for the six data frequencies.

Appendix C

MATLAB Codes

Contents

- Main code
- Outputs:

```
function BM_outputs=Bayesian_Method(BM_inputs)
```

```
% Inputs:
Area=BM_inputs.Area;
riverdata1=BM_inputs.riverdata1;
TimeValues2=BM_inputs.TimeValues2;
TimeValues=BM_inputs.TimeValues;
Surface_water_elevation2=BM_inputs.Surface_water_elevation2;
Surface_water_elevation=BM_inputs.Surface_water_elevation;
River_data=BM_inputs.River_data;
% Year=BM_inputs.Year;
Tw=BM_inputs.Tw;
WP=BM_inputs.WP;
Rh=BM_inputs.Rh;
Rh2=BM_inputs.Rh2;
Tw2=BM_inputs.Tw2;
Area2=BM_inputs.Area2;
S=BM_inputs.S;
g=BM_inputs.g;
S0=BM_inputs.S0;
Y_Centroid_BU=BM_inputs.Y_Centroid_BU;
Y_Centroid=BM_inputs.Y_Centroid;
% Date_start_num=BM_inputs.Date_start_num;
% Date_end_num=BM_inputs.Date_end_num;
thalweg_elevation=BM_inputs.thalweg_elevation;
thalweg_elevation_BU=BM_inputs.thalweg_elevation_BU;
```

Not enough input arguments.

Error in Bayesian_Method (line 4)
Area=BM_inputs.Area;

Main code

Finding Ks in this method: Manning's n will be calculated using 6 methods based on USGS website (<https://il.water.usgs.gov/proj/nvalues/equations.shtml>) At the end, we added another Manning's n (n7) found in the paper Lopez and Barragan (2008). This Manning's is based on a logarithmic method similar to Kuelegan roughness. (Kinematic discharge calculations based on SWE measured at the site)

```
Dis_Bay.Sw.n1=zeros([1,length(Area)]);
Dis_Bay.S0.n1=zeros([1,length(Area)]);
Dis_Bay.Sw.n2=zeros([1,length(Area)]);
Dis_Bay.S0.n2=zeros([1,length(Area)]);
Dis_Bay.Sw.n3=zeros([1,length(Area)]);
Dis_Bay.S0.n3=zeros([1,length(Area)]);
Dis_Bay.Sw.n4=zeros([1,length(Area)]);
Dis_Bay.S0.n4=zeros([1,length(Area)]);
Dis_Bay.Sw.n5=zeros([1,length(Area)]);
Dis_Bay.S0.n5=zeros([1,length(Area)]);
Dis_Bay.Sw.n6=zeros([1,length(Area)]);
Dis_Bay.S0.n6=zeros([1,length(Area)]);
Dis_Bay.S0.n7=zeros([1,length(Area)]);
D84=riverdata1.D84;
Se=zeros([1,length(Area)]);
distace_between_2_gauges=riverdata1.distace_between_2_gauges;
timestep_backup=(6.95*(10)^(-4))/4;

Num_High_Cn=[];
for t=1:length(Area)
    %finding Se using the backup section
    for i=1:(length(TimeValues2));
        Diff_time=TimeValues(t)-TimeValues2(i);
        if Diff_time<=timestep_backup && Diff_time>0;
            Se(t)=(abs(Surface_water_elevation2(i)-Surface_water_elevation(t)))/distace_between_2_gauges;
            Rh2_t(t)=Rh2(i);
            Tw2_t(t)=Tw2(i);
            Area2_t(t)=Area2(i);
        elseif Diff_time>=timestep_backup && Diff_time<0;
            Se(t)=(abs(Surface_water_elevation2(i)-Surface_water_elevation(t)))/distace_between_2_gauges;
            Rh2_t(t)=Rh2(i);
            Tw2_t(t)=Tw2(i);
            Area2_t(t)=Area2(i);
        elseif Diff_time==0;
            Se(t)=(abs(Surface_water_elevation2(i)-Surface_water_elevation(t)))/distace_between_2_gauges;
            Rh2_t(t)=Rh2(i);
            Tw2_t(t)=Tw2(i);
            Area2_t(t)=Area2(i);
        end
    end

    %finding dhoverdx using the backup section
```

```

tf1 = strcmp(River_data.section,'GCrk10');
if tf1==1
for i=1:(length(TimeValues2));
Diff_time=TimeValues(t)-TimeValues2(i);
if Diff_time<=timestep_backup && Diff_time>0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
elseif Diff_time>=timestep_backup && Diff_time<0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
elseif Diff_time==0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
end
end
else
for i=1:(length(TimeValues2));
Diff_time=TimeValues(t)-TimeValues2(i);
if Diff_time<=timestep_backup && Diff_time>0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
elseif Diff_time>=timestep_backup && Diff_time<0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
elseif Diff_time==0;
dhooverdx(t)=-(((Surface_water_elevation2(i)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(i)*Y_Centroid_BU(i))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(i)-Area(t))/Area(t))*100;
end
end
end

if TimeValues(t)>(737616+2550/3600) && TimeValues(t)<(737620+61200/3600/24)
if t>=21662-5;
Se(t)=(abs(Surface_water_elevation2(21660)-Surface_water_elevation(t)))/distace_between_2_gauges;
dhooverdx(t)=-(((Surface_water_elevation2(21660)-thalweg_elevation_BU)-(Surface_water_elevation(t)-thalweg_elevation)))/distace_between_2_gauges;
Area_yc(t)=-(((Area2(21660)*Y_Centroid_BU(21660))-(Area(t)*Y_Centroid(t)))/distace_between_2_gauges));
Area_diff(t)=((Area2(21660)-Area(t))/Area(t))*100;
Rh2_t(t)=Rh2(21660);
Tw2_t(t)=Tw2(21660);
Area2_t(t)=Area2(21660);
end
end

% if t==(21661-5)
% disp('asal')
% end

if Se(t)==0;
error('I think the delta t between measured data is more than 2 mins. If it is, you have to change 6.95*(10)^(-4) to another value. This is the variable ca
end

%sometimes some data are missing from either the main cross section or
%the backup cross section. That is why I need to cut those times or
%find a way for my code to run.
% if t==28303;
% timestep_backup=3.473*(10^(-3));
% end

% % 1. Strickler (1923)
% n1=(1/21.1)*(D50^(1/6));
% Ks1=1/n1;
% % 2. Meyer Peter Mueller (1948)
% n2=(1/26)*(D90^(1/6));
% Ks2=1/n2;

% % 3. Limerinos (1970)
% From 3 to 6 n changes by depth
n3=(0.113*(Rh(t)^(1/6)))/(1.16+(2*log10(Rh(t)/D84)));
Ks3(t)=1/n3;
% 4. Bray and Davar1 (1987)
n4=0.094*(Se(t)^(1/6));
Ks4(t)=1/n4;
% 5. Bray and Davar2 (1987)
n5=(0.113*(Rh(t)^(1/6)))/(1.9*((Rh(t)/D84)^(1/4)));
Ks5(t)=1/n5;
% 6. Bray and Davar3 (1987)
n6=(0.113*(Rh(t)^(1/6)))/(1.1+(2*log10(Rh(t)/D84)));
Ks6(t)=1/n6;
% 7. Lopez and Barragan (2008)
kprimes=2.8*D84;
n7=(Rh(t)^(1/6))/(5.75*sqrt(g)*log10(12.2*Rh(t)/kprimes));
Ks7(t)=1/n7;
% finding Discharge
% Dis_Bay.Sw.n1(t)=Area(t)*Ks1*(Se(t)^(1/2))*(Rh(t)^(2/3));

