

ABSTRACT

Title of Dissertation: BEYOND PEAK RATE FACTOR 484: USING RADAR RAINFALL, GAUGED STREAMFLOW, AND DISTRIBUTED WATERSHED MODELING TO INVESTIGATE PARAMETERS OF THE NATURAL RESOURCES CONSERVATION SERVICE CURVILINEAR UNIT HYDROGRAPH

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Accurate estimation of runoff and peak discharge is crucial in hydrology for engineering design and flood management. The Natural Resources Conservation Service's (NRCS) Unit Hydrograph (UH) is a widely used model to predict the runoff response of an ungauged watershed to a precipitation event. The NRCS UH model makes use of a Peak Rate Factor (PRF) to quantify the peak discharge. The standard value of PRF is 484; however, PRF can be adjusted as a user input variable in NRCS tools such as WinTR-20 software. Little guidance is available to appropriately estimate PRF for specific regions and evaluate its overall usefulness in the runoff and peak discharge estimation. Time of concentration (t_c) is another input variable in the NRCS UH model; inconsistent definitions of t_c and diverse methods of calculating it, contribute to uncertainty in hydrologic estimates and predictions. The NRCS UH approach assumes that the watershed's temporal runoff response to each

increment of precipitation is identical in shape and proportional to precipitation excess in that increment of time. The UH, PRF, and t_c are often assumed to be time-invariant properties of a watershed.

This dissertation sought to improve the knowledge and understanding of PRF and t_c . First, it evaluated if a unique UH and t_c exist for a given watershed from various storm events. It then assessed whether variations in PRFs can be explained by watershed predictor variables and if PRFs in neighboring watersheds followed a local trend. This phase of study employed a gamma function representation of the NRCS UH, with two parameters: time to peak (t_p) and shape (m). Precipitation inputs were watershed-averaged time series of NEXRAD level III data, and streamflow data were obtained from the United States Geological Survey (USGS) National Water Information System (NWIS). The UHs were derived from a constrained optimization approach, and PRF and t_c were estimated for each event. Subsequently, a fully distributed model was created to provide insight on PRF and t_c , and investigate the impact of detailed soil profiles on runoff and peak discharge. Finally, a fully distributed model was applied to simple, synthetic watersheds to investigate the impact of selected watershed parameters on PRF, time to peak, peak discharge and overall shape of the UH. To the best of the author's knowledge, this study is the first attempt to generate UHs from a simple distributed model and estimate associated PRFs.

The findings suggest that there is no unique UH and t_c for a given watershed, and UH shape and parameters change for every event in a given watershed. Additionally, the variations in PRFs cannot be explained by variations in selected watershed predictor variables. The distributed model results provided insights about the application of detailed soil profiles in runoff and peak discharge estimation. The findings also suggest that, except

for Manning's roughness, selected watershed characteristics cannot be used to estimate PRF in a synthetic V-shaped watershed. These findings suggest that the application of PRF to estimate peak discharge should be used with caution due to the inherent uncertainties and lack of physical meaning of the parameter.

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HYDROGRAPH

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Foreword

Materials presented in Chapter 3, Chapter 4 and Chapter 6 of this dissertation are based on a technical report (Chapter 3) and two journal papers (Chapter 4 and Chapter 6). The technical report was published in August 2021 on Maryland Department of Transportation State Highway Administration (MDOT SHA) domain. The journal papers are expected to be published as peer-reviewed articles in the next few months following the dissertation defense. The dissertation presented here is conducted in its entirety by Mani Shehni Karam Zadeh. Please see the list of publications below.

Shehni Karam Zadeh, M. and Brubaker, K. (2024) Peak Rate Factor Estimation in Response of Selected Maryland and Delaware Watersheds to Hurricane Sandy. ASCE Journal of Hydrologic Engineering (Submitted).

Shehni Karam Zadeh, M. and Brubaker, K. (2024) Investigating Unit Hydrographs with a Simple Distributed Model. ASCE Journal of Hydrologic Engineering (In preparation).

This study is partially funded by Maryland Department of Transportation State Highway Administration (MDOT SHA) as well as through funding from the Maryland Water Resources Research Center (MWRRC).

Dedication

To my lifetime partner and soulmate, Forough.

To my loving parents, Manizheh and Rostam.

Acknowledgments

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List of Abbreviations

ADE	Alternating Direction Explicit
ADEPC	Alternating Direction Explicit Predictor-Corrector
AEP	Annual Exceedance Probability
AMS	American Meteorological Society
AWBM	Australian Water Balance Model
ARS	Agricultural Research Service
CCP	California Culvert Practice
CEE	Civil and Environmental Engineering
CN	Curve Number
COV	Coefficient of Variation
CPS	Capillary Pressure-Saturation
CS	Channel Slope
CSV	Comma-separated Values
DA	Drainage Area
DEM	Digital Elevation Model
DPA	Digital Precipitation Array
DPR	Digital Precipitation Rate
DRO	Direct Runoff
DUH	Dimensionless Unit Hydrograph
ESRI	Environmental Systems Research Institute
FAA	Federal Aviation Administration
FOR	Percent Forest
GA	Green and Ampt
GAR	Green and Ampt with Redistribution
GIS	Geographic Information System
GridMET	Gridded Surface Meteorological
GSSHA	Gridded Surface Subsurface Hydrologic Analysis
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System

HPCC	High-Performance Computing Clusters
HRU	Hydrology Response Unit
IA	Percent Impervious
LS	Land Slope
MATLAB	Matrix Laboratory
MDOT SHA	Maryland Department of Transportation State Highway Administration
MS	Microsoft
MSP	Maryland State Plane
MWRRC	Maryland Water Resources Research Center
NCEI	National Centers for Environmental Information
NEH	National Engineering Handbook
NEXRAD	Next Generation Weather Radar
NHD	National Hydrography Dataset
NLDAS-2	North American Land Data Assimilation System
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NWIS	National Water Information System
PRF	Peak Rate Factor
PEX	Precipitation Excess
RMSE	Root Mean Square Error
RRL	Rainfall Runoff Library
SCS	Soil Conservation Service
SSE	Sum of Squares Error
SSR	Sum of Squares Regression
SSURGO	Soil Survey Geographic Database
ST	Percent Storage
SWAT	Soil and Water Assessment Tool
SWMM	Storm Water Management Model
TxDOT	Texas Department of Transportation
UCAR	University Corporation for Atmospheric Research
UH	Unit Hydrograph

UHPF	Unit Hydrograph Peaking Factor
UMD	University of Maryland
USDA	United States Department of Agriculture
USGS	United States Geological Survey
WCT	Weather and Climate Toolkit
WSR-88D	Weather Surveillance Radar 88 Doppler

Chapter 1: Introduction

Hydrological models are essential tools in understanding and predicting watershed behavior and floods. Among them, the unit hydrograph (UH) approach is widely used in estimating peak discharges and event hydrographs from ungauged watersheds. Specifically, the Natural Resources Conservation Service (NRCS) curvilinear dimensionless unit hydrograph (DUH) is assumed in numerous watershed analysis tools and applications. Scaling the DUH to a watershed-specific UH requires an estimate of the time to peak (t_p) and a scale factor, known as the peak rate factor (PRF).

Standard practice estimates t_p as 2/3 the watershed time of concentration, t_c , variously defined among researchers but usually assumed to represent the travel time of water from the point on the watershed boundary that is hydraulically most distant from the watershed outlet. In practice, the t_c is usually estimated using a velocity method, summing travel times through assumed flow segments of the watershed. In the NRCS DUH model, t_c is one of the most debatable components. Numerous studies have proposed multiple approaches and equations to estimate t_c . Both the PRF and t_c are generally assumed to be properties of a watershed, not dependent upon rainfall rate.

For the PRF, a value of 484 (in US Customary System units) is generally applied, with a value of 284 recommended for the DelMarVa Peninsula; however, recently, the NRCS has

reported that the PRF is an adjustable parameter of the DUH model [[Natural Resources Conservation Service \(NRCS\), 2007](#)].

The original aim of this research project was to provide guidance for selection of appropriate PRF and t_c for watershed modeling to improve runoff estimation. It was anticipated that PRF and t_c could be selected and/or computed as functions of readily observable watershed characteristics. In the course of the research, the findings did not support that expectation, but did provide in-depth exploration of these unit hydrograph properties.

1.1 Research Objectives

This research aims to explore key aspects of NRCS UH modeling and its associated parameters. The specific objectives of this research are:

1. To assess if a unique UH and time of concentration exist for a watershed from different storm events.
2. To test if there is any connection between UH shape and storm event size (i.e., depth of the runoff).
3. To investigate if any local/regional trend for predicted PRFs can be observed in selected watersheds.
4. To determine if variations in predicted PRFs can be explained by selected watershed predictor variables.

5. To explore the effect of detailed soil profiles on peak discharge using a fully distributed hydrologic model.
6. To explore the impact of selected watershed characteristics on PRF, peak discharge, time to peak, and overall hydrograph shape in a simple, open-book watershed.

1.2 Research Questions

After a review of the literature and background on the NRCS DUH, t_c and PRF (Chapter 2), the chapters of this dissertation address these objectives through the research questions listed below.

1. *Chapter 3: Initial Study of Maryland Watersheds.* 1- Does a unique UH exist for a watershed? Does the UH vary among different storm events? 2- Is there any connection between PRF and storm event size? 3- Is time of concentration an invariant property of watershed? 4- Is t_c the best watershed time characteristic to estimate the t_p required by the NRCS UH method?
2. *Chapter 4: Estimation of the NRCS Unit Hydrograph Peak Rate Factor for Maryland Watersheds in Response to Hurricane Sandy.* 1- How different are the PRFs in selected watersheds for a major rainfall-runoff event? 2- Can a spatial trend be established among PRFs in selected watersheds? 3- Can the variation in the PRFs be explained by the selected watershed characteristics?
3. *Chapter 5: Distributed Model of Jones Falls Watershed.* 1- Can a detailed, distributed hydrologic model using distributed rainfall data provide insight on the assumptions

of the UH method? 2- What is the role of detailed soil data for estimating runoff using a distributed model? 3- Do they significantly affect the runoff simulation?

4. *Chapter 6: Investigating Unit Hydrographs with a Simple Distributed Model.* 1- Which selected watershed parameters (e.g., surface roughness, side slope, and longitudinal slope) using a distributed model significantly change the shape of unit hydrograph, t_p , peak discharge, and PRF? 2- How do the rainfall duration and intensity matching 1 unit of rainfall (1 cm or 1 inch) impact the time to peak, peak discharge and PRF in the UHs generated by a distributed model? 3- Can a relationship be established between selected watershed parameters and resulting PRFs from a simple distributed model?

Chapter 2: Literature Review and Background

This chapter provides an overview of the fundamental concepts of unit hydrographs (UHs), time of concentration (t_c), and peak rate factor (PRF) as discussed in various literature sources and scholarly works. It also covers the analysis of the NRCS curvilinear unit hydrograph. It explores the behavior of t_c and PRF in diverse watersheds across different regions and continental scales, drawing from existing literature analyses. Additionally, the chapter reviews various techniques employed to enhance the accuracy of t_c and PRF computation, considering their application within the context of the NRCS curvilinear unit hydrograph and other relevant analyses.

2.1 Hydrographs and Unit Hydrographs

In this section, basic concepts and definitions pertaining to hydrograph, unit hydrograph (UH), and their characteristics are discussed.

2.1.1 Definitions and Descriptors

A hydrograph delineates the temporal variation in surface water flow discharge subsequent to a rainfall event. [Linsley et al. \[1975\]](#) defined hydrograph as a plot representing discharge versus time. Typically, these hydrographs are characterized by single-peak curves,

with the peak discharge value providing insight into past flood occurrences [Sen, 2018]. Horst and Gurriell [2019] defined hydrograph as a representative of watershed response, in terms of volumetric flow rate, to a certain rainfall event as a time function. The hydrographs are generally used by hydrologists and engineers to assist in designing hydraulics structures (e.g., dams, bridges ...). For design purpose, one of the most practical ways to estimate the design hydrograph is to convolve UHs of watersheds with associated rainfall events (i.e., 20-year storm).

Hydrographs can be classified into different types. NRCS classified four types of hydrographs [Natural Resources Conservation Service (NRCS), 2007]:

- Natural Hydrograph: This type of hydrograph is derived directly from the data gathered via the streamflow records of a monitored or gauged stream.
- Synthetic Hydrograph: This is developed by applying watershed variables and storm features to model or replicate a natural hydrograph.
- Unit Hydrograph (UH): This represents the flow or discharge hydrograph that results from one unit (one inch or one centimeter) of direct runoff, uniformly distributed across a specific watershed, caused by a rainfall event of a predetermined duration.
- Dimensionless Unit Hydrograph (DUH): This is a type of hydrograph constructed with the purpose of representing an array of UHs. This is achieved by plotting the hydrograph using the ratio of two fundamental units - time to peak and peak rate. Occasionally referred to as an index hydrograph, the DUH offers a standardized approach to hydrograph representation and analysis.

In the process of hydrograph analysis, it is crucial to accurately define a set of key parameters pertaining to the analysis. These parameters consist of, but are not limited to: peak discharge (q_p), time to peak (t_p), lag time (t_l), time of concentration (t_c), rainfall excess duration (ΔD), and recession time (t_r). Based on [Natural Resources Conservation Service (NRCS), 2007], peak discharge (q_p) refers to the maximum rate of runoff, quantified as volume per unit time (often measured in cubic feet per second, cfs). This rate corresponds to a specified rainfall event over a defined drainage area. The time to peak (t_p) in hydrology is the duration between the start of a precipitation event and the moment at which the highest flow rate, or peak discharge, is observed in the resulting runoff. Another concept in hydrograph theory, is the lag time (t_l) defined as the time interval between the center of excess rainfall mass and the rate of peak runoff [Natural Resources Conservation Service (NRCS), 2010]. In the context of hydrograph analysis, the term time of concentration (t_c) refers to the duration from the cessation of excess precipitation to the inflection point on the recession limb of the DUH, which is the beginning of the recession curve [Natural Resources Conservation Service (NRCS), 2010]. The hydrograph excess rainfall duration (ΔD) is the duration of the rainfall excess mass curve. [Natural Resources Conservation Service (NRCS), 2007] stated that the storm duration is the actual interval of rainfall excess. Finally, the recession time (t_r) is the time from the peak discharge to the end of the hydrograph. The aforementioned parameters are all illustrated and can be examined in detail in Figure 2.1.

The concept of a hydrograph, time to peak, time of concentration, and peak discharge along with their diverse properties and features, are crucial for successful flood prediction applications. This includes several fundamental types of hydrographs such as natural,

unit, dimensionless, instantaneous, and synthetic discussed earlier. Each type has unique relationships and significance in the computation of peak flood discharge [Sen, 2018].

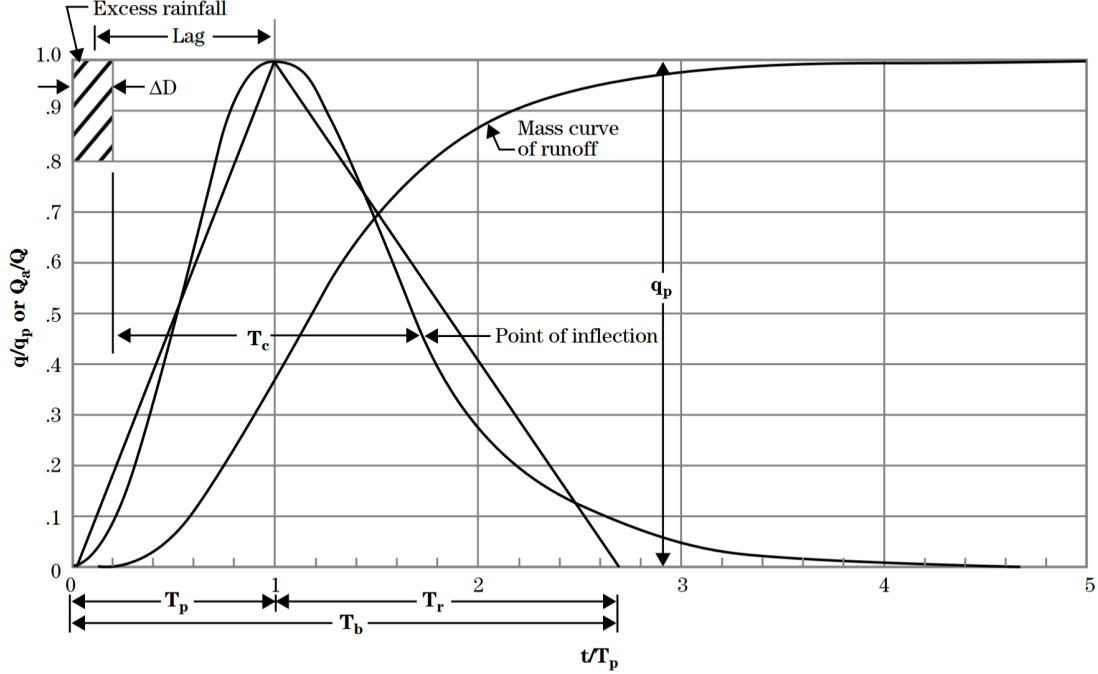


Figure 2.1: Representation of a NRCS DUH: This figure displays NRCS DUH, which illustrates the catchment's response, in terms of runoff, to a unit pulse of excess rainfall. Important components including the rising limb, peak flow, and recession limb are indicated, demonstrating the temporal distribution of flow resulting from a unit of excess rainfall. Figure courtesy of the Natural Resources Conservation Service [Natural Resources Conservation Service (NRCS), 2007].

Given that hydrographs can be generated using multiple approaches, a variety of methods for calculating peak flood discharge are discussed in the literature, starting from the relatively simple rational method to more advanced techniques. These include, but are not limited to, the Snyder method, the Soil Conservation Service (SCS) approach which is the same as NRCS approach, and geomorphological techniques.

2.1.2 Unit Hydrograph Theory

The concept of the UH was initially introduced by [Sherman, 1932, 1940]. A theoretical foundation for the UH was initially absent. Dooge [1959] proposed a general theory for the UH. He discussed that the UH model to estimate stream flow is one of the most powerful tools in applied hydrology. This novel theory contributed significantly to mitigating some of the subjective aspects inherent in the analysis of UHs. Singh [2005] performed a follow-up study on UH theory and stated that UH theory remains a highly effective method, applicable to numerous challenges in the fields of hydrology and environmental engineering. In practical applications, this theory continues to be regularly employed. Despite the fact that the majority of hydrologic and environmental systems exhibit nonlinearity, their linear elements often take precedence. This predominance of linear elements largely accounts for the high effectiveness of this theory in hydrology.

UH is a specific type of direct runoff hydrograph that is generated by a uniform, constant intensity rainfall of one unit (one inch or one centimeter) that is evenly distributed across the entire catchment area. The analytical or synthetic procedures for all UHs are based on two fundamental principles [Dooge, 1959]:

- Invariance - The hydrograph, illustrating the surface runoff from a catchment as a response to a specified pattern of rainfall excess, defined as rainfall reduced by infiltration and related losses, exhibits a state of invariability. That means, if an identical unit of excess rainfall were to occur at two different points in time, the resulting UHs would be identical. This assumes that the characteristics of the catchment remain constant.

- Superposition - The hydrograph, derived from a predetermined pattern of rainfall excess, can be constructed through the superposition of UHs corresponding to individual quantities of rainfall excess occurring during each unit time period. This process encapsulates the principle of proportionality, whereby the ordinates of the hydrograph maintain a proportional relationship with the volume of rainfall excess.

The UH concept is rooted in the linear systems theory and adheres to the principles of superposition and proportionality. To illustrate, if a one-inch rainfall excess yields a direct runoff peak of 100 cubic feet per second (cfs), then a two-inch excess will result in a direct runoff peak of 200 cfs, which is twice the initial amount. Similarly, if a one-inch rainfall is succeeded by a two-inch rainfall, the resulting hydrographs from both rainfall events are simply summed, after taking into account the appropriate time shifts.

2.1.3 Unit Hydrograph Studies

The UH serves a crucial function in estimating the direct runoff hydrograph that results from a specified excess rainfall hyetograph. To utilize the UH as an instrument for forecasting the direct runoff hydrograph, it is initially necessary to derive a UH specific to a given watershed [[Merwade and Ruddell, n.d.](#)].

The main application of the UH is on the runoff calculation on ungauged watersheds. In this case, they are referred to as synthetic UHs. Methodological advancement presented by [Sherman \[1932, 1940\]](#) in UH theory, postulated that the discharge (volume of water flowing past a certain point in a given period of time) observed at any particular moment is directly proportional to the volume of runoff. Additionally, it assumed that the temporal

elements influencing the shape of the hydrograph remain consistent. UH models have been developed by [Clark \[1945\]](#), [Snyder \[1938\]](#), and [Mockus \[1957\]](#), and their application spans various hydrological modeling contexts [[Sheridan et al., 2002](#)]. A UH can be developed via analysis of the rainfall-runoff events which are accessible through historical records or through the analysis of different catchment characteristics applied in combination with a synthetic UH model.

Probability distribution functions (PDFs) are used as functional forms for UHs. Several authors assessed different probability distributions for UH.