```

```

% Dis_Bay.S0.n1(t)=Area(t)*Ks1*(S0^(1/2))*(Rh(t)^(2/3));
% Dis_Bay.Sw.n2(t)=Area(t)*Ks2*(Se(t)^(1/2))*(Rh(t)^(2/3));
% Dis_Bay.S0.n2(t)=Area(t)*Ks2*(S0^(1/2))*(Rh(t)^(2/3));
Dis_Bay.Sw.n3(t)=Area(t)*Ks3(t)*(Se(t)^(1/2))*(Rh(t)^(2/3));
Dis_Bay.S0.n3(t)=Area(t)*Ks3(t)*(S0^(1/2))*(Rh(t)^(2/3));
Dis_Bay.Sw.n4(t)=Area(t)*Ks4(t)*(Se(t)^(1/2))*(Rh(t)^(2/3));
Dis_Bay.S0.n4(t)=Area(t)*Ks4(t)*(S0^(1/2))*(Rh(t)^(2/3));
Dis_Bay.Sw.n5(t)=Area(t)*Ks5(t)*(Se(t)^(1/2))*(Rh(t)^(2/3));
Dis_Bay.S0.n5(t)=Area(t)*Ks5(t)*(S0^(1/2))*(Rh(t)^(2/3));
Dis_Bay.Sw.n6(t)=Area(t)*Ks6(t)*(Se(t)^(1/2))*(Rh(t)^(2/3));
Dis_Bay.S0.n6(t)=Area(t)*Ks6(t)*(S0^(1/2))*(Rh(t)^(2/3));
Dis_Bay.S0.n7(t)=Area(t)*Ks7(t)*(S0^(1/2))*(Rh(t)^(2/3));
G=' ';
negative_dis_num=[];
if Dis_Bay.S0.n7(t)<0
%   Dis_Bay.S0.n7(t)=1*(10^(-10));
   Dis_Bay.S0.n7(t)=3.1*(10^(-4));
   G='NOTE: The water depth is very low that gives negative discharge. I changed the negative discharge to a super low value in order to eliminate my code from gi
negative_dis_num=[negative_dis_num,t];
end
tf1 = strcmp(River_data.section,'SCrk40');
if tf1==1
    if TimeValues(t)>736330 && TimeValues(t)<736695
        if Dis_Bay.S0.n7(t)<3.1*(10^(-4))
            Dis_Bay.S0.n7(t)=3.1*(10^(-4));
        end
    end
    if TimeValues(t)>736696 && TimeValues(t)<737060
        if Dis_Bay.S0.n7(t)<3.216*(10^(-4))
            Dis_Bay.S0.n7(t)=3.216*(10^(-4));
        end
    end
end

% velocity for the backup section:
kprimes=2.8*riverdata1.D84_BU;
% kprimes=2.8*D84;
n7=(Rh2_t(t)^(1/6))/(5.75*sqrt(g)*log10(12.2*Rh2_t(t)/kprimes));
Ks7(t)=1/n7;
velocity_BU(t)=Ks7(t)*(S0^(1/2))*(Rh2_t(t)^(2/3));
Discharge_BU(t)=velocity_BU(t)*Area2_t(t);

diff_dis_x(t)=(Discharge_BU(t)-Dis_Bay.S0.n7(t))*100/Dis_Bay.S0.n7(t);
diff_dis_over_dx(t)=(Discharge_BU(t)-Dis_Bay.S0.n7(t))/distace_between_2_gauges;

% Courant Number at time step t
%for the cross section 1
vel_term=abs(Dis_Bay.S0.n7(t)/Area(t))+sqrt(g*Area(t)/Tw(t));
% for the backup section (section2)
vel_term_BU=abs(velocity_BU(t))+sqrt(g*Area2_t(t)/Tw2_t(t));
max_vel_term=max(vel_term,vel_term_BU);
if t~=1
dt=(TimeValues(t)-TimeValues(t-1))*24*60*60;
else
    dt=(TimeValues(2)-TimeValues(1))*24*60*60;
end
dx=distace_between_2_gauges;
if dt>600
    Cn(t)=0;
else
    Cn(t)=(dt/dx)*max_vel_term;
end
% if t==39202-5
%   disp('asal')
% end
if Cn(t)>1
    Num_High_Cn=[Num_High_Cn,t];
end
end
Count_high_Cn=length(Num_High_Cn);
mean_diff_area=mean(Area_diff);
mean_diff_dis=mean(diff_dis_x);
deltat=timestep_backup*24*60*60;
for i=2:length(Area)
    Areadiff_m(i)=(Area(i)-Area(i-1))/deltat;
%   Areadiff_bu(i)=(Area2_t(i)-Area2_t(i-1))/deltat;
%   Areadiff(i)=(Areadiff_bu(i)+Areadiff_m(i))/2;
end
continuity_eq=diff_dis_over_dx+Areadiff_m;
mean_continuity_eq=mean(continuity_eq);
mean_cn=mean(Cn);
mean_Se=mean(Se);

```

Outputs:

```

BM_outputs.Dis_Bay=Dis_Bay;
BM_outputs.Se=Se;
BM_outputs.dhoverdx=dhoverdx;
BM_outputs.G=G;
BM_outputs.negative_dis_num=negative_dis_num;

```

```
BM_outputs.velocity_BU=velocity_BU;  
BM_outputs.Cn=Cn;  
BM_outputs.Num_High_Cn=Num_High_Cn;  
BM_outputs.Count_high_Cn=Count_high_Cn;  
BM_outputs.Area_yc=Area_yc;  
BM_outputs.Area_diff=Area_diff;  
BM_outputs.continuity_eq=continuity_eq;
```

```
end
```

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Contents

- Main code
- First Jone's formula based on the data from two gauges
- second method for Jone's formula (modified Jone's formula). This method is the method that can be used for when we only have the data of one gauge at the creek. This method does not have the changes of the flow variables in distance
- second method for Jone's formula. This method is the method that can be used for when we only have the data of one gauge at the creek. It is based on the channel bottom slope
- finding the matrix with baseflow equal to -1000
- Outputs:

```
function JM_outputs=Jones_Method(JM_inputs)
```

```
% inputs:
Area=JM_inputs.Area;
nvalue=JM_inputs.nvalue;
Slope1=JM_inputs.Slope1;
Dis_Bay=JM_inputs.Dis_Bay;
eval(['Dis_Bay_S_n = Dis_Bay.',Slope1, '.',nvalue, ';']);
Se=JM_inputs.Se;
dhoverdx=JM_inputs.dhoverdx;
S0=JM_inputs.S0;
h=JM_inputs.h;
TimeValues=JM_inputs.TimeValues;
K_mat=JM_inputs.K_mat;
r_mat=JM_inputs.r_mat;
Area_yc=JM_inputs.Area_yc;
% Dis_Bay1_hydrography1
% TimeValues_hydrography1
```

Not enough input arguments.

Error in Jones_Method (line 4)
Area=JM_inputs.Area;

Main code

```
Dis_Jon=zeros([1,length(Area)]);
Dis_Jon(1)=Dis_Bay_S_n(1);
%using Sw and n1
for t=2:length(Area);
% Dis_spec=Dis_Bay.Sw.n1(t);
% Dis_assumed=Dis_spec;
timestep=(TimeValues(t)-TimeValues(t-1))*24*60*60; % time step should be in seconds
% Dis_found=Dis_spec*((1+((Area(t)*0.85)/(Se(t)*Dis_assumed)*(abs(h(t-1)-h(t))/timestep)))^(1/2));
% Diff1=Dis_found-Dis_assumed;
% while Diff1>10^(-4) || Diff1<-10^(-4)
%     if Diff1>10^(-4)
%         Dis_assumed=Dis_assumed+(Diff1/5);
%     end
%     if Diff1<-10^(-4)
%         Dis_assumed=Dis_assumed-(abs(Diff1/5));
%     end
% Dis_found=Dis_spec*((1+((Area(t)*0.85)/(Se(t)*Dis_assumed)*(abs(h(t-1)-h(t))/timestep)))^(1/2));
% Diff1=Dis_found-Dis_assumed;
% end
% Dis_Jon.Sw.n1(t)=Dis_found;
```

```

%using S0 and n1
Dis_spec=Dis_Bay_S_n(t);

%this is a new version of finding Qs. The data from here is very similar to
%the data from my code below. Dis_Jon.S0.n1_2 is almost the same as
%Dis_Jon.S0.n1. I can replace them with this. Here I am using "Set" instead
%of "Se". "Set" is the mean value of the Se between this time step and next
%time step. The reason is that after the time step the wave moves. The
%water slope at time step t is not between this gauge and next gauge at t.
%it is water slope at this gauge at time step t.
a2=Dis_spec^2;
c=((0.85*Area(t))/Se(t))*(abs(h(t)-h(t-1))/timestep);
a2c=a2*c;
P=[1,0,-a2,-a2c];
r=roots(P);
for i=1:3
    if r(i)>0;
        Dis_Jon_2(t)=r(i);
    end
end
Dis_assumed=Dis_spec;
Dis_found=Dis_spec*((1+((Area(t)*0.85)/(Se(t)*Dis_assumed)*(abs(h(t-1)-h(t))/timestep)))^(1/2));
Diff1=Dis_found-Dis_assumed;
while Diff1>10^(-4) || Diff1<-10^(-4)
    if Diff1>10^(-4)
        Dis_assumed=Dis_assumed+(Diff1/5);
    end
    if Diff1<-10^(-4)
        Dis_assumed=Dis_assumed-(abs(Diff1/5));
    end
end
Dis_found=Dis_spec*((1+((Area(t)*0.85)/(Se(t)*Dis_assumed)*(abs(h(t-1)-h(t))/timestep)))^(1/2));
Diff1=Dis_found-Dis_assumed;
end
Dis_Jon(t)=Dis_found;
end

```

First Jone's formula based on the data from two gauges

```

Dis_Jon_2_Gauges=zeros([1,length(Area)]);
Dis_Jon_2_Gauges(1)=Dis_Bay_S_n(1);
Sf_2_Gauges=zeros([1,length(Area)]);
Sf_2_Gauges(1)=S0;
% Dis_Jon=zeros([1,length(Area)]);
% Dis_Jon(1)=Dis_Bay_S_n(1);
for t=2:length(Area);
    timestep=(TimeValues(t)-TimeValues(t-1))*24*60*60;
    Dis_spec=Dis_Bay_S_n(t);
    Dis_assumed=Dis_spec;
% Dis_found=Dis_spec*((1-(dhoverdx(t)/(S0)))^(1/2));
Dis_found=Dis_spec*((1-(Area_yc(t)/(Area(t)*S0)))^(1/2));
Diff1=Dis_found-Dis_assumed;
while Diff1>10^(-4) || Diff1<-10^(-4)
    if Diff1>10^(-4)
        Dis_assumed=Dis_assumed+(Diff1/5);
    end
    if Diff1<-10^(-4)
        Dis_assumed=Dis_assumed-(abs(Diff1/5));
    end
    Dis_found=Dis_spec*((1-(Area_yc(t)/(Area(t)*S0)))^(1/2));
    Diff1=Dis_found-Dis_assumed;
end
Dis_Jon_2_Gauges(t)=Dis_found;
Sf_2_Gauges(t)=((Dis_found/Dis_spec)^2)*S0;
end

```

second method for Jone's formula (modified Jone's formula). This method is the method that can be used for when we only have the data of one gauge at the creek. This method does not have the changes of the flow variables in distance

```
Dis_Jon_one_Gauge2=zeros([1,length(Area)]);
Dis_Jon_one_Gauge2(1)=Dis_Bay_S_n(1);
Sf_Jon_one_Gauge2=zeros([1,length(Area)]);
Sf_Jon_one_Gauge2(1)=S0;
for t=2:length(Area);
    timestep=(TimeValues(t)-TimeValues(t-1))*24*60*60; % time step should be in seconds
    Dis_spec=Dis_Bay_S_n(t);
    Dis_assumed=Dis_spec;
    Dis_found=Dis_spec*((1+(Area(t)/(S0*K_mat(t)*Dis_assumed)*((h(t)-h(t-1))/timestep))+2/(3*(r_mat(t))^2)))^(1/2));
    Diff1=Dis_found-Dis_assumed;
    while Diff1>10^(-4) || Diff1<-10^(-4)
        if Diff1>10^(-4)
            Dis_assumed=Dis_assumed+(Diff1/5);
        end
        if Diff1<-10^(-4)
            Dis_assumed=Dis_assumed-(abs(Diff1/5));
        end
        Dis_found=Dis_spec*((1+(Area(t)/(S0*K_mat(t)*Dis_assumed)*((h(t)-h(t-1))/timestep))+2/(3*(r_mat(t))^2)))^(1/2));
        Diff1=Dis_found-Dis_assumed;
    end
    Dis_Jon_one_Gauge2(t)=Dis_found;
    Sf_Jon_one_Gauge2(t)=((Dis_found/Dis_spec)^2)*S0;
    if t==37487
        disp('asal')
    end
end
```

second method for Jone's formula. This method is the method that can be used for when we only have the data of one gauge at the creek. It is based on the channel bottom slope

```
Dis_Jon_one_Gauge=zeros([1,length(Area)]);
Dis_Jon_one_Gauge(1)=Dis_Bay_S_n(1);
for t=2:length(Area);
    timestep=(TimeValues(t)-TimeValues(t-1))*24*60*60; % time step should be in seconds
    Dis_spec=Dis_Bay_S_n(t);
    Dis_assumed=Dis_spec;
    Dis_found=Dis_spec*((1+(Area(t)/(S0*K_mat(t)*Dis_assumed)*((h(t)-h(t-1))/timestep))))^(1/2));
    Diff1=Dis_found-Dis_assumed;
    while Diff1>10^(-4) || Diff1<-10^(-4)
        if Diff1>10^(-4)
            Dis_assumed=Dis_assumed+(Diff1/5);
        end
        if Diff1<-10^(-4)
            Dis_assumed=Dis_assumed-(abs(Diff1/5));
        end
        Dis_found=Dis_spec*((1+(Area(t)/(S0*K_mat(t)*Dis_assumed)*((h(t)-h(t-1))/timestep))))^(1/2));
        Diff1=Dis_found-Dis_assumed;
    end
    Dis_Jon_one_Gauge(t)=Dis_found;
end
```