[Rew and McCuen \[2012\]](#) argued that the variation of rainfall events even in small distances might result in various UHs in terms of the shape and scale. They asserted that Weibull distribution gives a better fit and has larger flexibility than the gamma distribution, which is commonly used and may lead to inaccurate UHs. They further concluded that transmission losses accounts for exponentially-shaped UHs. [Hare \[1970\]](#) also found remarkable storm-to-storm variation in fitted UHs.

[Ghorbani et al. \[2017\]](#) analyzed different probability density functions and concluded that the log-normal distribution function has a great potential in predicting the time to peak and the peak flow of the UH. They further concluded that the Gamma and Pearson distributions can preserve the rising and recession limbs of the UH.

Numerous studies have been conducted and documented regarding the formulation of the UH for measured watersheds in several regions across the United States, including Maryland [[McCuen, 1989](#)], Georgia, Florida, and Texas [[Sheridan and Merkel, 1993](#)]. Various key parameters such as the UH, t_c , runoff curve number, stream Manning's n values, among others, may be refined and adjusted in accordance with gauged data. The calibra-

tion of a hydrological model might be founded on actual metrics specific to the watershed in question or extrapolated from research conducted on adjacent measured watersheds, employing methods like regional regression equations. A regional investigation of the UH in the context of measured watersheds can provide valuable insights into the selection of an appropriate UH for an ungauged watershed situated within the same geographical region. Additionally, numerous engineering textbooks, including “Hydrology for Engineers” [Linsley et al., 1975], describe methods for constructing a UH based on gauged rainfall and streamflow data.

2.1.4 Dimensionless Unit Hydrograph

The DUH is a UH where the abscissa (time) and ordinate (discharge) are normalized by time to peak (t_p) and peak discharge (q_p) respectively.

The DUH employed by NRCS was established by Mockus [1957]. The NRCS DUH was devised empirically from numerous natural UHs from a wide range of watershed sizes and geographical locations. The dimensionless curvilinear hydrograph, has its ordinate values represented as a dimensionless ratio q/q_p and its abscissa values as t/t_p . This particular UH features a point of inflection at roughly 1.7 times the time to peak (t_p). Figure 2.1 illustrates the NRCS DUH.

The variation in the DUH, a dimension-free unit, between different watersheds can be attributed to numerous influential factors. These factors encompass the watershed’s size, its geomorphic and geologic features, the slope and length of the watershed, storage capacity, the extent of the stream network’s channelization, and the level of urbanization

within the watershed [[Natural Resources Conservation Service \(NRCS\), 2007](#)].

2.1.5 Synthetic Dimensionless Unit Hydrograph and Unit Hydrograph

The term “synthetic” in the context of a synthetic UH refers to a UH that is constructed based on the properties of a watershed, rather than being derived from actual rainfall-runoff data [[Arora, 2004](#); [Bernard, 1935](#)]. The onset of the synthetic UH methodology can be attributed to the distribution graph introduced by [Bhunya et al. \[2003\]](#). This approach was designed to construct the UH based on the attributes of the watershed, as opposed to relying on rainfall-runoff data [[Singh, 1988](#)]. Several traditional methodologies for constructing Synthetic UH are detailed in references [[Espey and Winslow, 1974](#); [Gray, 1961](#); [Mockus, 1957](#); [Natural Resources Conservation Service \(NRCS\), 2007](#); [Snyder, 1938](#); [Taylor and Schwarz, 1952](#)]

Synthetic UHs are generally used in hydrologic analysis to estimate streamflow in response to a rainfall event, especially for ungauged watersheds where streamflow data is not available. Utilizing the convolution method, a given storm event can be systematically convolved with a synthetic UH. This process facilitates the generation of a runoff hydrograph.

2.2 NRCS Curvilinear Unit Hydrograph

In this section, the history, definitions, and gamma generalization of NRCS curvilinear UH are discussed.

2.2.1 History and Definitions of NRCS Curvilinear Unit Hydrograph

The standard NRCS DUH was established by [Mockus, 1957], following an analysis of small watersheds where both rainfall and streamflow were measured [Natural Resources Conservation Service (NRCS), 2007]. However, a distinct version of the DUH has been constructed for the DelMarVa region, incorporating Delaware, Maryland, and the Virginia peninsula. This alternate version based on data obtained from four monitored watersheds was established by Welle and Woodward [1989]. Also, the approach in [Natural Resources Conservation Service (NRCS), 2007] to estimate DUH deviates slightly, offering a practical method implemented through the NRCS WinTR-20 software [Natural Resources Conservation Service (NRCS), 2007].

The procedure for the DUH, established by the NRCS, was developed based on a vast array of natural UHs from watersheds of diverse sizes and geographical positions. This DUH method has been extensively employed in numerous hydrological design applications [Fang et al., 2005].

The central idea guiding this methodology is the premise that the DUH is intended for application within WinTR-20 for the scrutiny of theoretical storms (such as design storms, centennial storms, or probable maximum floods). User provides t_c , and WinTR-20 creates the UH from the DUH. The DUH is employed to construct an incremental runoff hydrograph. This is related to a brief period of excess rainfall, which is presumed to manifest at a consistent intensity. The distribution of rainfall for the storm is broken down into multiple short-duration segments, each maintaining an assumption of uniform rainfall intensity and a consistent depth across the whole watershed (or a specific subarea). This

approach adheres to the UH theory, which assumes spatial and temporal uniformity.

[James et al. \[1988\]](#) and [Meadows and Ramsey III \[1991\]](#) created computer programs that streamline this process through the use of optimization strategies. The only necessary input data are the distribution and magnitude of rainfall, observed hydrograph data, and the area of watershed drainage. The output generated by these programs encompasses the event runoff curve number, UH, PRF, plots, and statistical analysis. These applications view the watershed as a singular entity, not accounting for subdivisions into subareas, and assume a consistent rainfall and a single runoff curve number. They have been employed in the study of UHs in numerous watersheds across the nation [[Woodward et al., 1995](#)]. If the watershed is segmented into subareas, it is recommended to employ the aforementioned procedure to develop a trial and error solution. As the number of subareas and reaches in the WinTR-20 input data increase, the process becomes increasingly complex. However, if the steps are adhered to, a hydrologic model calibrated in this manner can optimally utilize limited gauge data from other locations within the watershed.

The NRCS's synthetic UH method is among the most popular UH models. It is worth noting that both the standard NRCS DUH and the DelMarva DUH were developed by empirical analysis. They are provided as sets of ordinates of (dimensionless time, dimensionless discharge). They are not described by explicit mathematical functions. The DUH is converted to a dimensional UH for a particular watershed by multiplying the ordinates by t_p and the abscissa by q_p . The NRCS recommends estimating $t_p = 0.6 \times t_c$. Peak flow is estimated in US Customary System Units as follows:

$$q_p = PRF \frac{AQ}{t_p} \quad (2.1)$$

where q_p is peak flow [*cfs*], PRF is PRF, A is watershed area [*mi*²], Q is precipitation excess/direct runoff depth [*in*] (1 for UH), and t_p is time to peak [*hr*]. The PRF is both a scaling factor and a unit conversion factor between a dimensionless UH (DUH) and the dimensional UH. Figure 2.1 shows the details of the NRCS UH initially developed by Victor Mockus [Mockus, 1957].

2.2.2 The Gamma Generalization of the NRCS Unit Hydrograph

Although it was developed empirically from many observation-based UHs, the NRCS DUH has been found to fit a gamma-distribution shape. For NRCS DUH, the gamma equation is formulated as follows [Natural Resources Conservation Service (NRCS), 2007]:

$$\frac{Q}{Q_p} = e^m \left[\left(\frac{t}{t_p} \right)^m \right] \left[e^{-m \left(\frac{t}{t_p} \right)} \right] \quad (2.2)$$

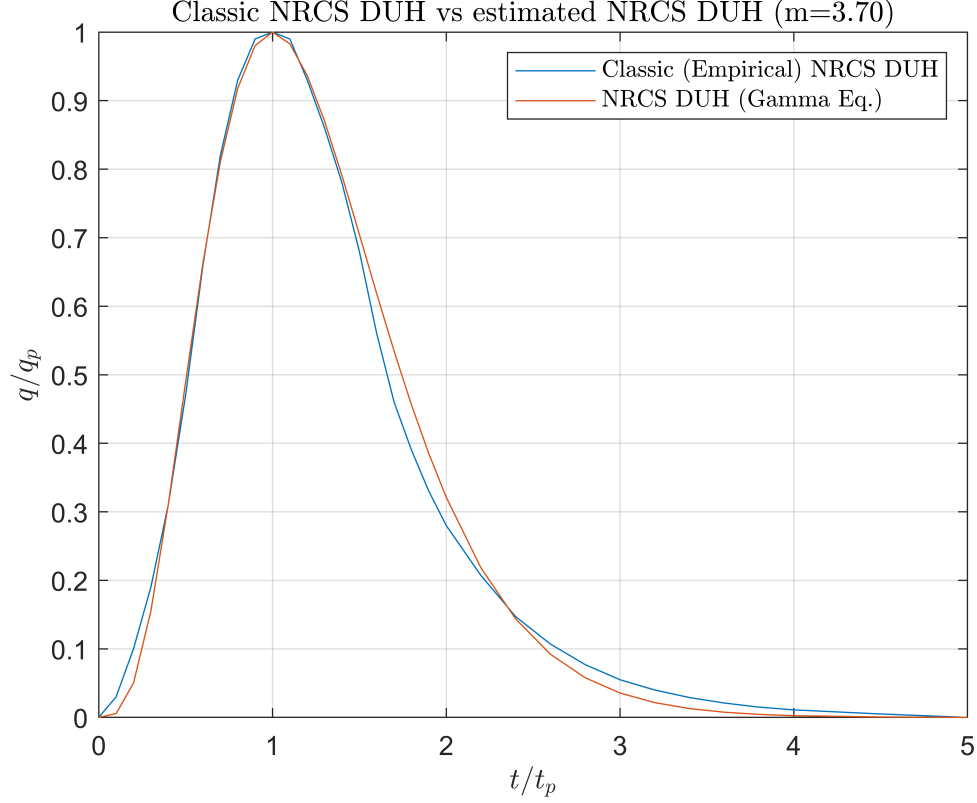


Figure 2.2: Standard (Empirical) NRCS DUH vs. Estimated NRCS DUH ($m=3.70$)

where Q/Q_p is the ratio of discharge at a certain time to the peak discharge of the UH, e is the mathematical constant (approximately 2.7183, base of the natural logarithm), m is the gamma equation shape factor, and t/t_p is the ratio of the time of UH coordinate to time to peak of the UH. Because the equation has only one parameter, m , one value of m creates a unique DUH. Figure 2.2 illustrates the close match between classic (empirical) NRCS DUH, and the NRCS DUH generated using the gamma equation with $m=3.70$. The empirical DelMarVa DUH and Gamma DelMarVa DUH ($m=1.37$) are depicted in Figure 2.3. Although the empirical NRCS DUH aligns closely with the gamma UH when $m = 3.7$, it is important to note that they are not identical. This subtle distinction is

further examined in the subsequent section of this literature review.

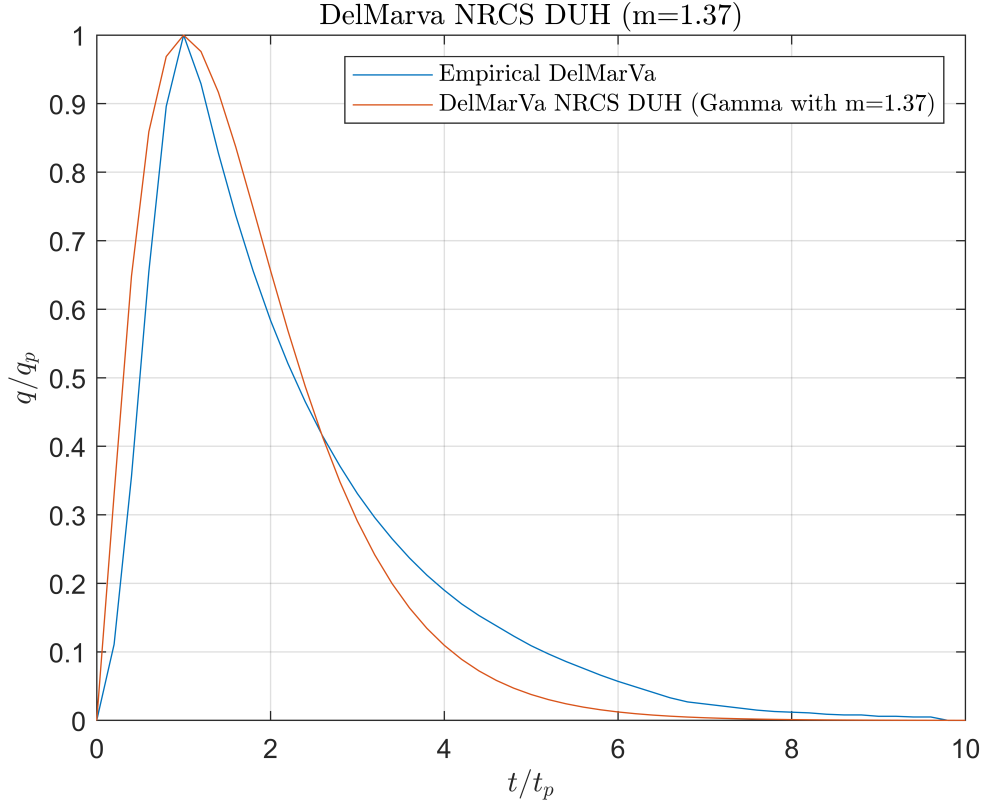


Figure 2.3: Empirical DelMarVa DUH vs. DelMarVa Gamma DUH

2.2.3 NRCS Unit Hydrograph and Instantaneous Unit Hydrograph

In Instantaneous Unit Hydrograph (IUH), the timing of the burst of rainfall is infinitesimally small, but the volume is 1 unit of rainfall, similar to the Dirac delta function. In this theoretical framework, the rainfall depth is indeed still assumed to be one unit, but the duration is effectively approaching zero [McCuen, 2016].

The NRCS Curvilinear UH, on the other hand, represents the catchment's response to a unit depth of rainfall occurring uniformly over a specific duration of time. This duration

is typically chosen based on catchment characteristics and is not infinitesimally small. For instance, the NRCS UH is often computed for a duration that corresponds to a specified fraction of t_c or the lag time of the watershed.

While the two concepts are related, the NRCS UH and IUH are not the same thing. The NRCS UH is an empirical relationship based on observed rainfall-runoff events, while the IUH is a more theoretical construct that requires additional steps to derive from observed data.

2.3 Time of Concentration

In this section, the time of concentration is defined, the NRCS velocity method used for its calculation is discussed, various empirical methods are explored, different procedures for computing the time of concentration are compared, and the role of the time of concentration in the NRCS curvilinear UH is elaborated.

2.3.1 Definition and Concept of Time of Concentration

The idea of the time of concentration (t_c) of a catchment has been recognized since at least [Mulvaney \[1851\]](#). It serves as the foundation for determining a suitable timescale for precipitation duration when using the rational method to calculate peak flows. The majority of hydrological analyses necessitate the calculation of hydrographs for a specified return period. From the hydrograph standpoint, t_c is estimated from the end of excess rainfall to the inflection point in which the recession curve starts [[Natural Resources Conservation Service \(NRCS\), 2010](#)]. The inflection point on the hydrograph is defined as the

time at which overland flow to the channel network stops; after that, the measured runoff is associated with drainage of channel storage [Melching and Marquardt, 1997]. Serrano [1997] defined t_c as the time from the end of rainfall excess to the second inflection point on the hydrograph. He refers to the second inflection point as the inflection point after the peak flow rate on the hydrograph. Out of all the time-related parameters, (including but not limited to residence time, lag time, travel time, recession time, and return period) both hydrological and classical hydraulics handbooks commonly cite t_c , as the primary parameter utilized for approximating the design discharge [Chow et al., 1988; Loukas and Quick, 1996; McCuen, 2016; Sharifi and Hosseini, 2011; Wong, 2009].

In the current literature [Bell and Kar, 1969; Ben-Zvi, 2020; Clark, 1945; Garg, 2020; Huber, 1987; Johansson, 1984; Kirpich, 1940; MacBroom, 1987; McCuen, 1997; McCuen et al., 1984; Natural Resources Conservation Service (NRCS), 2010; Schulz and López, 1974] and International Glossary of Hydrology [World Meteorological Organization (WMO) and United Nations Educational, Scientific, and Cultural Organization (UNESCO), 1992], t_c is defined as the amount of time required for the storm runoff to travel from the most distant part of a watershed to a reference point, usually the outlet downstream. Su and Fang [2004] discussed that the term time of concentration describes the length of time that elapses from the commencement of the effective rainfall until the runoff reaches 98% of its equilibrium state. Regrettably, the hydrological literature frequently neglects to differentiate between various time parameters or to clarify the correlation among the time variables utilized to calculate the time parameters such as time of concentration, lag time, and time to peak. This causes confusion and leads to several definitions being used to explain the same time parameter. McCuen [2009] pointed out that six distinct definitions

are commonly used interchangeably to describe time parameters extracted from observed rainfall and streamflow data. These definitions are:

- The duration from the cessation of excess rainfall to the inflection point on the total runoff hydrograph;
- The interval from the centroid of excess rainfall to the centroid of direct runoff;
- The interval from the intensity of maximum rainfall to the time of peak discharge;
- The period from the center of rainfall excess mass to the time of the direct runoff peak;
- The time from the centeroid of mass of precipitation excess to the total runoff peak;
- The duration from the beginning of total runoff to the total runoff peak discharge.

2.3.2 Watershed Lag-Time Method to Estimate Time of Concentration

The watershed lag estimation technique proposed by [Mockus \[1961\]](#), known as the SCS (Soil Conservation Service) method, encompasses a wide range of hydrological conditions. Lag time is a parameter used to quantify the speed of a watershed's response to a rain event that generates runoff. In essence, it is the time delay that occurs between the moment of maximum rainfall intensity and the peak discharge, or the highest point of water flow [[Horst et al., 2020](#)]. The method was developed in heavily forested watersheds characterized by steep channels and a substantial portion of runoff attributed to subsurface flow. Additionally, the method accounts for meadows that contribute to surface runoff retardance and land surfaces with minimal roughness as well as extensive areas covered by

impervious surfaces [Natural Resources Conservation Service (NRCS), 2010]. The SCS Lag Time equation is:

$$L = \frac{l^{0.8}(S + 1)^{0.7}}{1,900Y^{0.5}} \quad (2.3)$$

where L is the lag time [hr], l is flow length [ft], Y is average watershed land slope %, and S is maximum potential retention [in]. The following equation is recommended to estimate S :

$$S = \frac{1,000}{cn'} \quad (2.4)$$

where cn' is the retardance factor. The retardance factor, cn' , serves as an indicator of surface conditions, specifically pertaining to the rate at which runoff accumulates at a point of interest. It is noteworthy that the retardance factor is very similar to the curve number, CN [Natural Resources Conservation Service (NRCS), 2010]. The computation of t_c in the watershed lag method involves defining flow length as the longest route that water traverses from the watershed divide to the outlet. If the slope of the terrain is less than 0.005 [ft/ft], the following empirical equation is suggested to relate the lag time to time of concentration:

$$L = 0.6t_c \quad (2.5)$$

where L is lag time [hr] and t_c is time of concentration [hr].

During the development of the regression equation for the lag method, the longest

flow path was employed to represent the hydraulically most distant location within the watershed. Flow length can be determined through various means, such as analyzing aerial photographs, utilizing quadrangle sheets, or employing GIS techniques. Empirically, a relationship was established between flow length and drainage area by using data obtained from watersheds managed by the Agricultural Research Service (ARS). The relationship between flow length and drainage area is [\[Kent, 1973\]](#):

$$l = 209A^{0.6} \quad (2.6)$$

where l is flow length [ft] and A is drainage area [acres].

The determination of the average land slope in the context of the lag method, which should not be mistaken for the slope of the flow path, can be accomplished through various approaches [\[Natural Resources Conservation Service \(NRCS\), 2010\]](#):

- Considering it as a weighted average of the slopes assigned to soil map units obtained through a localized soil survey.
- Employing a clinometer for field measurements.
- Drawing three to four lines perpendicular to the contour lines on a topographic map and evaluating the slopes of these lines and taking the weighted average slope
- Computing the average of the land slope from grid points utilizing a dot counter
- Using the following equation [Strahler \[1964\]](#):

$$Y = \frac{100(CI)}{A} \quad (2.7)$$

where Y is average land slope [%], C is summation of the length of the contour lines that pass through the watershed drainage area on the quad sheet [ft], I is contour interval [ft], and A is drainage area [ft²] (1 acre = 43,560 ft²).

2.3.3 NRCS Velocity Method to Estimate Time of Concentration

An alternative approach employed by the NRCS for estimating t_c is known as the velocity method. This method operates under the assumption that the t_c is the cumulative sum of travel times for individual segments along the hydraulically most remote flow path. Therefore:

$$t_c = T_{t1} + T_{t2} + T_{t3} + \dots + T_{tn} \quad (2.8)$$

where t_c is time of concentration [hr], T_{ti} is travel time of a segment i [hr], n is the number of segments consisting of the total hydraulic path. Within the velocity method, the segments can be classified into three categories: sheet flow, shallow concentrated flow, and open channel flow [[Natural Resources Conservation Service \(NRCS\), 2010](#)].