finding the matrix with baseflow equal to -1000

```
a=1;
for i=1:length(JM_inputs.Surface_runoff)
    for j=1:length(JM_inputs.Point_C_num_j)
        if i>JM_inputs.Point_A_num_j(j) && i<=JM_inputs.Point_C_num_j(j)
            b=j;
            Dis_Jon_hydrography1(a,b)=-1000;
        end
    end
end
```

```

    end
    if j>=2 && i>=JM_inputs.Point_A_num(j-1) && i<=JM_inputs.Point_B_num(j-1)
        b=j;
        Dis_Jon_hydrography1(a,b)=Dis_Jon(i);
    end
    if i==JM_inputs.Point_C_num_j(j);
        a=0;
    end
end
a=a+1;
end
sizee=size(JM_inputs.SWE_hydrography);
Dis_Jon_hydrography1((sizee(1)),:)=0;

```

Outputs:

```

JM_outputs.Dis_Jon=Dis_Jon;
JM_outputs.Dis_Jon_hydrography1=Dis_Jon_hydrography1;
JM_outputs.Dis_Jon_one_Gauge=Dis_Jon_one_Gauge;
JM_outputs.Dis_Jon_2_Gauges=Dis_Jon_2_Gauges;
JM_outputs.Sf_2_Gauges=Sf_2_Gauges;
JM_outputs.Dis_Jon_one_Gauge2=Dis_Jon_one_Gauge2;
JM_outputs.Sf_Jon_one_Gauge2=Sf_Jon_one_Gauge2;

```

```

end

```

Contents

- The second method of finding unsteady formulation using data from two gauges
- Now we make the output files back to a vector format:
- Output:

```
function Modified_Fread_Method_outputs=Modified_Fread_Method(MFM_Inputs)
```

```
Dis_Bay1_hydrography1=MFM_Inputs.Dis_Bay1_hydrography1;
Dis_Bay1_hydrography2=MFM_Inputs.Dis_Bay1_hydrography2;
Depth_hydrography=MFM_Inputs.Depth_hydrography2;
Depth_hydrography1=MFM_Inputs.Depth_hydrography1;
TimeValues_hydrography=MFM_Inputs.TimeValues_hydrography;
SWE_hydrography=MFM_Inputs.SWE_hydrography;
Area_hydrography=MFM_Inputs.Area_hydrography;
WP_hydrography=MFM_Inputs.WP_hydrography;
thalweg_elevation=MFM_Inputs.thalweg_elevation;
Depths=MFM_Inputs.D Depths;
Area_for_each_depth=MFM_Inputs.Area_for_each_depth;
S0=MFM_Inputs.S0;
g=MFM_Inputs.g;
Area=MFM_Inputs.Area;
Tw_hydrography=MFM_Inputs.Tw_hydrography;
velocity_BU_hydrography= MFM_Inputs.velocity_BU_hydrography;
dhoverdx_hydrography=MFM_Inputs.dhoverdx_hydrography;
distance_between_2_gauges=MFM_Inputs.distance_between_2_gauges;
section=MFM_Inputs.section;
Baseflow_hydro=MFM_Inputs.Baseflow_hydro;
Point_P_num=MFM_Inputs.Point_P_num;
bankful_depth=MFM_Inputs.bankful_depth;
Area_vc_hydrography=MFM_Inputs.Area_vc_hydrography;

%inputs for the second method:
TimeValues_hydrography_method2=TimeValues_hydrography;
Depth_hydrography_method2=Depth_hydrography;
Area_hydrography_method2=Area_hydrography;

Dis_Frd_hydrography1(1,1)=Dis_Bay1_hydrography1(1,1);
Dis_Frd_hydrography2(1,1)=Dis_Bay1_hydrography2(1,1);
size_hydrograph=size(Dis_Bay1_hydrography1);

for i=1:size_hydrograph(2)
    for j=1:size_hydrograph(1)
        if Depth_hydrography(j,i)==max(Depth_hydrography(:,i))
            G(i)=j;
            max_depth(i)=max(Depth_hydrography(:,i));
            peaktimestep(i)=(TimeValues_hydrography(G(i),i)-TimeValues_hydrography(1,i));
            break
        end
    end
end
zero_time=peaktimestep(peaktimestep==0);
empty_V=isempty(zero_time);
if empty_V==0
    error('select another start of the hydrograph')
end
Dis_Bay1_hydrography2_loop=Dis_Bay1_hydrography2;
for j=1:size_hydrograph(2)
    for i=2:size_hydrograph(1)
        if j==5 && i==2;
            disp('asal')
        end
        u=1;
        if SWE_hydrography(i,j)==0;
            if j~=(size_hydrograph(2))
                if SWE_hydrography(i-1,j)~=0;
                    u=0;
                end
                TimeValues_hydrography(i,j)=TimeValues_hydrography(1,j+1);
                Depth_hydrography(i,j)=Depth_hydrography(1,j+1);
                Area_hydrography(i,j)=Area_hydrography(1,j+1);
                WP_hydrography(i,j)=WP_hydrography(1,j+1);
            %
                Dis_Bay1_hydrography1(i,j)=Dis_Bay1_hydrography1(1,j+1);
                Dis_Bay1_hydrography2_loop(i,j)=Dis_Bay1_hydrography2_loop(1,j+1);
            else
                break
            end
        end
    end
    K(i,j)=(5/3)-((2/3)*(Area_hydrography(i,j)/WP_hydrography(i,j))*((WP_hydrography(i,j)-WP_hydrography(i-1,j))/(Area_hydrography(i,j)-Area_hydrography(i-1,j)))));
    % K(i,j)=1.416;
    tf1 = strcmp(section,'MCRk20');
    tf2 = strcmp(section,'GCRk10');
    if tf1==1
        if K(i,j)<0.75
            K(i,j)=0.75;
```

```

end
end
% if tf2==1;
    if Depth_hydrography(i,j)>bankful_depth
        K(i,j)=K(2,1);
    end
% end
timestep=(TimeValues_hydrography(i,j)-TimeValues_hydrography(i-1,j))*24*60*60; % time step should be in seconds
%Start of the calculations for specific n and S
Discharge=Dis_Bay1_hydrography2_loop;
Dis_last_time=Dis_Frd_hydrography2(i-1,j);
Dis_assumed=Discharge(i,j);
Average_SWE=((max_depth(j)+Depth_hydrography(1,j))/2)+thalweg_elevation;
Average_area=interp1(Depths,Area_for_each_depth,Average_SWE);
r(i,j)=(43200*(max(Discharge(:,j))+min(Discharge(:,j)))/((max(Depth_hydrography(:,j))-min(Depth_hydrography(:,j)))*Average_area)*peaktimestep(j)*S0*K(i,j); % in t
Dis_found=Discharge(i,j)*(((S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_ass
Dis_Frd_Sw_n1_termA_t=S0;
Dis_Frd_Sw_n1_termB_t=((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep));
Dis_Frd_Sw_n1_termC_t=((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrography(i,j)^2)))*((Area_hydrography(i,j)-Area_hydrography(i-1,j))/timestep));
Dis_Frd_Sw_n1_termD_t=((1/g)*(((Dis_last_time/Area_hydrography(i-1,j))-(Dis_assumed/Area_hydrography(i,j)))/(timestep)));
Dis_Frd_Sw_n1_termE_t=((((2*S0)/(3*(r(i,j)^2)))*(1-((Tw_hydrography(i,j)*(Dis_assumed^2))/(g*(Area_hydrography(i,j)^3))))));
Sf_t=(S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrogra
Diff1=Dis_found-Dis_assumed;
while Diff1>5*10^(-4) || Diff1<-5*10^(-4)
    if Diff1>5*10^(-4)
        Dis_assumed=Dis_assumed+(Diff1/5);
    end
    if Diff1<-5*10^(-4)
        Dis_assumed=Dis_assumed-(abs(Diff1/5));
    end
    if Dis_assumed<2*(10^(-4))
        Dis_assumed=2*(10^(-4));
    end
    Dis_found=Discharge(i,j)*(((S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis
Diff1=Dis_found-Dis_assumed;
Dis_Frd_Sw_n1_termA_t=S0;
Dis_Frd_Sw_n1_termB_t=((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep));
Dis_Frd_Sw_n1_termC_t=((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrography(i,j)^2)))*((Area_hydrography(i,j)-Area_hydrography(i-1,j))/timestep));
Dis_Frd_Sw_n1_termD_t=((1/g)*(((Dis_last_time/Area_hydrography(i-1,j))-(Dis_assumed/Area_hydrography(i,j)))/(timestep)));
Dis_Frd_Sw_n1_termE_t=((((2*S0)/(3*(r(i,j)^2)))*(1-((Tw_hydrography(i,j)*(Dis_assumed^2))/(g*(Area_hydrography(i,j)^3))))));
Sf_t=(S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrogra
end
Dis_Frd_hydrography2(i,j)=Dis_found;
Dis_Frd_TermA_hydrography2(i,j)=Dis_Frd_Sw_n1_termA_t;
Dis_Frd_TermB_hydrography2(i,j)=Dis_Frd_Sw_n1_termB_t;
Dis_Frd_TermC_hydrography2(i,j)=Dis_Frd_Sw_n1_termC_t;
Dis_Frd_TermD_hydrography2(i,j)=Dis_Frd_Sw_n1_termD_t;
Dis_Frd_TermE_hydrography2(i,j)=Dis_Frd_Sw_n1_termE_t;
Sf_hydrography2(i,j)=Sf_t;
A=Discharge(i,j)*(((S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_assumed
B=((S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydro
C=(S0+((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep))+((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrogr
D=((2*S0)/(3*(r(i,j)^2)))*(1-((Tw_hydrography(i,j)*(Dis_assumed^2))/(g*(Area_hydrography(i,j)^3)))));
E=((1/g)*(((Dis_last_time/Area_hydrography(i-1,j))-(Dis_assumed/Area_hydrography(i,j)))/(timestep)));
F=((1-(1/K(i,j)))*(Dis_assumed/(g*(Area_hydrography(i,j)^2)))*((Area_hydrography(i,j)-Area_hydrography(i-1,j))/timestep));
G=((Area_hydrography(i,j)/(K(i,j)*Dis_assumed))*((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep));
G1=(Area_hydrography(i,j)/(K(i,j)*Dis_assumed));
G2=((Depth_hydrography(i,j)-Depth_hydrography(i-1,j))/timestep);
if i==299 && j==12
    disp('asal')
end
if i==2 && j==6
    disp('asal')
end
if i==2 && j==5
    disp('asal')
end
if i==2 && j==4
    disp('asal')
end
if i==1734 && j==3
    disp('asal')
end
if u==0;
    Dis_Frd_hydrography2(1,j+1)=Dis_Frd_hydrography2(i,j);
    Dis_Frd_TermA_hydrography2(1,j+1)=Dis_Frd_TermA_hydrography2(i,j);
    Dis_Frd_TermB_hydrography2(1,j+1)=Dis_Frd_TermB_hydrography2(i,j);
    Dis_Frd_TermC_hydrography2(1,j+1)=Dis_Frd_TermC_hydrography2(i,j);
    Dis_Frd_TermD_hydrography2(1,j+1)=Dis_Frd_TermD_hydrography2(i,j);
    Dis_Frd_TermE_hydrography2(1,j+1)=Dis_Frd_TermE_hydrography2(i,j);
    Sf_hydrography2(1,j+1)=Sf_hydrography2(i,j);
    Dis_Frd_hydrography2(i,j)=0;
    Dis_Frd_TermA_hydrography2(i,j)=0;
    Dis_Frd_TermB_hydrography2(i,j)=0;
    Dis_Frd_TermC_hydrography2(i,j)=0;
    Dis_Frd_TermD_hydrography2(i,j)=0;
    Dis_Frd_TermE_hydrography2(i,j)=0;
    Sf_hydrography2(i,j)=0;
    break
end
end