Sheet flow refers to the flow that occurs over surfaces of uniform slopes. It is commonly observed in the upper reaches of a stream near the crest of the watershed boundary. Sheet flow typically persists for a distance of up to 100 feet before transitioning into shallow concentrated flow [[Merkel, 2001](#)].

To compute the travel time for sheet flow, a simplified version of Manning's kinematic solution can be employed. This simplified form of the kinematic equation was devised by [Welle and Woodward \[1986\]](#), following their investigation into the effects of different

parameters on the estimation process. Hence:

$$T_t = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}} \quad (2.9)$$

where T_t is travel time [hr], n is Manning's roughness coefficient (can be estimated as discussed in [Engman, 1986]), l is sheet flow length [ft], P_2 is 2-year, 24-hour rainfall [in], and S is slope of land surface [ft/ft]

This simplification is based on several assumptions: the presence of shallow, steady, and uniform flow; constant temporal and spatial rainfall excess intensity (representing the portion of rainfall available for runoff); the application of standard NRCS rainfall intensity-duration relationships for a 2-year, 24-hour rainfall event (Types I, II, and III); and the minor impact of infiltration on travel time. For sheet flow, the Manning's coefficient encompasses factors such as geometric roughness, raindrop impact including drag over the surface, obstacles like litter, crop ridges, rocks, erosion, and sediment transport. It is important to note that tabulated sheet-flow n values are only applicable to flow depths of approximately 0.1 foot or less [Natural Resources Conservation Service (NRCS), 2010].

Kibler and Aron [1982] and other researchers have suggested that the maximum length of sheet flow is below 100 feet. Merkel [2001] conducted a comprehensive review of technical papers on sheet flow to support the limit of 100 feet. McCuen and Spiess [1995] highlighted that using flow length as the limiting factor in Equation 2.9 might result in less precise designs. As an alternative, they proposed an equation that should serve as the basis for determining the limitation:

$$l = \frac{100\sqrt{S}}{n} \quad (2.10)$$

where n is the Manning’s roughness coefficient, l is limiting length of flow [ft] and S is the slope [ft/ft] [Natural Resources Conservation Service (NRCS), 2010].

Beyond a distance of approximately 100 feet, sheet flow typically transitions into shallow concentrated flow, accumulating in swales, small rills, and gullies. Shallow concentrated flow is characterized by a lack of well-defined channels and by flow depths ranging from 0.1 to 0.5 feet. To estimate the travel time of shallow concentrated flow, velocities can be determined using “Velocity vs. Slope” figure developed by Natural Resources Conservation Service (NRCS) [2010], where the average velocity is influenced by the type and slope of the watercourse [Kent, 1964].

After computing average velocity, the travel time for the shallow concentrated flow segment can be computed using the following equation:

$$T_t = \frac{l}{3,600V} \quad (2.11)$$

where T_t is travel time [hr], l is distance between the two points under consideration [ft], V is the average velocity of flow between the two points [ft/s], and 3600 is the the conversion factor from [sec] to [hr].

The occurrence of shallow concentrated flow is postulated to continue at shallow depths ranging from 0.1 to 0.5 feet. Beyond this point, channel flow is presumed to take place. The initiation of open channels is assumed to coincide with the availability of surveyed cross-sectional data, the identification of channel bed and banks through aerial

photographs, or the presence of bluelines (indicating streams) on U.S. Geological Survey (USGS) topographic maps [[Natural Resources Conservation Service \(NRCS\), 2010](#)].

In the channel flow, the estimation of average flow velocity can be achieved using either Manning’s equation or water surface profile information. Typically, average flow velocity is computed at the bankfull elevation. The Manning’s equation in U.S. Customary System Units is:

$$V = \frac{1.49}{n} \cdot R_h^{2/3} \cdot S^{1/2} \quad (2.12)$$

where V is the average velocity [ft/s], R_h is hydraulic radius [ft], S is slope of the hydraulic grade line equal to channel slope in uniform flow [ft/ft], and n is Manning’s roughness value for open channel flow. Manning’s roughness n values for open channel flow may be obtained from hydraulic references including [[Chow, 1959](#); [Linsley et al., 1975](#)] or other resources [[Arcement and Schneider, 1989](#); [Barnes, 1967](#); [Cowan, 1956](#); [Phillips and Ingersoll, 1998](#)].

The velocity approach for determining t_c is based on hydraulic principles, and affords the flexibility to modify individual flow segments as necessary. This method stands as the optimal approach for computing the t_c in the context of urbanizing watersheds or when there are prospective hydraulic modifications to the watercourse under consideration.

[Fang et al. \[2007\]](#) analyzed t_c for Texas watersheds. They concluded that the t_c estimated using the NRCS velocity method is subject to large variation, and is dependent upon the method and parameters selected for estimating flow velocity. They concluded that there is no single value that represents the t_c for a watershed and recommend caution

in application of the NRCS velocity method to compute t_c .

[Perdikaris et al. \[2018\]](#) estimated the reference t_c for nine catchments in the Credit River watershed, Ontario, Canada. The catchments were subjected to calibration and testing using nine different empirical models. Results indicated that the NRCS velocity method was the most reliable and accurate approach. Their study highlighted that the reason for the superior accuracy of the NRCS velocity method over the other empirical equations is due to the structure of the model that incorporates the Manning's roughness coefficient in the approach.

[McCuen \[2009\]](#) critiqued the velocity method and principal flow path concept to calculate t_c in an analysis on different watersheds. He concluded that the velocity method designed for flow along the principal flow path led to shorter estimated t_c than any of the computations derived from hyetograph-hydrograph analyses. He found that on average, the runoff velocity on the principal flow path was roughly 43% faster than velocity of runoff across the entire watershed.

[Kibler \[1982\]](#) and [Sharifi and Hosseini \[2011\]](#) applied kinematic wave theory and derived equations to estimate t_c .

2.3.4 Empirical Methods

In this section, a variety of empirical approaches to estimate t_c are discussed, and comparisons are made among the procedures used to calculate t_c as presented in various scholarly sources.

Numerous studies developed empirical formulas and applied them on watersheds with

different sizes and at various geographical scales. [McCuen et al. \[1984\]](#) assessed eleven empirical equations for estimating t_c . The watersheds had areas less than 4,000 acres. They found there was significant variation in bias and accuracy.

Most empirically-based methods to estimate t_c calibrate the equation to a specific geographic region; the equation cannot be applied in other regions without proper calibration. Among those empirical equations derived to estimate t_c , [Ravazzani et al. \[2019\]](#) identify the 24 equations most frequently used. These empirical equations are: 1- Bransby-William's Model [[Williams, 1922](#)], 2- Giandotti's Model [[Giandotti, 1934](#)], 3- Kirpich's Model [[Kirpich, 1940](#)], 4- Johnstone and Cross's Model [[Johnstone and Cross, 1949](#)], 5- California Culvert Practice's Methodology [[California Highways and Public Works, 1955](#)], 6- Federal Aviation Agency's Formula [[Federal Aviation Administration \(FAA\), 1970](#)], 7- NRCS-SCS Method [[Mockus, 1972](#)], 8- Texas Department of Transportation's Formula [[Marek, 2011](#)], 9- Ventura's Formula [[Ventura, 1905](#)], 10- Pasini's Formula [[Pasini, 1910](#)], 11- Tournon's Formula [[Merlo, 1973](#)], 12- Puglisi and Zanframundo's Formula [[Puglisi and Zanframundo, 1978](#)], 13- Fattorelli and Marchi's Formula [[Fattorelli and Marchi, 1988](#)], 14- Pezzoli's Formula [[Indelicato, 1988](#)], 15- Eaton's Method [[Eaton, 1954](#)], 16- Sheridan's Formula [[Sheridan, 1994](#)], 17- T  mez's Formula [[T  mez, 1978](#)], 18- Chow's Model [[Chow et al., 1988](#)], 19- Dooge's Formula [[Dooge, 1973](#)], 20- Haktanir and Sezen [[Haktanir and Sezen, 1990](#)], 21- Espey's Method [[Espey et al., 1966](#)], 22- Pilgrim and McDermott's Formula [[Pilgrim and McDermott, 1981](#)], 23- Ferro's Model [[Ferro, 2006](#)], and 24- Bocchiola's Model [[Bocchiola et al., 2004](#)]. [Martins et al. \[2016\]](#) observed that the empirical equations are easy to use because they are based solely on the geometrical characteristics of the watershed.

[Salimi et al. \[2017\]](#) concluded that the Bransby-Williams formula is the most effective

way to approximate t_c in a watershed, and that the size of the area is a crucial factor. They also explained that this is due to the fact that there is a large correlation between t_c and the area of the watershed (A). In a study of Indonesian watersheds, [Yulianur et al. \[2019\]](#) created a simple multiple regression formula and used the length of the flow path on the land, L and slope of the land, S . They indicated that the formula created only applies to land cover by *Zoysia Japonica* grass and cautioned that it should not be extended to other regions with different land covers.

2.3.5 Comparison and Evaluation of Time of Concentration Methods

A few researches discussed the significance of t_c and its impact in structures and accurate estimation of peak discharge.

2.3.5.1 Application, Importance, and Impact of Time of Concentration

[Gericke \[2018\]](#) found that catchment response time, design rainfall, and to some extent, runoff coefficient, are the crucial components for design flood approximation in ungauged watersheds, and they have a major impact on the design of hydraulic structures.

According to [Fang et al. \[2008\]](#), underestimation and overestimation of peak flow and t_c are inversely related. Therefore, when t_c is overestimated, the peak flow is underestimated and vice versa. This is consistent with the NRCS peak flow equation, if t_p is proportional to t_c .

2.3.5.2 Empirical/Novel Methods for Determining Time of Concentration

[Roussel et al. \[2005\]](#) argued that the preferable approaches for t_c computation is the Kerby-Kirpich approach for the selected watersheds. The authors combined Kerby approach [[Kerby, 1959](#)], which measures the time for overland flow component, and Kirpich approach [[Kirpich, 1940](#)], which measures the time for channel flow component. They further discussed that the Kerby-Kirpich method is relatively straightforward to apply and that it mimics the time to peak in NRCS approach. This preference is also based on the simplicity of the approach and ease of input-data acquisition. The Kerby-Kirpich approach produces t_c values, which, through the conventional NRCS conversion, mimic time to peak from auxiliary analysis of observed rainfall and runoff data for the 92 watersheds in their study.

[Fang et al. \[2008\]](#) applied three different methods using digital elevation models and geographic information system (GIS) software, the manual procedure with watershed delineation, and the manual method without watershed delineation to estimate t_c and compare them with empirical equations. They concluded that the t_c developed by different methods are qualitatively similar but empirical equations produce larger differences in estimating t_c .

[Li and Chibber \[2008\]](#) carried out two kinds of lab experiments to gauge the duration of overland flow on surfaces with minimal inclines. The first involved a rainfall test using a portable artificial rainfall simulator, while the second was an impulse runoff test. They further argued that the empirical equations to estimate t_c were based on the data obtained from existing watersheds and these equations might not be very useful to estimate t_c in flat terrains. They measured t_c in a laboratory setting; their results indicated that antecedent

soil moisture data can be useful for estimating t_c .

[Pavlovic and Moglen \[2008\]](#) explored the effect of spatial resolution in estimating travel time from a digital longitudinal slope profile. The central question addressed in the paper is whether there is a variation between t_c approximates computed using traditional approaches on a coarsely discretized flow path compared to computational approaches that take full advantage of the high resolution of GIS data. The authors examined an idealized system with an analytical solution. They also studied a data set compiled from watersheds across Maryland and modeled a channel from a Vermont watershed for which field observations were available. The authors found that the t_c estimate rises as the level of discretization increases. They developed an anticipatory equation that estimates the high discretization level t_c as a function of a single-segment t_c estimate and findings derived from a high-resolution longitudinal slope profile.

[Sharifi and Razaz \[2014\]](#) combined GIS and genetic programming to develop an empirical equation for a case study watershed to estimate t_c . The authors claim that their equations outperform conventional empirical t_c equations and require fewer input parameters. The accuracy of their equations to estimate t_c in other regions was not tested.

According to [De Almeida et al. \[2016\]](#), the majority of studies focused on the development of relevant equations to t_c , have been based on data for watersheds of temperate regions. The authors asserted that t_c and runoff generation processes may differ under various conditions, such as in tropical and subtropical climates. Their model only uses the hydrograph of an event at the base level of a watershed (outlet) without precipitation data. The Harmony Search optimization algorithm was employed to optimize the model parameters.

[Gericke and Smithers \[2016\]](#) pointed out that the estimation of catchment response time parameters, including t_c , is crucial in design floods, especially in ungauged watersheds. However, they noted that flood hydrology practice in South Africa lacks unique working definitions for each of the parameters, leading to high variability and uncertainty in the estimation of time parameters. They further argued that conceptual t_c values might be computed using both the t_c in response to the event means, and a linear basin response function. They believed this approach would provide a more robust and objective outcome.

[Gericke and Smithers \[2018\]](#) applied a consistent approach to estimate catchment response time parameters in South Africa. They discussed that by using solely the observed stream flow data to assume that t_c approximately equals time to peak, it was found that the precision of any time-related parameter derived from observed hyetographs or hydrographs is reliant on the accuracy of the corresponding input variables used in the computation. They applied the proposed time to peak estimation procedure based on a linear catchment response time, which is dependent only on observed stream flow variables. The procedure is also less impacted by the lack of precipitation data and non-uniform spatial and temporal precipitation distribution in medium to large watersheds. Authors concluded that the approximation of $t_c \simeq t_p$ could be used as a basis for an alternative approach at large catchment scales.

2.3.5.3 Uncertainty in Time of Concentration Concept and Approaches

[McCuen \[2009\]](#) addressed the potential bias in watershed runoff timing due to the use of the principal flow path for computing times of concentration. The author argued

that travel times based on the principal flow path and application of velocity method are shorter than times based on the entire watershed, which might lead to inaccurate estimated peak discharge rates. The author investigated the potential bias using measured rainfall-runoff data from 27 watersheds and computer simulation. Several definitions of the t_c were compared, and the expected uncertainty in the alternative definitions was evaluated. In his results, the potential bias occurred regardless of the time parameter applied in the method to compute t_c , indicating these methods do not accurately reflect the temporal distributions of actual flood runoff.

[Gericke and Smithers \[2014\]](#) determined that large errors in peak discharge estimates at watershed scales can be attributed to errors in the approximation of basin response time. The authors conducted a review of various methods used to estimate catchment response time. They compared a selection of overland flow t_c approaches using varied slope-distance classes and roughness parameter categories. They concluded that the lag time and t_c should be further investigated to expand a substitute method to compute watershed response times that lead to improved estimates of peak discharges. Additionally, the application of empirical methods, without local correction factors, beyond their original developmental regions must be avoided.

[Nagy et al. \[2016\]](#) highlighted that t_c is one of the most frequently used parameters that describe morphology and runoff from a watershed. However, the accuracy of runoff predictions greatly depends on a good initial estimate of t_c . The researchers employed 14 distinct empirical equations to calculate t_c for their investigation. Tarján Creek, situated in northeastern Hungary, was the experimental gauged watershed. The accuracy of the empirical equations was assessed using the Hydrologic Engineering Center’s Hydrologic

Modeling System (HEC-HMS) software to simulate the runoff. Depending on the selected method in HEC-HMS, the software makes specific assumptions about t_c , t_p , and UH. The measured runoff data were used to calibrate both continuous and event-based models. The study demonstrated that the empirical estimates for t_c vary greatly. The Wisnovszky equation, which is widely used in Hungary, was found to underestimate t_c . The authors concluded that substantial variability exists among methods to estimate t_c , and design discharges derived from these estimates can potentially contain significant errors.

[Azizian \[2017\]](#) discussed the shortcomings of empirical equations to calculate t_c and the calibration required for each specific region. This study concluded that there is no best equation to estimate t_c .

[Azizian \[2018\]](#) went on to evaluate and analyze the uncertainty of t_c equations, while also taking into account the uncertainty associated with varying map scales and inputs. The author applied the coefficient of variation as a statistical test to quantify uncertainty. Further examination of the parameter uncertainties in the t_c equations demonstrates that the uncertainties pertaining to rainfall attributes and land cover coefficients significantly affect the outcomes of the t_c equations. Additionally, the impact of these uncertainties varies between humid regions and dry or semi-dry regions.

[Yogi et al. \[2021\]](#) examined the sensitivity of flow estimation to variations in the Soil Conservation Service (SCS, now called NRCS) parameters, including recurrence interval, t_c , and curve number. Multiple models for approximating direct runoff were used [[Alves et al., 2019](#); [De Mello et al., 2016](#); [Mohammad and Adamowski, 2015](#); [Soomro et al., 2019](#); [Yu, 2012](#)] but the SCS-CN approach was found to be the best for rainwater flow modeling applied to the design of hydraulic and hydrological simulation in various locations.

Investigation confirmed the flow’s response to land cover progression, watershed water retention time, and recurrence duration, which was further corroborated through hypothesis testing and analysis of variance.

2.3.5.4 The Effect of Shape and Geometry of Terrain on Time of Concentration

[Sabzevari et al. \[2013\]](#) discussed the variability in t_c estimation on different hillslopes. They argued that the geometry and shape of complex hillslopes including the degree of flow convergence (convergent, parallel, or divergent) and the profile curvature (concave, straight, or convex) affect the travel time of surface flow. The authors further generalized the shape factor of complex hillslopes introduced by [Agnese et al. \[2007\]](#) and presented a new model for calculating t_c of complex hillslopes. This model considers the shape of plan, the type and degree of profile curvature, the Manning’s roughness coefficient, the flow pattern, the length, the average slope, and the excess rainfall intensity. Based on their results, t_c for convergent and divergent hillslopes is roughly twice as much as those of parallel and divergent ones. The magnitude of t_c in convex hillslopes was close to that in straight and concave hillslopes. Although the influence of convergence on t_c is remarkable, the curvature effect was confirmed to be inconsequential. Finally they showed that, in convergent hillslopes, t_c increases with the degree of convergence, but in divergent hillslopes, it reduces as degree of divergence expands.

[Baiamonte and Singh \[2016\]](#) illustrated the estimation of overland flow travel time for hillslopes of complicated topography. They extended the kinematic wave t_c which

was obtained for rectangular and curved parallel hillslopes. They considered degree of convergence or divergence of the hillslopes and compared their results with previous studies.

2.3.5.5 Concept of Celerity and Velocity and the Impact on Time of Concentration

[McDonnell and Beven \[2014\]](#) asserted that velocities and celerities should be distinguished in hydrological analysis. The consideration of the difference between celerities and velocities in watershed responses results in the conclusion that process representations should be scale dependent [[Beven, 2006, 2012](#); [Rinaldo et al., 2011](#)]. The celerity, the speed at which a perturbation to a flow propagates, is crucial in understanding the outflow hydrograph and the pattern of velocity, the actual rate of displacement of water at a given place and time [[Beven, 2020](#)]. In distinguishing velocity and celerity, however, it is essential to deal not only with the differences between them but also how they adjust with time and space and connectivity. New developments in current theories are essential to reconcile the observations that become available [[Davies et al., 2011, 2013](#)].

2.3.6 Time of Concentration in Clark’s Unit Hydrograph

The Clark’s UH method uses an IUH concept to route excess precipitation to the subbasin outlet. This is done by applying a unit depth (e.g., one inch or one centimeter) of excess precipitation over a watershed instantaneously [[Clark, 1945](#)]. Although it is theoretical (since real precipitation is not instantaneous), this method effectively characterizes a watershed’s response to rainfall without depending on the duration of the precipitation

excess [Chow et al., 1988]. This method incorporates two essential processes in converting precipitation excess to runoff: translation (movement), which is the movement of excess precipitation through the watershed to the outlet, and attenuation (reduction), which reduces the discharge magnitude as precipitation excess is stored within the watershed temporarily. Conceptually, water moves from distant points within the watershed to the outlet with a delay but without reduction in flow. The reduction in flow, known as attenuation, is then conceptually applied at the watershed outlet. In addition to t_c , there are two other parameters including a watershed storage coefficient R , which is equivalent to reduction of flow because of storage effects within the watershed [Kull and Feldman, 1998], and Time-Area histogram representing the area of the watershed which contributes to flow at the outlet as a function of time.

Kar et al. [2015] applied Clark’s UH parameter estimation including t_c and R . Authors found a comprehensive relationship among t_c , R , and the Time-Area histogram. Authors further claimed that the observed and simulated runoff responses were conveniently accessed by the Clark’s UH method and application of Clark’s UH can be regarded as a guide for ungauged catchments in which precipitation data and stream runoff data are available.

2.3.6.1 Paradox in the Concept of Time of Concentration

This section discusses studies that critique or question the concept, definition or methodologies of t_c .

Grimaldi et al. [2012] analyzed four small basins using empirical approaches. These case studies revealed that the available methods for the computation of t_c might result in

numerical estimates that differ from each other by 500%.

[Michailidi et al. \[2018\]](#) contend that the estimation of t_c creates a paradox in hydrologic science. In their paper, they adopted the rational method and kinematic approach, using a GIS environment for approximating travel time. They concluded that the t_c is a negative power function of intensity of precipitation excess, contradicting widespread belief that t_c is an invariant property of a watershed. They also highlighted the necessity of an approach considering varying t_c in hydrological modelling and major shifts in current engineering practice.