```

```

end
% I dont need the first value to be zero. I'll make it equal to the next
% value at row two.
Dis_Frd_TermA_hydrography2(1,1)=Dis_Frd_TermA_hydrography2(2,1);
Dis_Frd_TermB_hydrography2(1,1)=Dis_Frd_TermB_hydrography2(2,1);
Dis_Frd_TermC_hydrography2(1,1)=Dis_Frd_TermC_hydrography2(2,1);
Dis_Frd_TermD_hydrography2(1,1)=Dis_Frd_TermD_hydrography2(2,1);
Dis_Frd_TermE_hydrography2(1,1)=Dis_Frd_TermE_hydrography2(2,1);
Sf_hydrography2(1,1)=Sf_hydrography2(2,1);

Dis_Frd_TermA_hydrography1=Dis_Frd_TermA_hydrography2;
Dis_Frd_TermB_hydrography1=Dis_Frd_TermB_hydrography2;
Dis_Frd_TermC_hydrography1=Dis_Frd_TermC_hydrography2;
Dis_Frd_TermD_hydrography1=Dis_Frd_TermD_hydrography2;
Dis_Frd_TermE_hydrography1=Dis_Frd_TermE_hydrography2;
Dis_Frd_hydrography1=Dis_Frd_hydrography2;
Sf_hydrography1=Sf_hydrography2;
for j=1:size_hydrograph(2)
    for i=1:size_hydrograph(1)
        if Dis_Bay1_hydrography1(i,j)==-1000;
            Dis_Frd_hydrography1(i,j)=-1000;
            Dis_Frd_TermA_hydrography1(i,j)=-1000;
            Dis_Frd_TermB_hydrography1(i,j)=-1000;
            Dis_Frd_TermC_hydrography1(i,j)=-1000;
            Dis_Frd_TermD_hydrography1(i,j)=-1000;
            Dis_Frd_TermE_hydrography1(i,j)=-1000;
            Sf_hydrography1(i,j)=-1000;
        end
    end
end

% Stream power
Stream_power_F1_hydro(1,1)=0;
for j=1:size_hydrograph(2)
    for i=2:size_hydrograph(1)-1
        if Dis_Frd_hydrography1(i,j)~=0 && Dis_Frd_hydrography1(i,j)~-1000
            % Stream_power_F1_hydro(i,j)=(((Dis_Frd_hydrography1(i,j)/Tw_hydrography(i,j))-(Dis_Frd_hydrography1(1,j)/Tw_hydrography(1,j)))+((Dis_Frd_hydrography1(i-1,j)/Tw_hydrography(i,j))-(Dis_Frd_hydrography1(i,j)/Tw_hydrography(i,j)))+(Baseflow_hydro(i,j)/Tw_hydrography(1,j)))+(Dis_Frd_hydrography1(i-1,j)/Tw_hydrography(i,j))-(Dis_Frd_hydrography1(i,j)/Tw_hydrography(i,j)));
            Stream_power_F1_hydro(i,j)=0;
        end
    end
    Stream_power_F1_hydro(size_hydrograph(1),j)=0;
    Stream_power_F1_total(j)=sum(Stream_power_F1_hydro(:,j));
    if j>1;
        Stream_power_F1_RL(j)=sum(Stream_power_F1_hydro(1:(Point_P_num(j-1)-((j-1)*size_hydrograph(1))),j));
        Stream_power_F1_FL(j)=sum(Stream_power_F1_hydro((Point_P_num(j-1)-((j-1)*size_hydrograph(1))):size_hydrograph(1),j));
    end
end
end