[Beven \[2020\]](#) conducted an exhaustive review of the concept of t_c . The author meticulously examined the inconsistencies and discrepancies present in the existing literature pertaining to this concept. The key points of the paper are outlined as follows: 1- The concept of celerity, which refers to the speed at which a perturbation to a flow propagates downstream, controls the outflow hydrograph. 2- The t_c should not be defined in terms of water travel times but rather in terms of celerities and time to equilibrium. This is because celerities can be greater than velocities and can vary for different intensities and durations of rainfall and antecedent conditions. 3- The difference between celerities and velocities is likely to be large in soil types with a minimal effective storage deficit above the water table, particularly in conditions that are nearly saturated or have a significant capillary fringe. 4- The glossary definition of t_c should be replaced by a discussion of time to equilibrium and celerities, noting that recession limb celerities are different from those that define a time to equilibrium under a steady uniform input. 5- An Appendix provides a quantitative analysis of the difference between velocity and celerity responses, making it clear why the traditional definition of t_c is inconsistent with the way in which it is commonly used in practice. 6-

The time to equilibrium for a headwater channel reach of length L could be considered as L/a , where a is an asymptotic velocity at high discharges. The t_c will approach the time to equilibrium at high flows. 7- The paper concludes that the concept of t_c is not really useful in explaining hydrological responses and recommends that it should be replaced by a discussion of celerities and time to equilibrium.

2.3.7 Role of Time of Concentration in the NRCS Curvilinear Unit Hydrograph

Based on the [Natural Resources Conservation Service (NRCS), 2007], t_c is defined physically/conceptually as: 1- The time for runoff to move from the most remote point (hydraulically) in the watershed to the outlet, and mathematically/graphically as: 2- The time from the end of precipitation excess to the inflection point on the receding limb of the UH.

The two aforementioned relationships are significant due to their computational implications. Specifically, t_c is determined based on the first definition, the second definition contributes to the calculation of t_p (time to peak) and ΔD (the unit storm duration) in the context of the NRCS UH. The calculated t_p is utilized across all points on the abscissa of the DUH by applying the ratio t/t_p .

As depicted in National Engineering Handbook, [Natural Resources Conservation Service (NRCS), 2007] the DUH according to the NRCS exhibits a time to peak at a single unit of dimensionless time, with an inflection point at roughly 1.7 units of dimensionless time. Through the application of the relationships $\text{Lag} = 0.6 t_c$ and the point of inflection =

1.7 t_p , the resultant ΔD equates to 0.2 t_p . [Natural Resources Conservation Service \(NRCS\) \[2007\]](#) procedures allow for minor fluctuation in ΔD , but recommend that it should not exceed 0.25 t_p . The standard NRCS DUH is provided in tabular form at an interval of 0.1 units of dimensionless time. When ΔD is 0.2 t_p , each second point is employed during calculations to derive the UH. If ΔD reaches 0.25 t_p or more, the form of the DUH is not accurately depicted. Based on the geometry of the DUH, the relation between ΔD and t_c is computed as follows:

$$t_c + \Delta D = 1.7t_p \quad (2.13)$$

$$\frac{\Delta D}{2} + 0.6t_c = t_p \quad (2.14)$$

combining Equations [2.13](#) and [2.14](#) results in a relationship between t_c and ΔD as follows:

$$\Delta D = 0.133t_c \quad (2.15)$$

$$t_c \approx 7.5\Delta D \quad (2.16)$$

and,

$$t_p = \frac{\Delta D}{2} + 0.6t_c \quad (2.17)$$

when t_c is replaced by ΔD ,

$$t_p \approx 5\Delta D \quad (2.18)$$

2.4 Peak Rate Factor

This section provides a brief overview of the underlying concept of the PRF and summarizes the literature on PRF computation. The academic literature pertaining to the PRF within the science of hydrology is notably limited. Furthermore, the PRF is absent from other UH models, such as Snyder’s UH and Clark’s UH, and is uniquely associated with the NRCS UH. Despite this, given its role as a scaling factor, the PRF could potentially be developed for any other UH (this idea is explored and tested in Chapter 6 of the present dissertation).

2.4.1 Definition

The PRF in the NRCS UH is defined as a scaling factor that influences the shape of the UH and magnitude of the UH’s peak flow. It is used in transforming the NRCS Curvilinear DUH to a watershed-specific UH and reflects the proportion of runoff that happens prior to the peak.

2.4.2 Role of Peak Rate Factor in Application of NRCS Unit Hydrograph

The NRCS synthetic UH method developed by [Mockus \[1957\]](#) is among the most popular UH models and that is the basis to estimate PRFs of various watersheds in the State of Maryland. Based on the NRCS formula, peak flow can be estimated in Equation

2.1. The PRF is both a unit conversion and a scaling factor between a DUH and the dimensional UH.

2.4.3 Calculation of Peak Rate Factor for Generalized Gamma NRCS DUH

In the US Customary Units, the UH integrates to 1 inch:

$$\int_0^{t_{\max}} q(t) dt = 1 \text{ inch} \quad (2.19)$$

where q is UH discharge [in/hr], t is time [hr], and $t_{\max}$ [hr] is the time extent of the UH.

The DUH does not integrate to 1 inch, as shown using the variable definitions:

$$\int_0^{t_{*,\max}} q_*(t_*) dt_* = \int_0^{t_{\max}/t_p} \frac{q}{q_p} \frac{dt}{t_p} = \frac{1}{q_p t_p} \underbrace{\int_0^{t_{\max}} q(t) dt}_{1 \text{ inch}} \quad (2.20)$$

where q_* and t_* are dimensionless time and discharge, q_p [in/hr] is peak discharge of the UH and t_p [hr] is time to peak of the UH. If the above equation is solved for q_p :

$$q_p = \frac{\int_0^{t_{\max}} q(t) dt}{\int_0^{t_{*,\max}} q_*(t_*) dt_*} \frac{1}{t_p} \quad (2.21)$$

the integral in the numerator represents the total depth of the UH, 1 inch, as given by Q in Equation 2.1. Based on Figure 2.1, approximating the UH as a triangle, the area under the UH can be computed as:

$$Q = \frac{q_p t_p}{2} + \frac{q_p t_r}{2} = \frac{q_p}{2} (t_p + t_r) \quad (2.22)$$

where Q is in inches and t is in hours. Solve for peak rate:

$$q_p = \frac{2Q}{t_p + t_r} \quad (2.23)$$

where q_p is in inches per hour. Now, by introducing a dimensionless variable K :

$$K = \frac{2}{1 + \frac{t_r}{t_p}} \quad (2.24)$$

consequently:

$$q_p = \frac{KQ}{t_p} \quad (2.25)$$

where q_p is in inches per hour. The general equation is obtained by converting the units from inches per hour to cubic feet per second and expressing it in commonly used terms. Drainage area (A) in square miles, the conversions from hours to seconds, from square miles to feet, and from inches to feet, and time (t) in hours are incorporated into the previous equation which becomes the following general equation:

$$q_p[ft^3/s] = \frac{645.33[\frac{(ft^3/s) \times h}{mi^2 \times in}] \times K[-] \times A[mi^2] \times Q[in]}{t_p[hr]} \quad (2.26)$$

where, q_p = peak discharge [ft^3/s], K = dimensionless factor, A = drainage area [mi^2], Q = runoff depth [in] (equals 1 for UH), t_p = time to peak [hr], and 645.33 is the dimensional conversion factor for the rate required to discharge 1 inch from 1 square mile in 1 hour, computed as follows:

$$\frac{A[mi^2] \times 5280^2 \left[\frac{ft^2}{mi^2} \right] \times Q[in] \times 0.0833 \left[\frac{ft}{in} \right]}{t_p[hr] \times 3600 \left[\frac{s}{hr} \right]} = 645.33 \left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right] \quad (2.27)$$

for the NRCS UH, the product of the dimensional conversion factor of 645.33 and dimensionless K parameter generates a PRF as follow:

$$PRF = 645.33 \times K \quad (2.28)$$

therefore, once K is estimated, the PRF can be computed. Applying the triangular approximation to the standard NRCS curvilinear DUH, $t_r/t_p = 5/3 \approx 1.67$, gives $K = 0.75$ (Equation 2.24) which results in PRF of 484. Therefore, for the standard NRCS DUH:

$$q_p = \frac{484AQ}{t_p} \quad (2.29)$$

Geometrical analysis of the triangular approximation shows that the fraction of runoff that occurs during the rising limb of the UH is equal to:

$$\frac{Q_{rise}}{Q} = \frac{t_p}{t_p + t_r} = \frac{1}{2}K \quad (2.30)$$

Since $K = 0.75$ for PRF 484, the fraction of runoff under the rising limb is 0.375. With modern computational tools, the gamma UH can be analyzed directly without making the triangular approximation.

A modification in the DUH corresponding to a different percentage of volume during the ascending limb will result in a corresponding adjustment of the shape factor. Conse-

quently, this alteration affects the constant value of 484. The PRF has been found to differ significantly, ranging from approximately 600 in areas with steep topography to 100 or less in flat, marshy regions. UHs characterized by specific PRFs are detailed in the National Engineering Handbook [[Natural Resources Conservation Service \(NRCS\), 2007](#)].

2.4.4 Studies of Peak Rate Factor

[McCuen and Bondelid \[1983\]](#) addressed the issue of estimating PRFs in the Soil Conservation Service (SCS, now called NRCS) rainfall-runoff model TR-20, which is widely used for hydrologic design. The model uses a standard value of 484 for the PRF, but the authors argued that this value may not be accurate for all designs, especially for coastal watersheds. The authors aimed to provide a systematic method for obtaining a value of the PRF that is unique to a watershed, which should increase the accuracy of hydrologic designs. The authors further proposed a method that requires a runoff curve number map, which should be developed to use the TR-20 model. The method was tested on six watersheds, including three coastal watersheds. The authors used a version of the SCS TR-20 program that generates automatic fitting to observed hydrographs to compute the best-fit relationship. The results of the study indicated that the values derived from topographic data highly agree with those from the hydrologic records. This suggested that the procedure outlined for deriving the DUH from topographic and land use data is a reasonable approach. The authors concluded that the use of the procedure should increase the accuracy of designs. They also recommended that the standard value of 484 for the PRF is too large for coastal watersheds on the DelMarVa peninsula.

[Sheridan et al. \[2002\]](#) proposed a method to determine the appropriate PRFs for flatland experimental watersheds. They evaluated the relationship between PRF and different developed hydrograph time parameters in flatland watersheds. In another study [Fang et al. \[2005\]](#) concluded that the shape parameter in the DUH should be adjusted if a different PRF is used. [Horst and Gurriell \[2019\]](#) explored the potential regional trends in the PRF for selected catchments in New Jersey during Hurricane Irene in 2011. Their examination did not identify any substantial local or regional trends. Furthermore, they evaluated numerous catchment parameters to assess any correlation with the PRF, but the results were not found to be statistically significant.

[Lasco et al. \[2022\]](#) focused on the estimation of peak discharge as one of the primary goals of hydrologic modeling. The authors explored the PRF recommended by the NRCS for determining peak discharges with the UH approach. They also investigated the PRF's dependency on watershed and storm characteristics. The study used a dataset of 1,648 rainfall-runoff events in 104 watersheds in Texas. The authors used the precipitation and flow dataset documented in [\[Asquith et al., 2004\]](#), as well as a second dataset that included watersheds in the Houston area. They generated event excess-rainfall hyetographs with the curve number method. For each event, a UH was estimated by deconvolving the direct runoff hydrograph with the excess rainfall hyetograph, assuming it follows a two-parameter gamma distribution. They also performed statistical analysis to determine which watershed characteristics have a significant effect on the PRF. The study concluded that the PRFs of the Houston events have a median of 0.58 and a 90th percentile of 1.35 (SI or Metric System). The PRF is primarily dependent on the watershed's geomorphology but not on its slope. This dependency, although statistically significant, explains only marginally the

PRF variability.

[Horst et al. \[2023\]](#) compared two methodologies: the hydrologic method using UH theory and basin averaged parameters, and the hydraulic method using 2D terrain and hydrodynamic equations. The goal of the study was to simulate runoff hydrographs using both methods and compare the results to measured values recorded at a USGS gauging station. The authors used the Hydrologic Engineering Center’s River Analysis System (HEC-RAS) for the hydraulic method and the NRCS procedure via HEC-HMS for the hydrologic method. A total of 29 watersheds were analyzed, with more than one historical storm in eleven of the watersheds, leading to a total of 45 rain events. The authors used high-resolution digital elevation models (DEMs) and various watershed features for the hydraulic modeling. The study found that neither technique consistently produced accurate results compared to historical data. The hydraulic simulations generated average peak flow and runoff depth errors of 103% and 67% respectively, while the hydrologic simulations had average peak flow and runoff depth errors of 105% and 81% respectively. However, the findings for both models tended to improve for experiments involving larger catchment areas, longer flow return periods, and larger precipitation events. The authors concluded that rainfall–runoff modeling using a hydraulics-based method delivers more accurate results with respect to both peak flow and direct runoff computations. As higher resolution terrain information and different catchment characteristics become more readily available, the findings of the hydraulic modeling should theoretically improve, potentially resulting in a substantial shift in the method that the practicing engineering community takes while performing this sort of analysis.

[Horst et al. \[2022\]](#) applied the NRCS synthetic UH method and determined the PRFs

based on the historical rain events. Using NRCS method, authors back-calculated site-specific PRFs for 14 different watersheds that have encountered two or more significant rainfall events since 2000. The results indicated that PRFs remained stable for five of these watersheds, regardless of the storm magnitude. However, nine watersheds exhibited a large variation in PRF among the various storm events. A correlation could not be definitively established. The authors determined “storm-specific” curve numbers by analyzing the hydrologic response of each storm event. They collected data on precipitation and runoff for each storm and used these observations to calculate the curve number, which reflects the relationship between rainfall and runoff for that specific event. The study also found that when the PRFs reach a certain level, the corresponding storm-specific curve number is much less than expected. In conclusion, the authors found that the PRF varies significantly for different storm events and watersheds. They also found that high PRFs correspond to storm-specific curve numbers that are much lower than expected.

Chapter 3: Initial Study of Maryland Watersheds

3.1 Introduction

The goal of this study was to investigate two properties of unit hydrographs (UH) for Maryland watersheds: time of concentration, t_c , and peak rate factor (PRF). The Natural Resources Conservation Service (NRCS) Unit Hydrograph (UH), as implemented in their WinTR-20 software, is widely used in Maryland. The PRF and t_c together define the shape and scale of the NRCS UH.

A hydrograph is a representation of watershed response to a precipitation event, in terms of volumetric flow rate as a time function. They are generally used by engineers to assist in designing hydraulic structures (e.g., culverts, bridges, dams...). For design purposes, one of the most practical ways to estimate final design hydrographs is to convolve unit hydrographs of watersheds with associated precipitation events (i.e., 20-year storm) [Horst and Gurriell, 2019]. UHs are crucial tools to estimate runoff hydrographs resulting from storm rainfall. The application of the UH is primarily on the runoff estimation on ungauged watersheds, in which they are known as synthetic unit hydrographs. Unit hydrograph models were developed by [Clark, 1945; Mockus, 1957; Snyder, 1938]; they are applied in variety of hydrological modelling [Sheridan et al., 2002].

A UH can be developed through analysis of the rainfall-runoff events which are acces-

sible through historical records or through the analysis of various watershed characteristics applied in combination with a synthetic unit hydrograph model. [Dooge \[1959\]](#) discussed that the UH model to estimate stream flow is one of the most powerful tools in applied hydrology. The NRCS UH method developed by [Mockus \[1957\]](#) is among the most popular unit hydrograph models and is the basis of most engineering practice in the state of Maryland. This UH model makes use of a PRF to quantify the peak of the UH. Based on the NRCS formula, peak flow can be estimated as follows:

$$q_p = PRF \frac{AQ}{t_p} \quad (3.1)$$

where q_p =peak flow [cfs]; PRF=peak rate factor; A =watershed area [mi²]; Q =runoff depth [in] (For UH Q equals 1); and t_p =time to peak [hr]. The PRF is both a unit conversion and a scaling factor between a dimensionless UH and the dimensional UH.

PRF is a representative of the unit hydrograph shape (i.e., volume distribution). That is, it shows a degree of peakedness of the unit hydrograph. The typical PRF reproduced by [\[Mockus, 1957\]](#) is 484, which represents the distribution of 3/8 of the volume of the streamflow runoff on the rising limb of the hydrograph and 5/8 of the streamflow volume on the recession limb of the unit hydrograph. Alternatively, a PRF of 284 which is applied in the Delmarva unit hydrograph [\[Welle and Woodward, 1989\]](#) approximates a volume distribution of 22% under the UH rising limb and 78% under the UH receding limb. PRF of 284 applies to watersheds in coastal areas. [\[Natural Resources Conservation Service \(NRCS\), 2007\]](#) stated that “the value of the PRF can vary from around 300 for roughly swampy and flat catchments to 600 for steep watersheds,” however, based on the literature

reviewed in this research, PRF ranges can vary from below 100 to above 600.

The accurate computation of PRF is essential for engineering design of hydraulic structures. Current methods do not accurately compute PRFs due to inconsistencies in the unit hydrograph modeling. This study uses fine-resolution sub-hourly rainfall data and associated sub-hourly runoff data for several events to simulate NRCS unit hydrograph and to compute PRF for selected watersheds in Maryland. This study also investigates if any regional or local pattern for computed PRFs can be identified and if each watershed can be represented with a unique PRF.

An additional objective of this chapter was to investigate catchment time of concentration (t_c) and its dependence on catchment features. This aspect of the study was intended to enhance an in-depth study by W. Thomas and colleagues in 2000 [Thomas et al., 2000] for the state of Maryland, which in turn built on work by [Dilow, 1998].

3.2 Study Location and Data

Study locations were selected from USGS gauging stations. The scope of this research was limited to the time period for which NEXRAD DPR product for the precipitation data are available, starting in 2012 (the DPR product is discussed later in this chapter). The annual maximum discharge time series for the gauges were assessed to identify events with peak flows greater than a 2-year return period (less than 50% annual exceedance probability). For consistency with the method assumption, jagged and two-peak shape hydrographs were not selected. Likewise, any watersheds greater than 100 mi² (259 km²) were excluded from this research as larger basins are less consistent with the methodology

of unit hydrograph.

A total of 54 watersheds were selected for this study in the two Maryland regions, 30 in region 1 (Piedmont, Blue Ridge and Appalachian Plateau geographic regions) and 24 in region 2 (Eastern and Western Coastal Plain). Tables 3.1 & 3.2 list the stations including the number of events for each station in each region. Figure 3.1 shows the locations of the watersheds. Further details are provided later in this chapter.

Table 3.1: Selected Maryland Watersheds (Region 1)

Stations	# of Events	Stations	# of Events	Stations	# of Events
1580000	1	1585200	1	1594000	2
1581500	1	1586000	1	1596500	2
1581700	3	1586210	3	1599000	1
1582000	1	1586610	2	1617800	3
1583100	2	1589300	3	1637500	2
1583500	4	1589440	7	1639140	1
1583570	1	1591000	6	1643500	4
1583580	1	1591400	3	1644371	2
1583600	1	1591700	3	1644372	1
1584500	2	1593500	1	1644375	1

Table 3.2: Selected Maryland Watersheds (Region 2)

Stations	# of Events	Stations	# of Events	Stations	# of Events
1483200	1	1493112	3	1594526	1
1484100	2	1493500	2	1649500	2
1485500	1	1581757	3	1651000	2
1486000	2	1585090	1	1653600	1
1486500	1	1585100	3	1658000	2
1490000	1	1585104	2	1660920	2
1491500	2	1585230	1	1661050	1
1492500	3	1589500	1	148471320	2

In the Figure 3.1, green color represents watershed located in Piedmont, Blue Ridge, and

Appalachian Plateau (region 1). Blue color represents watersheds located in Eastern and Western coastal plains (region 2). Some of the watersheds are very small at this scale.