```

Not enough input arguments.

Error in Modified_Fread_Method (line 2)

Dis_Bay1_hydrography1=MFM_Inputs.Dis_Bay1_hydrography1;

The second method of finding unsteady formulation using data from two gauges

```

Dis_Bay1_hydrography2_loop_method2=Dis_Bay1_hydrography2;
Dis_Frd_hydrography2_method2(1,1)=Dis_Bay1_hydrography2(1,1);
for j=1:size_hydrograph(2)
    for i=2:size_hydrograph(1)
        u=1;
        if SWE_hydrography(i,j)==0;
            if j~=(size_hydrograph(2))
                if SWE_hydrography(i-1,j)~=0;
                    u=0;
                end
                TimeValues_hydrography_method2(i,j)=TimeValues_hydrography_method2(1,j+1);
                Depth_hydrography_method2(i,j)=Depth_hydrography_method2(1,j+1);
                Area_hydrography_method2(i,j)=Area_hydrography_method2(1,j+1);
                Area_yc_hydrography(i,j)=Area_yc_hydrography(1,j+1);
                WP_hydrography(i,j)=WP_hydrography(1,j+1);
                Dis_Bay1_hydrography1(i,j)=Dis_Bay1_hydrography1(1,j+1);
                Dis_Bay1_hydrography2_loop_method2(i,j)=Dis_Bay1_hydrography2_loop_method2(1,j+1);
            else
                break
            end
        end
        timestep=(TimeValues_hydrography_method2(i,j)-TimeValues_hydrography_method2(i-1,j))*24*60*60; % time step should be in seconds
        Discharge=Dis_Bay1_hydrography2_loop_method2;
        Dis_last_time=Dis_Frd_hydrography2_method2(i-1,j);
        Dis_assumed=Discharge(i,j);
        tf1 = strcmp(section,'GCrk10');
        if tf1==1;
            Dis_found=Discharge(i,j)*((1-((1/S0)*((Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j))-(Dis_assumed/(g*Area_hydrography_method2(i,j))))*(velocity
        else
            Dis_found=Discharge(i,j)*((1-((1/S0)*((Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j)))+(Dis_assumed/(g*Area_hydrography_method2(i,j))))*(velocity
        end
    end
end

```

```

Sf_t2=((Dis_found/Discharge(i,j))^2)*S0;
SfoverS0sqrt_t=Dis_found/Discharge(i,j);
Term1_t=S0;
Term2_t=-(Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j));
Term3_t=-(Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity_BU_hydrography(i,j)-(Dis_assumed/Area_hydrography_method2(i,j)))/distace_between_2_gau
if tf1==1;
    Term3_t=((Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity_BU_hydrography(i,j)-(Dis_assumed/Area_hydrography_method2(i,j)))/distace_between_2_
end
Term4_t=-(1/g)*((Dis_assumed/Area_hydrography_method2(i,j))-(Dis_last_time/Area_hydrography_method2(i-1,j)))/timestep);
Diff1=Dis_found-Dis_assumed;
while Diff1>50*10^(-4) || Diff1<-50*10^(-4)
    if Diff1>50*10^(-4)
        Dis_assumed=Dis_assumed+(Diff1/5);
    end
    if Diff1<-50*10^(-4)
        Dis_assumed=Dis_assumed-(abs(Diff1/5));
    end
    if tf1==1;
        Dis_found=Discharge(i,j)*((1-((1/S0)*(Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j))-(Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity
    else
        Dis_found=Discharge(i,j)*((1-((1/S0)*(Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j)))+(Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity_
    end
    Diff1=Dis_found-Dis_assumed;
Sf_t2=((Dis_found/Discharge(i,j))^2)*S0;
SfoverS0sqrt_t=Dis_found/Discharge(i,j);
Term1_t=S0;
Term2_t=-(Area_yc_hydrography(i,j)/Area_hydrography_method2(i,j));
Term3_t=-(Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity_BU_hydrography(i,j)-(Dis_assumed/Area_hydrography_method2(i,j)))/distace_between_2_gauges));
if tf1==1;
    Term3_t=((Dis_assumed/(g*Area_hydrography_method2(i,j)))*((velocity_BU_hydrography(i,j)-(Dis_assumed/Area_hydrography_method2(i,j)))/distace_between_2_gauges));
end
Term4_t=-(1/g)*((Dis_assumed/Area_hydrography_method2(i,j))-(Dis_last_time/Area_hydrography_method2(i-1,j)))/timestep);
end
    Dis_Frd_hydrography2_method2(i,j)=Dis_found;
    Sf_hydrography2_method2(i,j)=Sf_t2;
    SfoverS0sqrt_hydrog(i,j)=SfoverS0sqrt_t;
    Term1_hydro(i,j)=Term1_t;
    Term2_hydro(i,j)=Term2_t;
    Term3_hydro(i,j)=Term3_t;
    Term4_hydro(i,j)=Term4_t;

if i==14530 && j==9
    disp('asal')
end
if i==2
    disp('asal')
end
if i==9741 && j==5
    disp('asal')
end

if u==0;
    Dis_Frd_hydrography2_method2(1,j+1)=Dis_Frd_hydrography2_method2(i,j);
    Sf_hydrography2_method2(1,j+1)=Sf_hydrography2_method2(i,j);
    SfoverS0sqrt_hydrog(1,j+1)=SfoverS0sqrt_hydrog(i,j);
    Term1_hydro(1,j+1)=Term1_hydro(i,j);
%    Term2_hydro(1,j+1)=-(Area_yc_hydrography(1,j+1)/Area_hydrography_method2(i,j));
    Term2_hydro(1,j+1)=Term2_hydro(i,j);
    Term3_hydro(1,j+1)=Term3_hydro(i,j);
    Term4_hydro(1,j+1)=Term4_hydro(i,j);
    Dis_Frd_hydrography2_method2(1,j)=0;
    Sf_hydrography2_method2(i,j)=0;
    SfoverS0sqrt_hydrog(i,j)=0;
    Term1_hydro(i,j)=0;
    Term2_hydro(i,j)=0;
    Term3_hydro(i,j)=0;
    Term4_hydro(i,j)=0;
    break
end
if i==2
    disp('asal')
end

end

Sf_hydrography2_method2(1,1)=Sf_hydrography2_method2(2,1);
SfoverS0sqrt_hydrog(1,1)=SfoverS0sqrt_hydrog(2,1);
Term1_hydro(1,1)=Term1_hydro(2,1);
Term2_hydro(1,1)=Term2_hydro(2,1);
Term3_hydro(1,1)=Term3_hydro(2,1);
Term4_hydro(1,1)=Term4_hydro(2,1);

% hydrography values with no base flow:
Sf_hydrography1_method2=Sf_hydrography2_method2;
Dis_Frd_hydrography1_method2=Dis_Frd_hydrography2_method2;
Term1_hydro_method2=Term1_hydro;
Term2_hydro_method2=Term2_hydro;
Term3_hydro_method2=Term3_hydro;