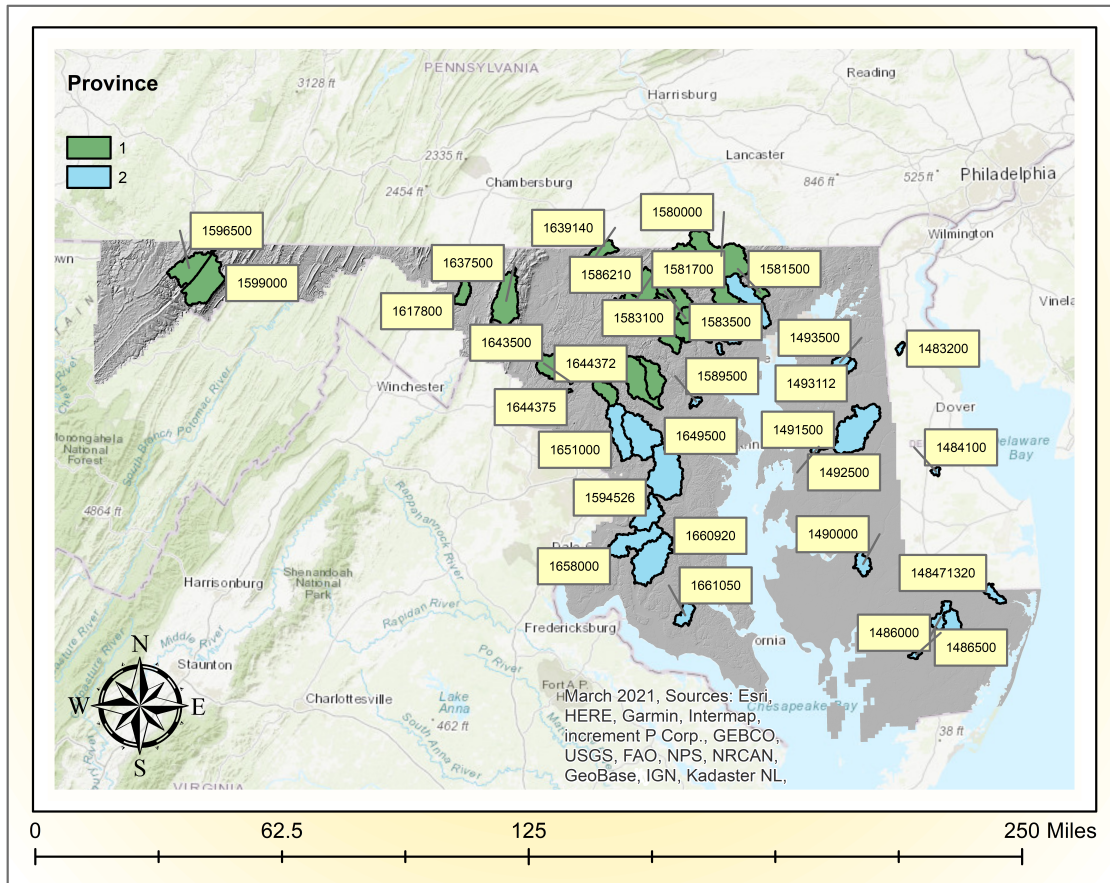


Figure 3.1: Green: Region 1 (Piedmont, Blue Ridge, and Appalachian Plateau); Blue: Region 2 (Eastern and Western Coastal Plains). Map created in ESRI ArcMap. Some small or nested watersheds are hidden at this scale.

3.3 Methods

3.3.1 Streamflow Data

For the selected Maryland watersheds, 15-min discharge data from stream gauges were downloaded from the United States Geologic Survey's (USGS) National Water Information

System (NWIS) [[U.S. Geological Survey \(USGS\)](#), n.d.].

3.3.2 Precipitation Data

Sub-hourly precipitation gauges are very scarce in the state of Maryland. Therefore this research applies Next Generation Weather Radar System (NEXRAD) products in developing hyetographs for storm events. Digital precipitation rate (DPR) data were obtained from the National Centers for Environmental Information (NCEI) online database [[National Oceanic and Atmospheric Administration \(NOAA\)](#), n.d.] as described in [[Brubaker et al., 2021](#)]. These are level-III data, indicating that the data has been processed from the raw reflectivity signal to a gridded precipitation intensity. This study obtains data from the WSR-88D radar at Sterling, Virginia (KLWX) since its range covers the entire state. Based on the NCEI recommendation, this research uses Weather and Climate Toolkit (WCT) application [[National Oceanic and Atmospheric Administration \(NOAA\)](#), 2021] to observe and process data. Then, WCT is used to export DPR values to comma-separated values (CSV) format text files.

The DPR data product is arranged on a radial grid centered on the KLWX radar; the grid elements are 1° by 250 *m* (radial distance). NEXRAD records the DPR product as frequently as 3-minutes during intense rainfall; therefore, a 24-hour day could have up to 480 CSV files, each identified by date and time. These CSV files were operated on, using Unix text editing tools, to simplify them to a data set containing three columns: precipitation intensity value, angle, and radial distance.

For a given reporting time, t , average rainfall depth over a watershed is given by the

integral of spatially-varying precipitation depth, $p(\mathbf{x},t)$:

$$\bar{p}(t) = \frac{1}{A} \int p(x, t) dA \quad (3.2)$$

where x is location on the radial grid, t is time, and A is total area of the watershed. To perform this integral, each study watershed is subdivided into cells corresponding to the DPR's radial grid, using the polygon intersection tool in ArcGIS. The DPR polygons were mapped by using the WCT to export a single time sweep with nearly full coverage in GIS shapefile format. With each row of the DPR CSV file corresponding to a cell in the grid, the average rainfall depth can be calculated as a weighted average,

$$\bar{p}(t) = \sum_{i=1}^n \frac{A_i}{A} p_i(t) \quad (3.3)$$

where n is the number of DPR radial grid cells that intersect with the watershed, A_i/A is the area fraction of the intersection, and $p_i(t)$ is the DPR precipitation rate in that grid cell at that time. The DPR precipitation values and the area fractions are referenced to the DPR grid by (radial distance, angle) indices, 1 to 920 for the radial distance and 1 to 360 for the angle. Watershed average precipitation rate can be efficiently calculated by elementwise multiplication (Hadamard product) of two matrices,

$$\bar{p}(t) = \boldsymbol{\alpha} \odot \mathbf{P}(t) \quad (3.4)$$

where $\mathbf{P}(t)$ is the (radial distance, angle) matrix of precipitation at time t , and $\boldsymbol{\alpha}$ is the (radial distance, angle) matrix of watershed area fractions. A MATLAB[®] script was used

to calculate a time series of watershed average precipitation for each selected event.

NEXRAD products database deliver data with small space and time resolution accompanied by unknown bias. Sub-hourly event precipitation time series were adjusted by combining daily gridMET data obtained from online Climate Engine database [Climate Engine, n.d.] and the NEXRAD level III product. Climate Engine delivers daily watershed average gridMET precipitation for an uploaded shapefile outline of the watershed.

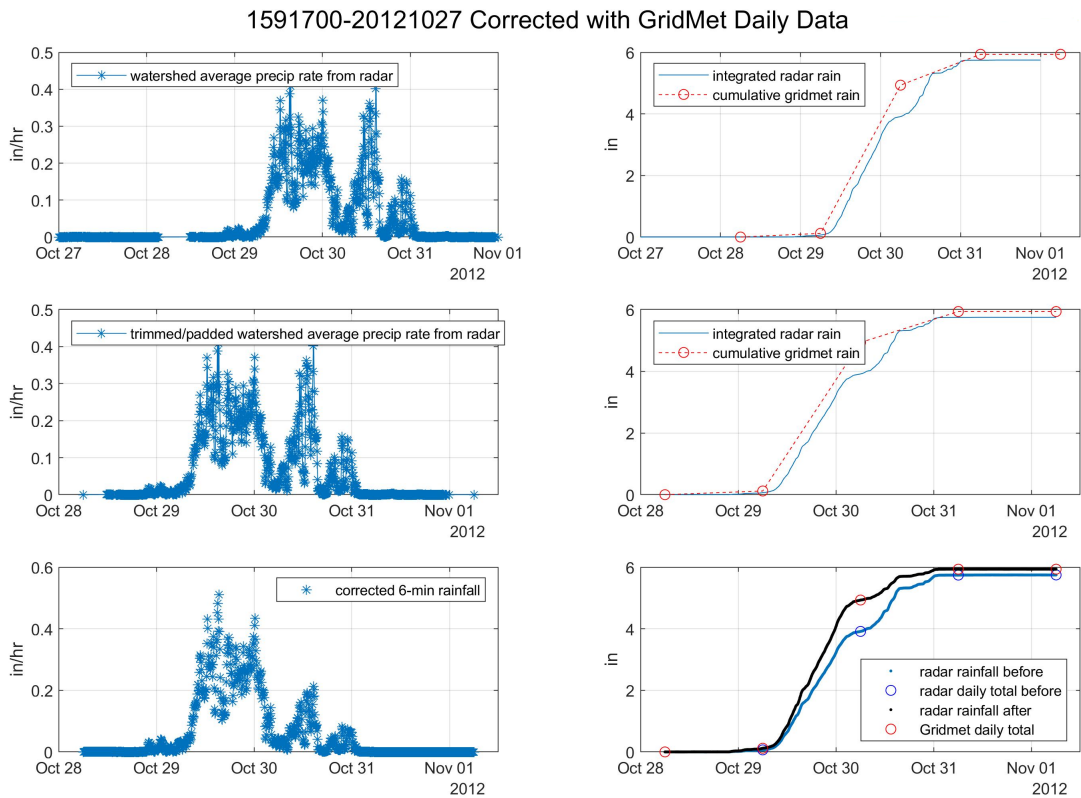


Figure 3.2: An example of adjusting watershed-average radar rainfall using gridMET watershed-average daily values. The original data, with radar data on a variable time step and gridMET data on a daily timestep (top row), are trimmed or extended as necessary to cover matching time periods (second row). The radar data are adjusted on a daily basis to match the gridMET, then disaggregated to a 6-minute time series (bottom row). (MATLAB® data dashboard graphical tool by K. Brubaker)

The watershed-averaged DPR time series at irregular time intervals (3 min or more) were modified on a daily basis to match the daily watershed values from gridMET. Corrected average precipitation was temporally disaggregated to 0.1 hour (6 minutes) time interval. The result of above steps is an event hyetograph of watershed average precipitation at 0.1 hour (6 minutes) interval. The steps are illustrated in Figure [3.2](#).

3.3.3 Event Hydrograph and Hyetograph Analysis

The Unit Hydrograph (UH) converts excess precipitation into direct runoff. To create a UH for a specific precipitation and runoff event, it is crucial to distinguish between the hyetograph components, which comprises both excess precipitation and losses or abstracted precipitation, and the runoff hydrograph, which consists of both direct runoff, the water that quickly flowed overland to reach the stream gauge, and baseflow, the water that infiltrated and moved slowly or was previously stored in the subsurface and then flowed to the gauge during the event. It is necessary to separate the hyetograph and hydrograph into their components.

3.3.4 Hydrograph Data

This section provides a brief discussion on the separation of baseflow and the estimation of direct runoff derived for an event.

3.3.4.1 Baseflow Separation

There are various techniques in the literature to separate baseflow from a hydrograph, such as constant-discharge, constant-slope, recursive filtering, and concave methods. Many graphical hydrograph separation methods require the analyst to determine an “inflection point” on the receding limb, which is believed to indicate the end of direct runoff and the start of pure baseflow. However, this point may not always match the mathematical definition and can be subjective. To address this issue, an algorithm was employed to automate the identification of baseflow recession and reduce subjectivity. The algorithm traces the recession curve and determines the time after which an exponential decay equation best describes the hydrograph shape. This time is regarded as the end of direct runoff and the start of pure baseflow. In a modified version of the concave method, it is assumed that baseflow rises linearly from the beginning of the event until the end of precipitation, where the linear rise intersects with the extended exponential recession curve. However, in certain cases, it may be necessary to continue pre-event baseflow recession in a downward-sloping line for a short period before connecting the rising linear segment to the recession curve to prevent the assumed baseflow-rise line from surpassing the total hydrograph curve. By constructing these segments, baseflow is estimated for the whole event. Similar to other graphical separation methods, this technique provides an estimate of the complicated, spatiotemporal variable processes taking place in a watershed.

3.3.4.2 Direct Runoff Hydrograph

The direct runoff (DRO) time series is determined by subtracting the baseflow from the total hydrograph of the event. The event depth of DRO is then calculated by numerically integrating the DRO hydrograph. Figure 3.3 displays the baseflow separation for event 2012/10/28, watershed ID 1591700. An algorithm generated in MATLAB® is used for this purpose, which requires as inputs the event hydrograph, watershed area, and the date/time of the end of the precipitation event. The MATLAB® script separates the baseflow and calculates the volume of DRO normalized by watershed area.

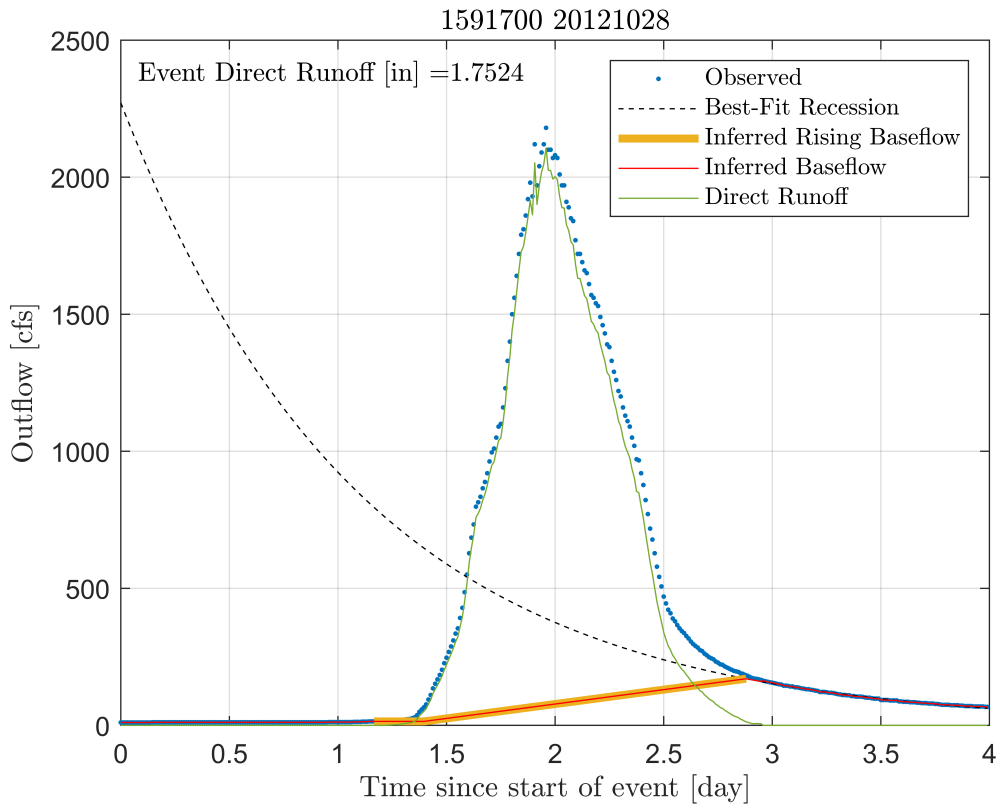


Figure 3.3: Baseflow Separation Example, Watershed ID: 1591700

3.3.5 Determination of Rainfall Excess

The method assumes that depth of DRO is the same as the depth of excess precipitation (PEX). To distinguish rainfall excess from the event hyetographs, the phi-index technique [McCuen, 2016] is employed using a algorithm generated in MATLAB®. An example is shown, corresponding to the rainfall excess estimation in Figure 3.4. The phi-index method assumes a constant potential loss rate for the entire event, which may result from storage or infiltration. The actual loss during a specific time increment equals precipitation or ϕ , whichever is less.

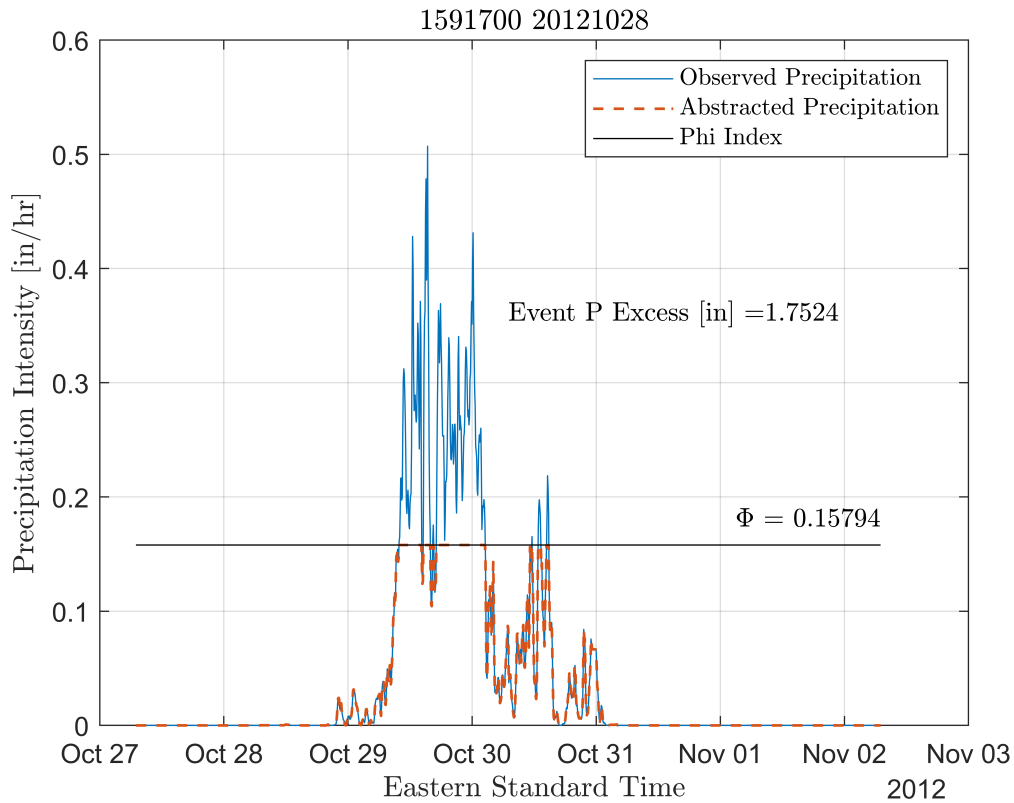


Figure 3.4: Phi-Index Precipitation Separation Example, Watershed ID: 1591700

The MATLAB[®] script iteratively determines the value of ϕ that balances the integrated event PEX with the event DRO. In this research the NRCS excess precipitation method was assessed as well; however, it posed difficulties and subjectivity due to the need for the watershed curve number (CN) computation in its procedure.

3.3.6 Unit Hydrograph Analysis

The next step is to estimate the unit hydrograph (UH) that converts the PEX (Section 3.3.5) to the DRO (Section 3.3.4). The event DRO hydrograph is obtained by integrating the PEX time series and the UH using a convolution integral. Multiple techniques exist in literature for determining a UH from PEX and DRO time series. In this research, an optimization algorithm using MATLAB[®] was implemented. The dimensionless unit hydrograph (DUH) was constrained by the gamma function in Equation 2.2. This equation can be restated in dimensionless form:

$$q^* = e^m (t^*)^m e^{-mt^*} \quad (3.5)$$

where q^* is dimensionless discharge (q/q_p) and t^* is dimensionless time (t/t_p). At $t^* = 1$, $q^* = 1$. At $t^* = 0$, q^* equals zero and as t^* becomes very large, q^* tends to zero. Equation 3.5, the DUH does not integrate to 1; the dimensional unit hydrograph (UH) expresses dimensional discharge as a function of dimensional time and must integrate to 1. The task is to estimate the gamma UH that best translates precipitation excess to direct runoff. Gamma UH has two parameters consisting of unit hydrograph time to peak, t_p (scale parameter) and gamma shape parameter, m .

The best-fit UH is identified by a quantitative objective function. In this study, two different objective functions were applied: 1- The sum of squared errors (differences) between the observed hydrograph and the convolved hydrograph, and 2- the match between observed and predicted peak discharge in terms of timing and magnitude. In this study, instead of applying an optimization algorithm that systematically searches the mathematical (m, t_p) space to optimize the objective function, the algorithm explores the entire parameter space and computes the objective functions for a range of parameter pairs. Using this method can help to observe how the fit between observed and convolved (predicted) direct runoff fluctuates with the parameter pairs. Shape parameter, m , is selected for the gamma function rather than PRF since m is dimensionless and more fundamental from a mathematical standpoint. The m parameter can easily be transferred to its equivalent PRF if necessary.

3.3.7 Estimation of the Unit Hydrograph

For each event, sets of parameters (t_p) and (m) are provided to the algorithm. For every possible (t_p, m) pair, the DUH is computed using Equation 3.5. The corresponding dimensional unit hydrograph (UH) is derived as follows: every ordinate of the DUH is divided by the area under the DUH curve (integral) to ensure that the area under the dimensional unit hydrograph equals 1. The values on the abscissa are then multiplied by the values of t_p , and the ordinates are scaled to maintain the area under the dimensional unit hydrograph equal to 1. The precipitation excess hyetograph is convolved with the UH and all 3 objective functions are computed: 1- sum of the squared errors (SSE), 2-

difference in peak discharge (Q_{peak}), and 3- difference in time to peak (T_{peak}). After all pairs of (t_p, m) are tested, two best-fit pairs of (t_p, m) are determined: the one leading to the minimum SSE, and the one at which the Q_{peak} error = 0 and $T_{\text{peak}} = 0$ contour lines intersect. An example is displayed in Figure 3.5.

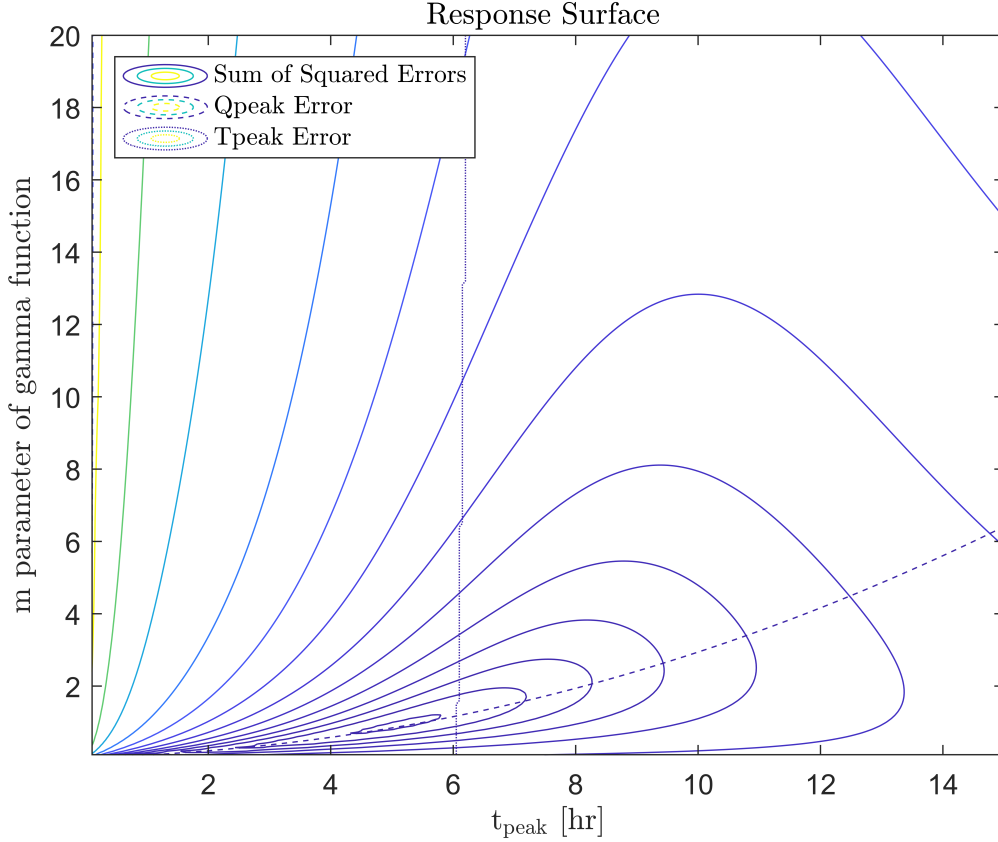


Figure 3.5: Response surface showing contours of three objective functions (measures of goodness of fit between predicted and observed hydrograph): overall sum of squared errors (solid contours), difference in magnitude of Q_{peak} (single dashed contour at Q_{peak} Error = 0), and difference in time to peak (single light dotted contour at T_{peak} error = 0), Watershed ID: 1483200

In the response surface, the solid lines are contours of SSE. The dashed line accounts for the (t_p, m) pairs in which the peak discharge difference (Q_{peak} error) is zero, meaning the

Q_{peak} in predicted hydrograph equals the Q_{peak} in observed hydrograph. The dotted line shows the (t_p, m) pairs in which the time to peak difference (T_{peak} error) is zero, meaning the T_{peak} in the predicted hydrograph equals the T_{peak} in the observed hydrograph. In this example, the minimum SSE takes place at $(t_p, m) \cong (5 \text{ hour}, 1)$. The predicted hydrograph peak matches the observed peak (both in magnitude and timing) where the dashed line and dotted line intersect and that is $(t_p, m) \cong (6 \text{ hour}, 1.2)$.

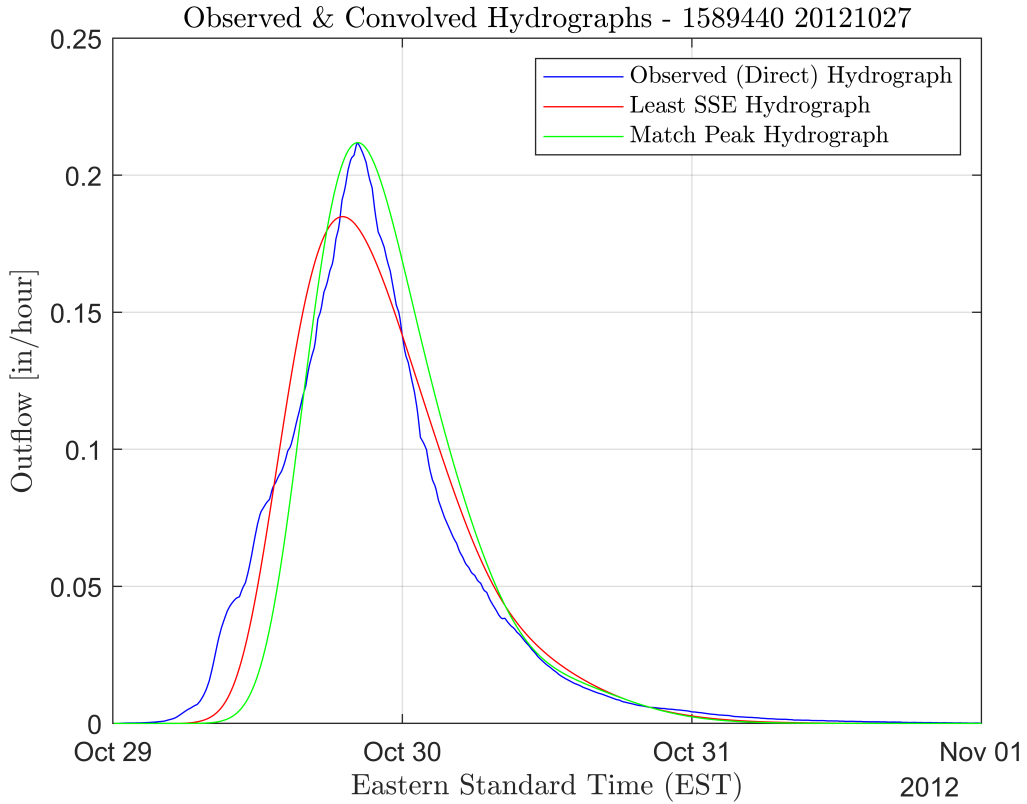


Figure 3.6: Hydrographs resulting from convolution of selected candidate unit hydrographs by two different goodness-of fit measures (objective functions): overall Sum of Squared Errors (“Least SSE”) and best match to timing and magnitude of peak discharge (“Match Peak”), Watershed ID: 1589440

The resulting predicted hydrographs and the observed hydrograph for Watershed ID:

1483200 are displayed in Figure 3.6. In addition, the dimensionless and dimensional unit hydrographs corresponding to the response surface and convolved hydrographs are displayed in Figure 3.7.

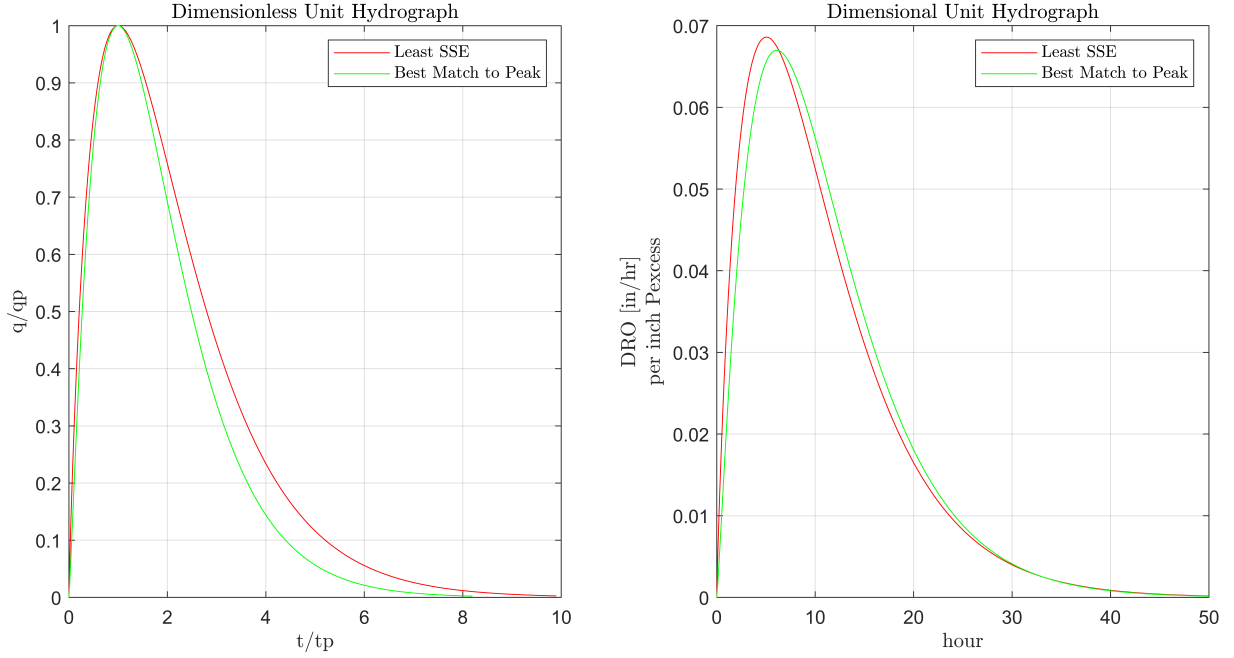


Figure 3.7: Dimensionless Unit Hydrograph (DUH) (Left) and Dimensional Unit Hydrograph (UH) (Right) corresponding to the optimal (m, t_p) selected by two different objective functions, as shown in Fig. 3.5. The DUH shape (left) reflect only the difference in the gamma parameter m , and do not integrate to 1. The dimensional UH shapes (right) reflect different t_p as well. Both UH curves integrate to 1 inch of direct runoff, Watershed ID: 1483200

In the state of Maryland, PRF is broadly used in water resources engineering practice and is one of the target parameters in this research. Thus, PRFs are calculated for each watershed event. Shape parameter of the gamma function, m , generates a unique dimensionless unit hydrograph and accordingly a unique PRF by applying the following equation:

$$PRF = \frac{645.33}{\int q^* dt^*} \quad (3.6)$$

where PRF is the peak rate factor, q^* is dimensionless discharge (q/q_p) and t^* is dimensionless time (t/t_p). The integral in the denominator is dimensionless. Both the PRF and constant 645.33 have units $\left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right]$ [Natural Resources Conservation Service (NRCS), 2007]. The MATLAB[®] script integrates the DUH numerically and reports the values of the PRFs.

3.3.8 Estimation of Time of Concentration from Event Data

Over many years of hydrologic practice, there have been various definitions for t_c in the literature (Section 2.3). Technically, the idea behind the time of concentration is that it is the time needed for the whole watershed to contribute to the runoff at the watershed outlet. However, identifying the end of direct runoff (also known as an mathematical “inflection point”) as the indicator for this response time may or may not be accurate.

A conceptually rigorous procedure to estimate t_c is to identify the time from the end of PEX to the inflection point on the receding limb of the event hydrograph. Mathematically, the inflection point is defined as the point where the hydrograph curve moves from concave downward to concave upward; that point represents the minimum value of the first derivative dQ/dt , that is, a zero of the second derivative, d^2Q/dt^2 . Physical reasoning and observations describe that the change in the concavity of the hydrograph receding limb indicates the time at which water contributes to the watershed from the baseflow or storage in the watershed; and hence the time once the entire watershed has responded to

the precipitation input. This is one of the most universally-used definition of t_c . This procedure is automated using another MATLAB® script. Figure 3.8 illustrates the time of concentration results using above procedure.

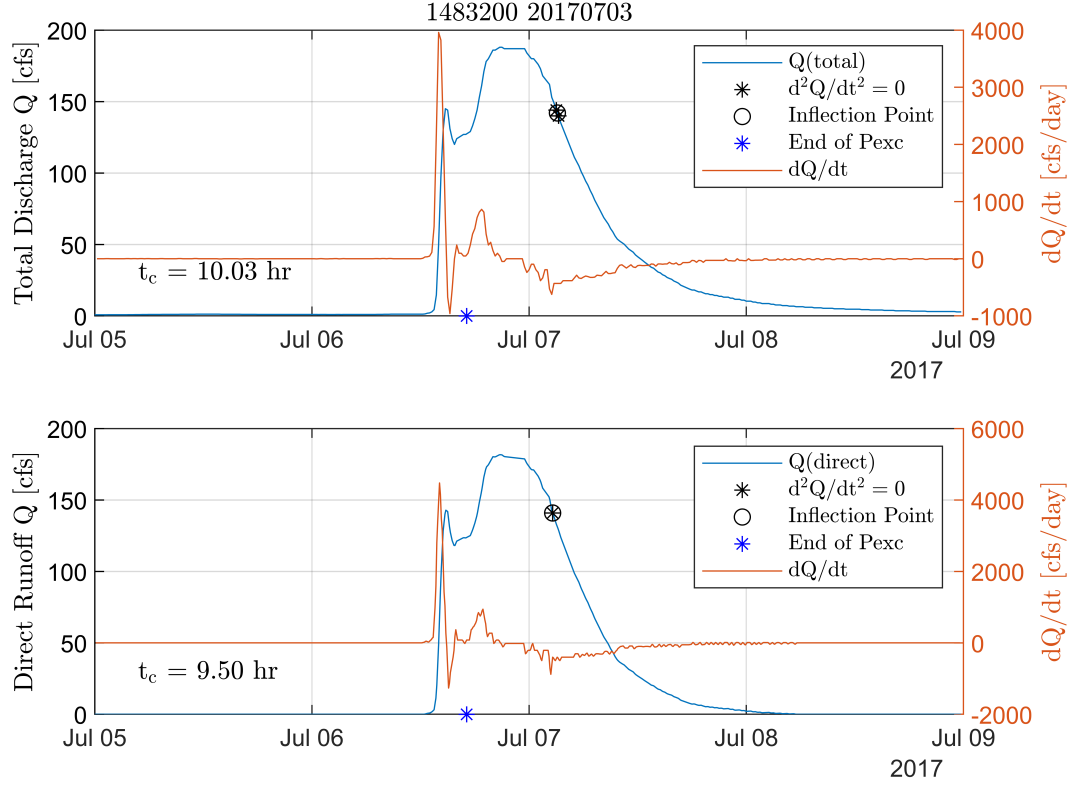


Figure 3.8: Identification of Time of Concentration using the event hydrograph recession inflection point. The hydrograph (blue) is numerically differentiated (red curve is first derivative dQ/dt), Watershed ID: 1483200

In this script, the total discharge and direct runoff hydrograph are numerically differentiated, identifying the minimum value of the first derivative on the receding limb. The minimum value of the dQ/dt is associated with the change in sign of the second derivative, and also the change from concave downward to concave upward. In case more than one point in time shows the minimum derivative values, the time and discharges are averaged.

In the method, the time when the PEX stops is captured from prior steps in the analysis. Time of concentration (t_c) is computed as the difference between the time at which inflection point takes place and the time at which PEX ceases. This procedure is performed for both the total discharge hydrograph (including baseflow) and the direct runoff hydrograph.

While there is no global consensus over the definition of time of concentration and there is no preferred method to calculate time of concentration in the literature, it was believed that this automated computation of time of concentration eliminates some subjectivity in identifying accurate t_c . While this automated procedure is arguably objective, it is still subject to multiple other assumptions and subjective decisions in the process of separating hyetograph and hydrograph to derive PEX and DRO. For a number of events, the time at which PEX ended takes place after the time at which inflection point occurred on the receding limb of the hydrograph, resulting in negative time of concentration (t_c) which is technically meaningless. It is not clear whether this meaningless outcome derives from inappropriate precipitation time series or inconsistent and inaccurate rainfall excess separation, or it weakens the inflection point concept to identify time of concentration (t_c). These negative results for time of concentration occurred in a number of watersheds for a few long-lasting, heavy precipitation events in the data set. Given the above findings, a different procedure was employed to estimate time of concentration (t_c) for the study watersheds and events in this research. The following section elucidates the method and demonstrates that it is a satisfactory replacement for the event hydrograph inflection point procedure.

3.3.9 Estimating Time of Concentration from Unit Hydrograph

The rationale of the NRCS curvilinear hydrograph assumes that t_c is the time from the end of the unit rainfall excess to the mathematical inflection point on the recession limb of the UH [Natural Resources Conservation Service (NRCS), 2007]. Applying the gamma UH, the inflection point can be determined in closed form by taking the second derivative with respect to time, and setting it equal to zero. Given the dimensionless gamma shape parameter, m , and t_p estimated from the optimization algorithm discussed previously, the corresponding time can be computed as a function of m and t_p . Since the end of precipitation excess on the dimensionless unit hydrograph (DUH) occurs at time 0.1 [hr], that value is subtracted from the time to inflection point to obtain the time of concentration. This method is among the least subjective methods available to calculate the time of concentration. In this demonstration, the independent variable t/t_p is defined as x . Equation 2.2 is differentiated twice, and the result is set to zero to estimate the time of concentration. The first derivative of $x^m e^{(m-mx)}$ with respect to x is computed initially, followed by the differentiation of this result to obtain the second derivative. Let:

$$f(x) = x^m e^{(m-mx)} \quad (3.7)$$

Step 1: Find the first derivative using the product rule: $f'(x) = u'v + uv'$. Let $u = x^m$ and $v = e^{(m-mx)}$; therefore:

$$u' = mx^{(m-1)} \quad (3.8)$$

$$v' = e^{(m-mx)}(m - mx)' = e^{(m-mx)}(-m) \quad (3.9)$$

$$f'(x) = mx^{(m-1)}e^{(m-mx)} + x^m e^{(m-mx)}(-m) \quad (3.10)$$

$$= (mx^{(m-1)} - mx^m)e^{(m-mx)} \quad (3.11)$$

Step 2: Find the second derivative by differentiating $f'(x)$. Using the product rule again, let $u = (mx^{(m-1)} - mx^m)$ and $v = e^{(m-mx)}$; therefore:

$$u' = m(m-1)x^{(m-2)} - m^2x^{(m-1)} \quad (3.12)$$

$$v' = e^{(m-mx)}(-m) \quad (3.13)$$

$$f''(x) = [m(m-1)x^{(m-2)} - m^2x^{(m-1)}]e^{(m-mx)} + (mx^{(m-1)} - mx^m)e^{(m-mx)}(-m) \quad (3.14)$$

$$= [m(m-1)x^{(m-2)} - m^2x^{(m-1)} - m^2x^{(m-1)} + m^2x^m]e^{(m-mx)} \quad (3.15)$$

$$= [m(m-1)x^{(m-2)} - 2m^2x^{(m-1)} + m^2x^m]e^{(m-mx)} \quad (3.16)$$

thus, the second derivative of $x^m e^{(m-mx)}$ with respect to x is:

$$f''(x) = [m(m-1)x^{(m-2)} - 2m^2x^{(m-1)} + m^2x^m]e^{(m-mx)} \quad (3.17)$$

the above expression is simplified by expanding and rearranging terms, resulting in the compact form shown in the Equation [3.18](#).

$$me^{(m-mx)}(m(x-1)^2 - 1)x^{m-2} = 0 \quad (3.18)$$

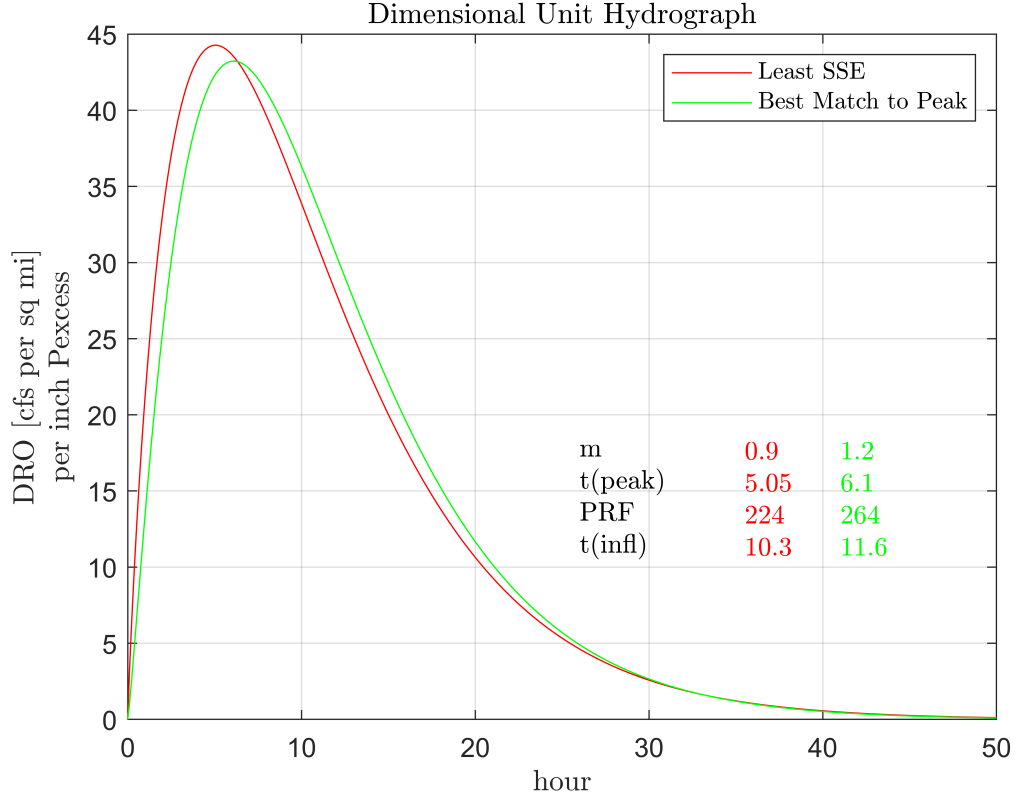


Figure 3.9: Example final results of unit hydrograph identification, giving values for two candidate UH's: gamma m, time to peak, PRF, and time from end of unit precipitation excess to recession inflection point, Watershed ID: 1483200

The equation above is rearranged and simplified to solve for x , yielding the following result:

$$x = \sqrt{\frac{1}{m}} + 1 \quad (3.19)$$

Therefore, the dimensional time from end of UH PEX to the inflection point is:

$$t_{infl} = \left(\sqrt{\frac{1}{m}} + 1 \right) t_p - 0.1 \quad (3.20)$$

The result of time of inflection and other gamma parameters for watershed ID 1483200 is

depicted in Figure 3.9. This time parameter is computed from each unit hydrograph and reported as time of inflection in the results. This research denotes this time parameter as time to inflection point, rather than time of concentration (t_c), to emphasize the method applied in estimating the parameter.