```

```

Term4_hydro_method2=Term4_hydro;
for j=1:size_hydrograph(2)
    for i=1:size_hydrograph(1)
        if Dis_Bay1_hydrography1(i,j)==-1000;
            Sf_hydrography1_method2(i,j)==-1000;
            Dis_Frd_hydrography1_method2(i,j)==-1000;
            Term1_hydro_method2(i,j)==-1000;
            Term2_hydro_method2(i,j)==-1000;
            Term3_hydro_method2(i,j)==-1000;
            Term4_hydro_method2(i,j)==-1000;
        end
    end
end

% Stream power
Stream_power_F2_hydro(1,1)=0;
for j=1:size_hydrograph(2)
    for i=2:size_hydrograph(1)-1
        if Dis_Frd_hydrography1_method2(i,j)~=0 && Dis_Frd_hydrography1_method2(i,j)~-1000
%           Stream_power_F2_hydro(i,j)=(((Dis_Frd_hydrography1_method2(i,j)/Tw_hydrography(i,j))-(Dis_Frd_hydrography1_method2(i,j)/Tw_hydrography(1,j)))+(Dis_Frd
            Stream_power_F2_hydro(i,j)=(((Dis_Frd_hydrography1_method2(i,j)/Tw_hydrography(i,j))-(Baseflow_hydro(i,j)/Tw_hydrography(1,j)))+(Dis_Frd_hydrography1_metho
        else
            Stream_power_F2_hydro(i,j)=0;
        end
    end
    Stream_power_F2_hydro(size_hydrograph(1),j)=0;
    Stream_power_F2_total(j)=sum(Stream_power_F2_hydro(:,j));
    if j>1;
        Stream_power_F2_RL(j)=sum(Stream_power_F2_hydro(1:(Point_P_num(j-1)-((j-1)*size_hydrograph(1))),j));
        Stream_power_F2_FL(j)=sum(Stream_power_F2_hydro((Point_P_num(j-1)-((j-1)*size_hydrograph(1))):size_hydrograph(1),j));
    end
end
end

```

Now we make the output files back to a vector format:

```

size_hydrograph=size(Dis_Bay1_hydrography1);
Dis_Frd1=[];
Dis_Frd_TermA1=[];
Dis_Frd_TermB1=[];
Dis_Frd_TermC1=[];
Dis_Frd_TermD1=[];
Dis_Frd_TermE1=[];
Dis_Frd2=[];
Dis_Frd_TermA2=[];
Dis_Frd_TermB2=[];
Dis_Frd_TermC2=[];
Dis_Frd_TermD2=[];
Dis_Frd_TermE2=[];
Sf1=[];
Sf2=[];
Depth1=[];
K_mat=[];
r_mat=[];
Depth12=[];
Sf2_method2=[];
Dis_Frd2_method2=[];
Sf1_method2=[];
Dis_Frd1_method2=[];
SfoverS0sqrt=[];
Term1_method2=[];
Term2_method2=[];
Term3_method2=[];
Term4_method2=[];
Term1_1_method2=[];
Term2_1_method2=[];
Term3_1_method2=[];
Term4_1_method2=[];
for i=1:size_hydrograph(2)
    Sf1=[Sf1;Sf_hydrography1(Sf_hydrography1(:,i)~=0,i)];
    Sf2=[Sf2;Sf_hydrography2(Sf_hydrography2(:,i)~=0,i)];
    Dis_Frd_TermA1=[Dis_Frd_TermA1;Dis_Frd_TermA_hydrography1(Dis_Frd_TermA_hydrography1(:,i)~=0,i)];
    Dis_Frd_TermA2=[Dis_Frd_TermA2;Dis_Frd_TermA_hydrography2(Dis_Frd_TermA_hydrography2(:,i)~=0,i)];
    Dis_Frd_TermB1=[Dis_Frd_TermB1;Dis_Frd_TermB_hydrography1(Dis_Frd_TermB_hydrography1(:,i)~=0,i)];
    Dis_Frd_TermB2=[Dis_Frd_TermB2;Dis_Frd_TermB_hydrography2(Dis_Frd_TermB_hydrography2(:,i)~=0,i)];
    Dis_Frd_TermC1=[Dis_Frd_TermC1;Dis_Frd_TermC_hydrography1(Dis_Frd_TermC_hydrography1(:,i)~=0,i)];
    Dis_Frd_TermC2=[Dis_Frd_TermC2;Dis_Frd_TermC_hydrography2(Dis_Frd_TermC_hydrography2(:,i)~=0,i)];
    Dis_Frd_TermD1=[Dis_Frd_TermD1;Dis_Frd_TermD_hydrography1(Dis_Frd_TermD_hydrography1(:,i)~=0,i)];
    Dis_Frd_TermD2=[Dis_Frd_TermD2;Dis_Frd_TermD_hydrography2(Dis_Frd_TermD_hydrography2(:,i)~=0,i)];
    Dis_Frd_TermE1=[Dis_Frd_TermE1;Dis_Frd_TermE_hydrography1(Dis_Frd_TermE_hydrography1(:,i)~=0,i)];
    Dis_Frd_TermE2=[Dis_Frd_TermE2;Dis_Frd_TermE_hydrography2(Dis_Frd_TermE_hydrography2(:,i)~=0,i)];
    Dis_Frd1=[Dis_Frd1;Dis_Frd_hydrography1(Dis_Frd_hydrography1(:,i)~=0,i)];
    Dis_Frd2=[Dis_Frd2;Dis_Frd_hydrography2(Dis_Frd_hydrography2(:,i)~=0,i)];
    Depth1=[Depth1;Depth_hydrography1(Depth_hydrography1(:,i)~=0,i)];
    Depth12=[Depth12;Depth_hydrography(Depth_hydrography(:,i)~=0,i)];
    K_mat=[K_mat;K(K(:,i)~=0,i)];
    r_mat=[r_mat;r(r(:,i)~=0,i)];
    Sf2_method2=[Sf2_method2;Sf_hydrography2_method2(Sf_hydrography2_method2(:,i)~=0,i)];
    Dis_Frd2_method2=[Dis_Frd2_method2;Dis_Frd_hydrography2_method2(Dis_Frd_hydrography2_method2(:,i)~=0,i)];
    Sf1_method2=[Sf1_method2;Sf_hydrography1_method2(Sf_hydrography1_method2(:,i)~=0,i)];
    Dis_Frd1_method2=[Dis_Frd1_method2;Dis_Frd_hydrography1_method2(Dis_Frd_hydrography1_method2(:,i)~=0,i)];