3.4 Statistical Analysis

The gamma unit hydrograph is described by two characteristics: the gamma m shape parameter, which is dimensionless, and the time to peak, measured in hours and denoted as t_{peak} . The study sought to determine whether these UH parameters are associated with – or can be predicted by – measurable watershed properties. The following section explores possible statistical associations using correlation coefficient metric.

In the analysis, the range of results for the gamma m (shape parameter) was between 0.1 and 12.9. This range corresponds to PRF of 49 $[\frac{(ft^3/s) \times h}{mi^2 \times in}]$ to 919 $[\frac{(ft^3/s) \times h}{mi^2 \times in}]$. Also, the range of results for t_{infl} was from 0.4 [hr] to 57.6 [hr] across 100 events in 54 watersheds located in two regions.

3.4.1 Correlation Coefficient Estimation

In Table 3.3, correlation coefficient (R), is illustrated among 8 predictor variables and 3 response variables. Predictor variables are, DA=drainage area [mi²], L=watershed length [mi], CS=channel slope [ft/mi], LS=land slope [ft/ft], FOR=percent (by area) forest, IA=percent (by area) impervious, ST=percent of area representing storage (lakes, ponds, wetlands), and DRO=direct runoff [in]. Response variables are m =Gamma UH m parame-

ter, t_{peak} =time to peak of the UH, t_{infl} =time from end of the rainfall to recession inflection point of the UH, used to estimate time of concentration. The data for these variables were reported in [Brubaker et al., 2021]. The strongest correlation between response variables and predictor variables is observed between t_{infl} and channel slope which is approximately -50%. That indicates the channel slope is negatively correlated with t_{infl} and that is considered a moderate correlation. Response variables t_{infl} and t_{peak} are strongly correlated with each other. Moreover, t_{peak} also has negative correlation with channel slope amounting to approximately -48%. Land slope shows a moderate negative correlation with t_{infl} , -0.41. Among predictor variables a trend of intercorrelation is observed and that indicates stepwise regression might be more accurate to establish a regression equation among predictor variables and 3 response variables.

Table 3.3: Correlation Coefficient

	DA	L	CS	LS	FOR	IA	ST	DRO	m	t_{peak}	t_{infl}
DA	1.00	0.92	-0.34	0.30	0.30	-0.12	-0.19	0.11	0.10	0.34	0.33
L	0.92	1.00	-0.40	0.39	0.29	-0.14	-0.23	0.11	0.08	0.26	0.25
CS	-0.34	-0.40	1.00	0.40	0.19	0.27	-0.17	-0.19	-0.04	-0.48	-0.50
LS	0.30	0.39	0.40	1.00	0.37	0.01	-0.27	-0.16	0.07	-0.38	-0.41
FOR	0.30	0.29	0.19	0.37	1.00	-0.44	-0.09	-0.05	-0.05	0.32	0.32
IA	-0.12	-0.14	0.27	0.01	-0.44	1.00	-0.17	0.04	0.09	-0.28	-0.29
ST	-0.19	-0.23	-0.17	-0.27	-0.09	-0.17	1.00	-0.05	0.00	0.11	0.11
DRO	0.11	0.11	-0.19	-0.16	-0.05	0.04	-0.05	1.00	0.10	0.23	0.24
m	0.10	0.08	-0.04	0.07	-0.05	0.09	0.00	0.10	1.00	0.21	0.12
t_{peak}	0.34	0.26	-0.48	-0.38	0.32	-0.28	0.11	0.23	0.21	1.00	0.99
t_{infl}	0.33	0.25	-0.50	-0.41	0.32	-0.29	0.11	0.24	0.12	0.99	1.00

3.5 Relation Between Time to Inflection of Unit Hydrograph and Event Time of Concentration

In Figure 3.10, the t_c estimated from the event hydrographs (Section 3.3.4) is compared to t_{infl} computed from the gamma function of the unit hydrograph. The values computed mathematically from the unit hydrograph, are larger in magnitude than those estimated from the event hydrograph. R^2 (i.e., coefficient of determination) values which are 66% and 69.8% indicate a moderate linear relationship between time to inflection of unit hydrograph and event time of concentration for total and direct hydrographs.

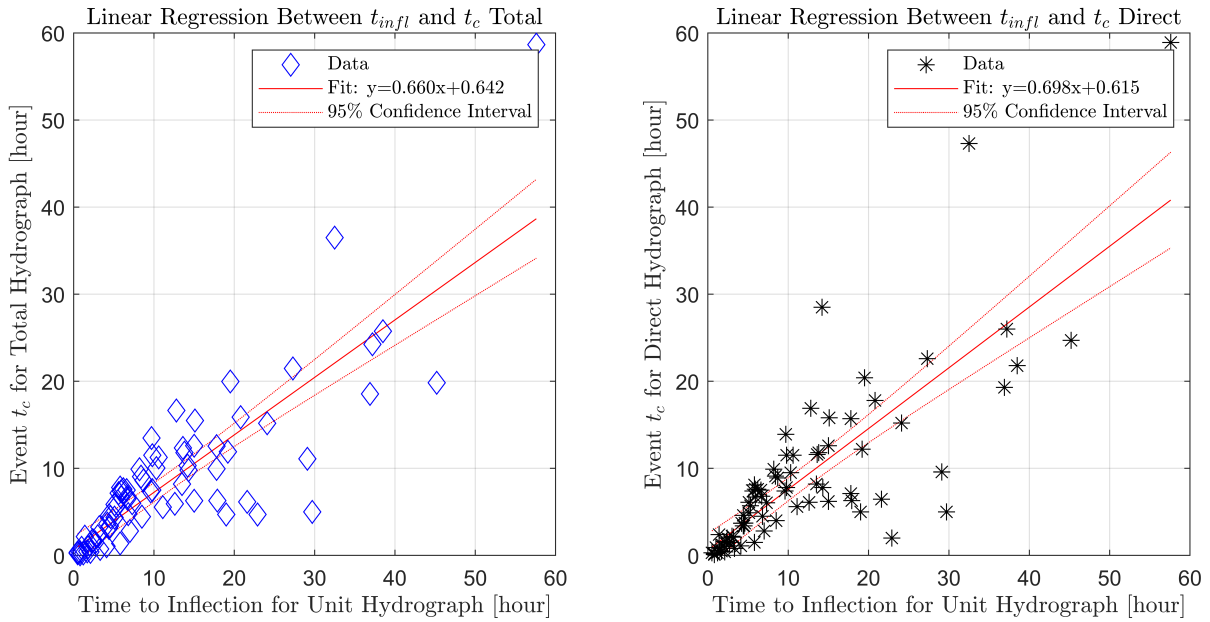


Figure 3.10: Regression Between Time to Inflection of Unit Hydrograph and Event Time of Concentration for Total and Direct Hydrograph

The t-test is significant and null hypothesis is rejected at $\alpha=0.05$ (confidence interval 95%) and the linear relationship exists among time to inflection of the unit hydrograph and event

time of concentration for total and direct hydrographs (Figure 3.10).

3.6 Concluding Remarks

For 100 events at 54 stations in Maryland, a gamma unit hydrograph was computed. The m parameter and the time to peak of the gamma unit hydrograph were selected as the main characteristics of the unit hydrograph. 18 events were not considered due to unsound results or poor fitting. This is defined by observation.

The results suggest that the form of the unit hydrograph can differ among different watersheds, and may even differ between individual events. An initial assessment also showed that there is no consistent connection between event size (measured by direct runoff depth) and m . There were not enough data about events available to explore how m changes with event size for a specific watershed.

This research chose the time to inflection (t_{infl}) of the unit hydrograph as the response time variable to represent time of concentration in watersheds. It is defined as the time from the end of the rainfall excess to the mathematical inflection point on the dimensional gamma unit hydrograph. There is no subjectivity in this process as the equation of gamma UH was used to derive this time variable.

In this research, t_{infl} was analyzed as a substitute for time of concentration, t_c . There is a large degree of subjectivity in inferring the t_c from an event hydrograph. The literature refers to an “inflection point” on the event hydrograph receding limb, but that point is frequently interpreted as a slope change, instead of the mathematical definition of “change in sign of second derivative”. This research attempted to produce an objective method to

estimate t_c as the time from end of rainfall excess to the inflection point on the event hydrograph recession limb. This procedure was only partially successful, since in multiple cases the inflection point took place before the end of rainfall excess, leading to negative t_c which is meaningless from hydrologic standpoint. Computing and mathematically analyzing the unit hydrograph is more robust.

The computation of event t_c , t_{infl} , t_{peak} , and m are subject to multiple sources of uncertainty. These uncertainties include: 1) bias in the NEXRAD precipitation data; 2) errors produced by the baseflow separation method; 3) errors produced by the simplicity of the procedure in the phi-index precipitation excess separation procedure and the timing of precipitation excess of the data sources used. This research has the most confidence in USGS streamflow measurement. Also, for the larger watersheds during the longer storm event connected to low probability discharge, the assumption of uniform rainfall over the basin is questionable.

Although the NRCS velocity method, which applies aggregate of travel time thorough segments in the watershed, is broadly applied in practice to calculate t_c , that method is not included in this research. The author based this decision on the judgement that the velocity method is a tool to estimate watershed t_c , not a measurement or a fundamental definition of watershed t_c . Although much of the hydrologic literature considers the velocity method as a measurement or definition, this assumption is being questioned broadly in the last decade [Beven, 2020; Grimaldi et al., 2012].

This research produced analytical tools to derive a watershed-average hyetograph at fine temporal resolution using GIS and radar data. Using these data, this research was released from the difficulties of obtaining hyetographs from remote and sparse precipitation

gauges. An independent daily precipitation data source was used to rectify the unquantified bias in the radar fields. Overall, the daily precipitation pattern was generally consistent.

In the course of calculating watershed average precipitation from the NEXRAD DPR product, this research also quantifies the spatial variance of precipitation over the watershed; however, the spatial variance of precipitation was not applied in this study. The UH approach assumes the rainfall is uniform over the watershed; this is a major simplifying assumption.

In the results, the convolved event hydrographs are sensitive to the UH shape and therefore gamma m (or PRF). Does this indicate the UH shape parameter m (or PRF) could be considered as a tuning variable in hydrologic modeling? Analysts should be aware that both t_{peak} and m are required to generate the dimensional UH. The NRCS method assumes a ratio of t_{peak} to t_c . If t_{infl} is accepted as a surrogate for t_c , then based on the results of this research, the ratio of t_{peak} to t_c is not constant, but varies with the m parameter. Hence, if a different approach is used to estimate t_c and m is adjusted as a tuning parameter, an appropriate scaling factor should be used to derive t_{peak} for calculating the UH.

This research put substantial effort in producing approaches to generate watershed event hyetographs from radar rainfall data. It should be noted that the procedures applied in separating baseflow from total runoff hydrographs and precipitation excess from hyetograph are overly simplified treatments of the physical properties of the watersheds. Some events were eliminated from the results, since direct runoff derived from baseflow separation was larger than total precipitation for that event; this was predominantly observed in a number of complex precipitation events. Some of these cases were identified when the algorithm generated a negative phi-index, which does not carry any physical meaning. Fu-

ture work can focus on the applying more advanced hydrograph and hyetograph separation procedures that are theoretically accurate, physically sound while still satisfying the UH assumptions, and mutually consistent, with minimal subjectivity.

A number of the events selected for analysis were associated with Hurricane Sandy in October 2012. The dissertation author designed a focused study comparing PRF for different watersheds in response to the same meteorological event in Chapter 4 of this dissertation. This study was inspired by [\[Horst and Gurriell, 2019\]](#), who investigated PRF for New Jersey watersheds in response to Hurricane Irene.

For many of the study watersheds, only a single event was identified within the constraints on this study. Therefore, the results were insufficient to investigate the assumption that the UH is a fixed property of a watershed. For the study watersheds with multiple events, the UHs were different but there were not enough events to draw any empirical conclusions about what might cause these differences. This motivated the idea of building a detailed distributed model of the Jones Falls watershed, as a numerical laboratory to investigate the physical processes that affect the UH. This was the motivation for Chapter 5 of this dissertation.

Chapter 4: Peak Rate Factor Estimation in Response of Selected Maryland and Delaware Watersheds to Hurricane Sandy

4.1 Introduction

The peak stream or river discharge during a flood event is an essential quantity in hydrologic and hydraulic design. One of the most practical ways to estimate peak discharge is to predict the runoff hydrograph by convolution of the associated precipitation time series with the watershed's unit hydrograph (UH). The UH represents the time-varying runoff response [m^3/s] or [ft^3/s] to a unit [cm or in] of precipitation excess over a specified time period, assumed to occur uniformly in time and space. Where concurrent precipitation and runoff data are available, the watershed UH can be identified. For ungauged watersheds, practitioners typically apply a synthetic UH approach, in which conceptually- and empirically-developed dimensionless unit hydrograph (DUH) models [Clark, 1945; Mockus, 1957; Snyder, 1938] are scaled using watershed properties [Sheridan et al., 2002].

The Natural Resources Conservation Service (NRCS) curvilinear DUH developed by [Mockus, 1957] is among the most popular unit hydrograph models and is the standard for state agencies in Maryland and other states. Converting the NRCS DUH to a watershed UH requires three parameters: PRF, time of concentration, and watershed area.

The default value of PRF in NRCS applications (e.g., WinTR-20) is 484 (in U.S. customary system units). This value was computed empirically by considering watersheds in the Midwest United States [Lasco et al., 2022]. An alternative DUH with a PRF of 284 was developed for the Delmarva (Delaware, Maryland, and Virginia) Peninsula [Welle and Woodward, 1989]. Several decades of study [Horst and Gurriell, 2019; McCuen and Bondelid, 1983; Meadows and Blandford, 1983; Sheridan et al., 2002] have shown that PRF values can vary among watersheds and even among events at a given watershed. The NRCS has suggested that other values may be applicable, and provides DUH tables for a range of PRF from 100 to 800 [Natural Resources Conservation Service (NRCS), 2007]. In a technical report published by the Texas Department of Transportation, Sharif [2019] discusses the possible variation of the PRF and makes provisions. Lasco et al. [2022] found that for watersheds in Houston, TX, a PRF lower than the default value of 484 might be a more suitable choice.

If the PRF can vary, how can an appropriate value be selected or assigned? In a case study of New Jersey watersheds during Hurricane Irene, Horst and Gurriell [2019] found no local or regional trends in PRF and no clear link between watershed characteristics and PRF. The study reported here investigates the PRF for 20 watersheds in Maryland during Hurricane Sandy in Oct. 2012, with the goal of providing insight on selection of appropriate PRF values in Maryland.

The following sections provide a summary of the DUH and UH concepts and a description of the locations and methods applied to estimate PRF. The results are presented in Section 4.5 and findings are summarized in Section 4.8.

4.2 Background

The NRCS DUH was initially developed by [Mockus, 1957] by analysis of small gauged watersheds. It is provided as a table of dimensionless discharge (q^*) as a function of dimensionless time (t^*). It is not described by an explicit mathematical equation. The DUH is transformed to a UH by multiplying the abscissa and ordinates. Therefore, $t = t^* \times t_p$ and $q = q^* \times q_p$, where t_p is the time to peak [hr] measured from the beginning of precipitation excess, and q_p is the peak discharge. According to [Natural Resources Conservation Service (NRCS), 2007] peak flow is calculated as follows:

$$q_p = PRF \frac{AQ}{t_p} \quad (4.1)$$

where PRF is the peak rate factor, A is watershed area [mi^2], and Q is total runoff depth, equal to 1 inch for a UH. The PRF is both a unit conversion and a scaling factor between the DUH and the dimensional UH. In US customary system units, the PRF is calculated from the DUH using [Natural Resources Conservation Service (NRCS), 2007]

$$PRF = \frac{645.33}{\sum DUH_{coordinates} \times \Delta T_{DUH}} \quad (4.2)$$

where $DUH_{coordinate}$ are the q^* ordinates of the DUH, ΔT_{DUH} is the nondimensional time interval of the DUH, and 645.33 establishes the units. The denominator in the Equation 4.2 is a discrete approximation to the integral:

$$\int_0^{t_{max}^*} q^*(t^*) dt^* \quad (4.3)$$

The discrete approximation is necessary because of the tabular definition of the DUH. Applying Equation 4.2 to the standard NRCS DUH produces the default PRF of 484 [ft³/s] per [mi²] per inch of precipitation. The alternative PRF for Delmarva region, also calculated by discrete integration of a tabular function, is 284 [Welle and Woodward, 1989].

The [Natural Resources Conservation Service \(NRCS\) \[2007\]](#) proposed a gamma function form as a generalization of their DUH:

$$\frac{q}{q_p} = e^m \left[\left(\frac{t}{t_p} \right)^m \right] \left[e^{(-m(\frac{t}{t_p}))} \right] \quad (4.4)$$

where m is the parameter of the gamma function. With a functional expression for the DUH, Equation 4.2 can be rewritten as follows:

$$PRF = \frac{645.33}{\int_0^{t_{*,max}} q_*(t_*) dt_*} \quad (4.5)$$

The analytical solution to the integral in the denominator of Equation 4.5 is available for any value of m . Equation 4.5 represents a generalization of the PRF values 484 and 284, for the original DUH and the Delmarva region respectively. This generalization provides a satisfactory approximation to the empirically-derived NRCS DUH, given $m = 3.7$ and $m = 1.37$, respectively. The [Natural Resources Conservation Service \(NRCS\) \[2007\]](#) provides tabular versions of the DUH using this equation for selected PRF values; the function can be evaluated directly for any value of m .

4.3 Study Event, Location, and Data

Following [\[Horst and Gurriell, 2019\]](#), this study analyzes discharge from multiple watersheds during a single event, Hurricane Sandy in October 2012.

4.3.1 Hurricane Sandy

Sandy, a quintessential late-season hurricane, formed in the southwestern Caribbean Sea. The hurricane first made landfall in Jamaica as a Category 1 hurricane, per the Saffir-Simpson Hurricane Wind Scale, followed by striking eastern Cuba as a powerful Category 3 hurricane with wind speeds reaching 100 knots. After its encounter with Cuba, it underwent a reduction in intensity to a Category 1 hurricane as it progressed through the central and northwestern Bahamas [\[Blake et al., 2013\]](#).

An intriguing transformation was observed in Sandy’s characteristics over the Bahamas. Despite a decrease in intensity, converting it to a tropical storm, Sandy expanded considerably in size. The system then regained its status as a hurricane as it pursued a northeastward course parallel to the southeastern United States coastline, reaching a secondary peak intensity of 85 knots while it changed its direction northwestward towards the mid-Atlantic states [\[Blake et al., 2013\]](#).

Sandy underwent another weakening phase before making landfall near Brigantine, New Jersey, as a post-tropical cyclone with maximum sustained winds of 70 knots. However, its considerable size precipitated a disastrous storm surge that severely impacted the coastlines of New Jersey and New York [\[Blake et al., 2013\]](#).

Preliminary damage assessments in the U.S. approximate \$50 billion, positioning

Sandy as the second most financially destructive cyclone to hit the United States since 1900. The hurricane was responsible for at least 147 direct fatalities across the Atlantic basin, 72 of which occurred in the mid-Atlantic and northeastern United States. This is the highest recorded number of U.S. direct fatalities caused by a tropical cyclone outside of the southern states since Hurricane Agnes in 1972 [Blake et al., 2013]. The storm rainfall depth is mapped in Figure 4.1. The maximum of 12.83 in occurred over the Eastern Shore of Maryland. Much of the Coastal Plains region received 7 to 10 inches of rainfall (dark red), while Maryland's Piedmont province was in the 5-7 inch area (red).