```

```

SfoverS0sqr=[SfoverS0sqr;SfoverS0sqr_hydrog(SfoverS0sqr_hydrog(:,i)~=0,i)];
Term1_method2=[Term1_method2;Term1_hydro(Term1_hydro(:,i)~=0,i)];
Term2_method2=[Term2_method2;Term2_hydro(Term2_hydro(:,i)~=0,i)];
Term3_method2=[Term3_method2;Term3_hydro(Term3_hydro(:,i)~=0,i)];
Term4_method2=[Term4_method2;Term4_hydro(Term4_hydro(:,i)~=0,i)];
Term1_1_method2=[Term1_1_method2;Term1_hydro_method2(Term1_hydro_method2(:,i)~=0,i)];
Term2_1_method2=[Term2_1_method2;Term2_hydro_method2(Term2_hydro_method2(:,i)~=0,i)];
Term3_1_method2=[Term3_1_method2;Term3_hydro_method2(Term3_hydro_method2(:,i)~=0,i)];
Term4_1_method2=[Term4_1_method2;Term4_hydro_method2(Term4_hydro_method2(:,i)~=0,i)];
% if length(Term2_1_method2)>45000
% disp('asal')
% end
end
Depth12(length(Depth12))=[];
K_mat=[K_mat(1);K_mat];
r_mat=[r_mat(1);r_mat];

```

Output:

```

Modified_Fread_Method_outputs.Dis_Frd_hydrography2=Dis_Frd_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_TermA_hydrography2=Dis_Frd_TermA_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_TermB_hydrography2=Dis_Frd_TermB_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_TermC_hydrography2=Dis_Frd_TermC_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_TermD_hydrography2=Dis_Frd_TermD_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_TermE_hydrography2=Dis_Frd_TermE_hydrography2;
Modified_Fread_Method_outputs.Sf_hydrography2=Sf_hydrography2;
Modified_Fread_Method_outputs.Dis_Frd_hydrography1=Dis_Frd_hydrography1;
Modified_Fread_Method_outputs.Dis_Frd_TermA_hydrography1=Dis_Frd_TermA_hydrography1;
Modified_Fread_Method_outputs.Dis_Frd_TermB_hydrography1=Dis_Frd_TermB_hydrography1;
Modified_Fread_Method_outputs.Dis_Frd_TermC_hydrography1=Dis_Frd_TermC_hydrography1;
Modified_Fread_Method_outputs.Dis_Frd_TermD_hydrography1=Dis_Frd_TermD_hydrography1;
Modified_Fread_Method_outputs.Dis_Frd_TermE_hydrography1=Dis_Frd_TermE_hydrography1;
Modified_Fread_Method_outputs.Sf_hydrography1=Sf_hydrography1;
Modified_Fread_Method_outputs.K_mat=K_mat;
Modified_Fread_Method_outputs.r_mat=r_mat;
Modified_Fread_Method_outputs.Sf1=Sf1;
Modified_Fread_Method_outputs.Sf2=Sf2;
Modified_Fread_Method_outputs.Dis_Frd_TermA1=Dis_Frd_TermA1;
Modified_Fread_Method_outputs.Dis_Frd_TermA2=Dis_Frd_TermA2;
Modified_Fread_Method_outputs.Dis_Frd_TermB1=Dis_Frd_TermB1;
Modified_Fread_Method_outputs.Dis_Frd_TermB2=Dis_Frd_TermB2;
Modified_Fread_Method_outputs.Dis_Frd_TermC1=Dis_Frd_TermC1;
Modified_Fread_Method_outputs.Dis_Frd_TermC2=Dis_Frd_TermC2;
Modified_Fread_Method_outputs.Dis_Frd_TermD1=Dis_Frd_TermD1;
Modified_Fread_Method_outputs.Dis_Frd_TermD2=Dis_Frd_TermD2;
Modified_Fread_Method_outputs.Dis_Frd_TermE1=Dis_Frd_TermE1;
Modified_Fread_Method_outputs.Dis_Frd_TermE2=Dis_Frd_TermE2;
Modified_Fread_Method_outputs.Dis_Frd1=Dis_Frd1;
Modified_Fread_Method_outputs.Dis_Frd2=Dis_Frd2;
Modified_Fread_Method_outputs.Depth1=Depth1;
Modified_Fread_Method_outputs.Sf2_2gauges=Sf2_method2;
Modified_Fread_Method_outputs.Dis_Frd2_2gauges=Dis_Frd2_method2;
Modified_Fread_Method_outputs.Sf1_2gauges=Sf1_method2;
Modified_Fread_Method_outputs.Dis_Frd1_2gauges=Dis_Frd1_method2;
Modified_Fread_Method_outputs.SfoverS0sqr=SfoverS0sqr;
Modified_Fread_Method_outputs.Term1_2gauges=Term1_method2;
Modified_Fread_Method_outputs.Term2_2gauges=Term2_method2;
Modified_Fread_Method_outputs.Term3_2gauges=Term3_method2;
Modified_Fread_Method_outputs.Term4_2gauges=Term4_method2;
Modified_Fread_Method_outputs.Dis_Frd_hydrography1_method2=Dis_Frd_hydrography1_method2;
Modified_Fread_Method_outputs.Term1_1_method2=Term1_1_method2;
Modified_Fread_Method_outputs.Term2_1_method2=Term2_1_method2;
Modified_Fread_Method_outputs.Term3_1_method2=Term3_1_method2;
Modified_Fread_Method_outputs.Term4_1_method2=Term4_1_method2;
Modified_Fread_Method_outputs.Term1_hydro=Term1_hydro;
Modified_Fread_Method_outputs.Term2_hydro=Term2_hydro;
Modified_Fread_Method_outputs.Term3_hydro=Term3_hydro;
Modified_Fread_Method_outputs.Term4_hydro=Term4_hydro;
Modified_Fread_Method_outputs.Term1_hydro_method2=Term1_hydro_method2;
Modified_Fread_Method_outputs.Term2_hydro_method2=Term2_hydro_method2;
Modified_Fread_Method_outputs.Term3_hydro_method2=Term3_hydro_method2;
Modified_Fread_Method_outputs.Term4_hydro_method2=Term4_hydro_method2;
Modified_Fread_Method_outputs.Sf_hydrography1_method2=Sf_hydrography1_method2;
Modified_Fread_Method_outputs.Stream_power_F2_hydro=Stream_power_F2_hydro;
Modified_Fread_Method_outputs.Stream_power_F2_total=Stream_power_F2_total;
Modified_Fread_Method_outputs.Stream_power_F1_hydro=Stream_power_F1_hydro;
Modified_Fread_Method_outputs.Stream_power_F1_total=Stream_power_F1_total;
Modified_Fread_Method_outputs.Stream_power_F2_FL=Stream_power_F2_FL;
Modified_Fread_Method_outputs.Stream_power_F2_RL=Stream_power_F2_RL;
Modified_Fread_Method_outputs.Stream_power_F1_FL=Stream_power_F1_FL;
Modified_Fread_Method_outputs.Stream_power_F1_RL=Stream_power_F1_RL;

```

end