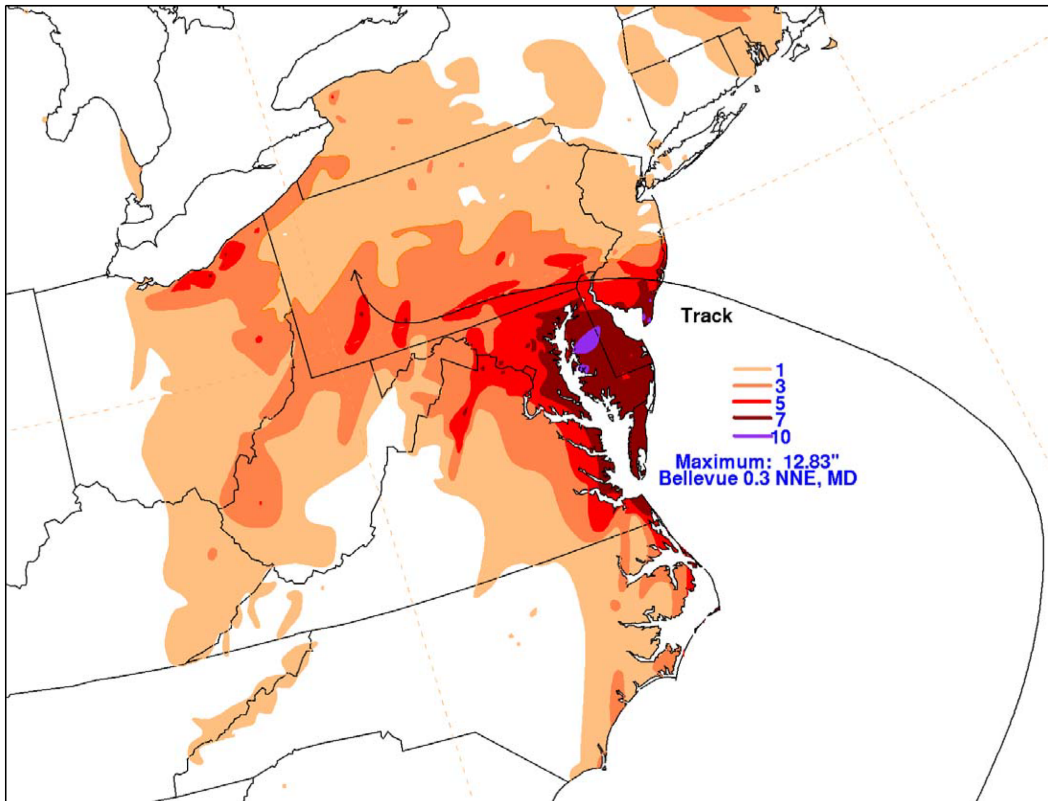


Figure 4.1: Rainfall (in inches) associated with Hurricane Sandy and its extratropical remnants from 27-31 October 2012. Figure courtesy of the Hydrometeorological Prediction Center [Blake et al., 2013]

4.3.2 Watersheds

Stream gauge stations in Maryland were selected from the USGS database at [U.S. Geological Survey (USGS), n.d.]. Selected watersheds were limited in size to 100 [mi²] (259 [km²]) as large watersheds are not consistent with NRCS UH approach. A total of 20 gauging stations in two regions of Maryland were selected: 9 from Maryland’s Piedmont, Blue Ridge Mountains and Valleys, and Appalachian Plateau physiographic provinces (Region 1) and 11 from the Western and Eastern Coastal Plain provinces (Region 2). Figure 4.2 depicts the locations and physiographic regions for the selected watersheds indexed from 1 to 20. The index table associated with station IDs is provided in Table 4.1.

Table 4.2 provides some characteristics of the watersheds. The sixth column estimates the annual exceedance probability (AEP) of the peak discharge resulting from Hurricane Sandy as follows:

$$AEP = \frac{1}{T} \quad (4.6)$$

where T is return period. AEP estimates were derived from the Maryland Hydrology Panel [Maryland Hydrology Panel, 2020].

Table 4.1: Watersheds Index and Station IDs

Index	Station ID	Index	Station ID	Index	Station ID	Index	Station ID
1	1599000	6	1591700	11	1649500	16	1491500
2	1617800	7	1594000	12	1594526	17	1484100
3	1637500	8	1589300	13	1653600	18	1486000
4	1639140	9	1589440	14	1660920	19	1485500
5	1591000	10	1651000	15	1589500	20	148471320

Index of Selected Watersheds in Maryland

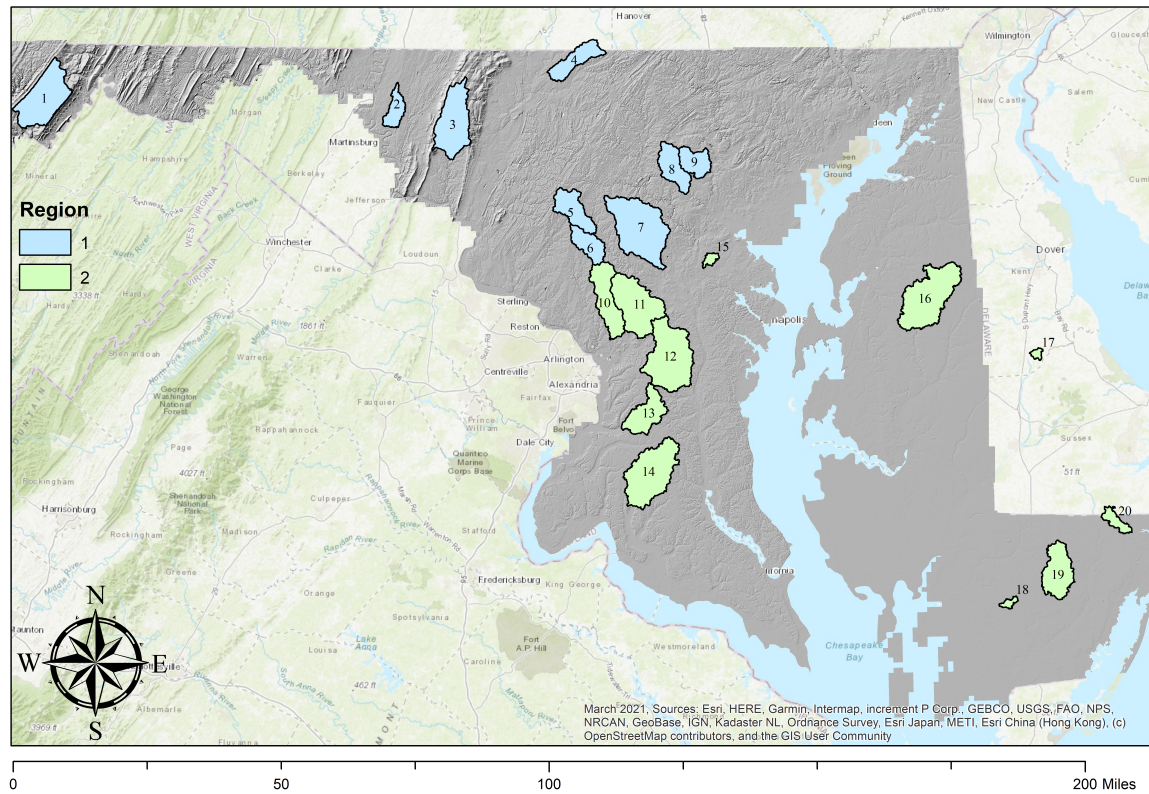


Figure 4.2: Locations of Selected Watersheds for Hurricane Sandy Analysis

4.3.3 Event Hydrograph Data

Fifteen-minute stream gauge data were downloaded from U.S. Geological Survey's (USGS) National Water Information System [[U.S. Geological Survey \(USGS\)](#), n.d.].

4.3.4 Event Hyetograph Data

Precipitation data to identify the UH and calculate the PRF should be at sub-hourly rates. Very few sub-hourly gauges exist in Maryland. This study used spatially distributed data from the WSR-88 Radar (NEXRAD) system.

Table 4.2: Maryland Watersheds Characteristics with Index

Index	Site Name	Region	USGS ID	Area [mi ²]	Q_p [cfs]	Approx. AEP	DPR [in]	GridMET [in]	DRO [in]
1	Georges Creek	1	01599000	72.4	2680	>0.2 & <0.5	2.42	3.69	0.86
2	Marsh Run	1	01617800	18.9	121	>0.2 & <0.5	6.88	4.15	0.15
3	Catoctin Creek	1	01637500	66.9	8330	>0.04 & <0.1	6.20	5.06	2.95
4	Piney Creek	1	01639140	31.3	6150	>0.02 & <0.04	6.89	5.07	3.39
5	Patuxent River	1	01591000	34.8	3310	>0.2 & <0.5	5.24	6.34	2.09
6	Hawlings River	1	01591700	27	2180	>0.2 & <0.5	5.74	5.93	1.75
7	Little Patuxent River	1	01594000	98.4	10600	>0.04 & <0.1	5.89	5.94	2.76
8	Gwynns Falls	1	01589300	32.5	4060	>0.1 & <0.2	6.62	6.31	2.61
9	Jones Falls	1	01589440	25.2	3510	>0.1 & <0.2	7.26	6.73	2.86
10	Northwest Branch	2	01651000	49.4	5770	>0.2 & <0.5	6.10	5.3	2.88
11	Northeast Branch	2	01649500	72.8	4750	>0.2 & <0.5	6.15	5.46	1.86
12	Western Branch	2	01594526	89.7	4100	>0.2 & <0.5	6.56	6.91	1.91
13	Piscataway Creek	2	01653600	39.5	1630	>0.2 & <0.5	6.11	6.63	1.35
14	Zekiah Swamp Run	2	01660920	79.9	2620	>0.1 & <0.2	6.17	7.05	0.8
15	Sawmill Creek	2	01589500	4.97	240	>0.02 & <0.04	6.74	6.91	1.32
16	Tuckahoe Creek	2	01491500	85.2	4940	>0.04 & <0.1	11.03	9.02	2.64
17	Beaverdam Branch	2	01484100	3.02	136	>0.02 & <0.04	9.98	8.87	1.20
18	Manokin Branch	2	01486000	4.8	288	>0.1 & <0.2	8.47	8.51	1.13
19	Nassawango Creek	2	01485500	44.9	1300	>0.1 & <0.2	5.65	8.45	0.72
20	Birch Branch	2	0148471320	6.38	1040	—	4.00	8.77	4.75

Archived digital precipitation rate (DPR) data were obtained from the NCEI online data server [[National Oceanic and Atmospheric Administration \(NOAA\)](#), n.d.]. The DPR data used in this study are Level III, which indicates the data have been processed from the raw reflectivity signal to a gridded precipitation intensity. DPR data were selected because they provide precipitation intensity at sub-hourly time intervals. Additionally, the DPR data are available from 2012 to present, which includes the date of Hurricane Sandy in October 2012. This research used data from WSR-88D radar at Sterling, Virginia (KLWX) since the range of the radar data covers the entire state (Figure 4.3). The grid elements are 1° by 250 m (radial distance).

For a given time, t , average rainfall depth over a watershed is computed by integrating spatially-varying precipitation depth, $p(\mathbf{x}, t)$:

$$\bar{p}(t) = \frac{1}{A} \int p(\mathbf{x}, t) dA \quad (4.7)$$

where $\mathbf{x}$ refers to a particular (angle, radius) location on the grid and dA is the (spatially variable) area corresponding to each grid element's spatial intersection with the watershed.

The NEXRAD DPR data are finely resolved in space and time; however, radar data are subject to bias. An independent source of precipitation data was used to adjust the watershed average time series.

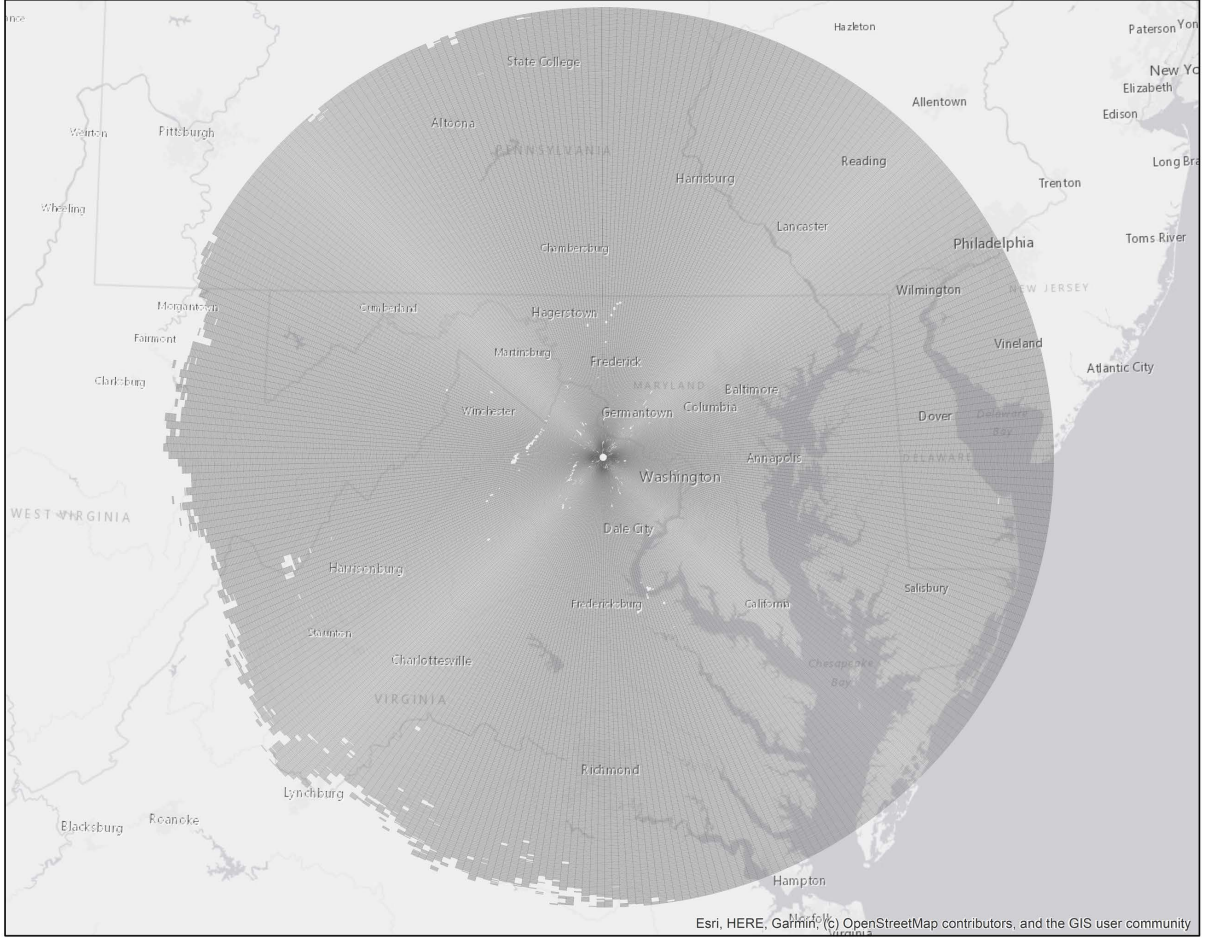


Figure 4.3: NEXRAD DPR image showing precipitation over the entire state of Maryland at a particular time during Hurricane Sandy (October 2012). The image illustrates the radial grid centered on the Sterling, VA, Each grid element is $1^{\circ} \times 250$ m.

For this purpose, daily precipitation data were obtained from the Climate Engine server [Climate Engine, n.d.]. Notably, gridMET data is the mixture of spatial attributes of gridded climate data from PRISM and from regional reanalysis (NLDAS-2) applying climatic interpolation. Based on the gridMET webpage [Climate Engine, n.d.], “The resulting product is a spatially and temporally complete, high-resolution ($1/24^{\text{th}}$ degree $\sim$ 4-km) gridded dataset of surface meteorological variables”. Climate Engine allows user to

download daily watershed average gridMET precipitation by uploading a shapefile outline of the watershed. Using a MATLAB[®] script, the fine-scale time series (at irregular time intervals) are accumulated and adjusted on a daily basis to match the daily watershed totals from gridMET. The corrected accumulated watershed average precipitation is disaggregated to a regular time interval of 0.1 hour (6 minutes). These steps are illustrated in Figure 4.4.

The result of the aforementioned steps is an event hyetograph of watershed average precipitation at 6-minute (0.1-hour) intervals for the study watershed. The watershed average precipitation as determined by integration of the DPR (Equation 4.7) and after Gridmet correction are tabulated in columns 7 of Table 4.2. Event precipitation was corrected to the Gridmet value before further analysis.

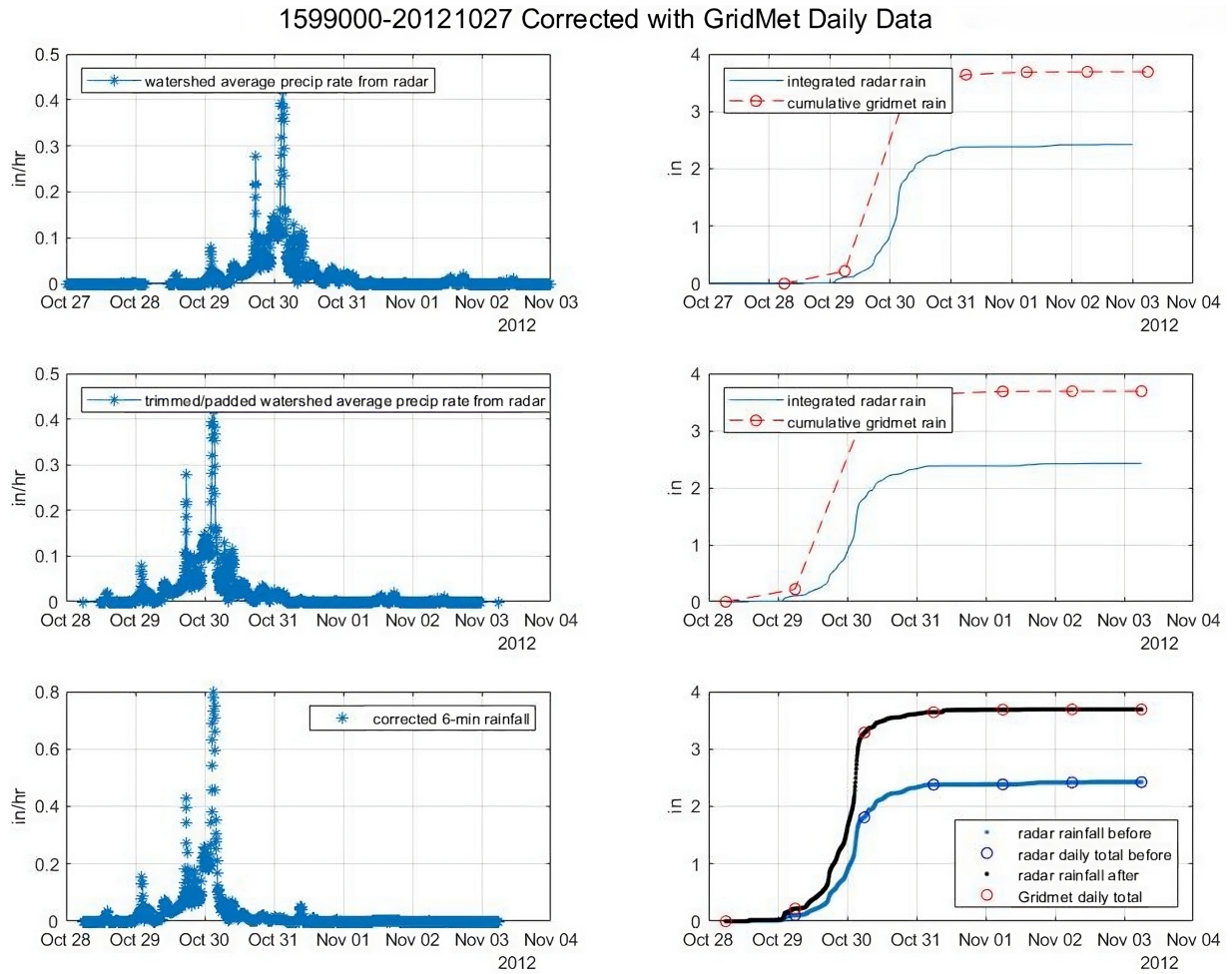


Figure 4.4: Adjusting watershed-average radar rainfall using gridMET watershed average

4.4 Methods

This section presents the methods used to estimate the DUH and calculate PRF for each watershed.

4.4.1 Baseflow Separation and Direct Runoff Estimation

The UH converts direct runoff to precipitation excess; therefore, baseflow was removed to obtain a time series of direct runoff. Several methods exist in the literature to separate baseflow from a hydrograph; many of these methods require the identification of an “inflection point” on the receding limb of the hydrograph, believed to indicate the end of the direct runoff and the beginning of the pure baseflow. In many cases, the visual inflection points do not match the mathematical definition and their identification is subjective. This study applied an automated algorithm to identify baseflow recession, aiming to reduce the level of subjectivity.

The baseflow separation algorithm is a modified version of the concave method. The original concave method was explained in detail by [McCuen \[2016\]](#). First, it is assumed that pure baseflow follows an exponential decay function. The algorithm traces the recession curve and identifies the time after which the hydrograph best fits an exponential decay; during this period of time, all discharge is baseflow. The exponential curve is extended backward to the time at which event rainfall ceased (t_{end}); during this period of time, baseflow is less than total discharge. Next, pre-event discharge is assumed to be pure baseflow, and baseflow is assumed to rise linearly when event runoff begins. A line is constructed from the time of rise to the t_{end} point on the extended exponential curve; during this period of time, baseflow is less than total discharge. (It was occasionally necessary to continue pre-event baseflow for a short period of time before starting the upward linear segment so that calculated baseflow did not exceed total discharge.) Direct runoff was calculated by subtracting the constructed baseflow time series from the observed discharge

time series. After removing the baseflow, the total depth of direct runoff was calculated by numerical integration. Figure 4.5 displays an example of the baseflow separation. The calculated direct runoff for each watershed appears in the final column of Table 4.2.

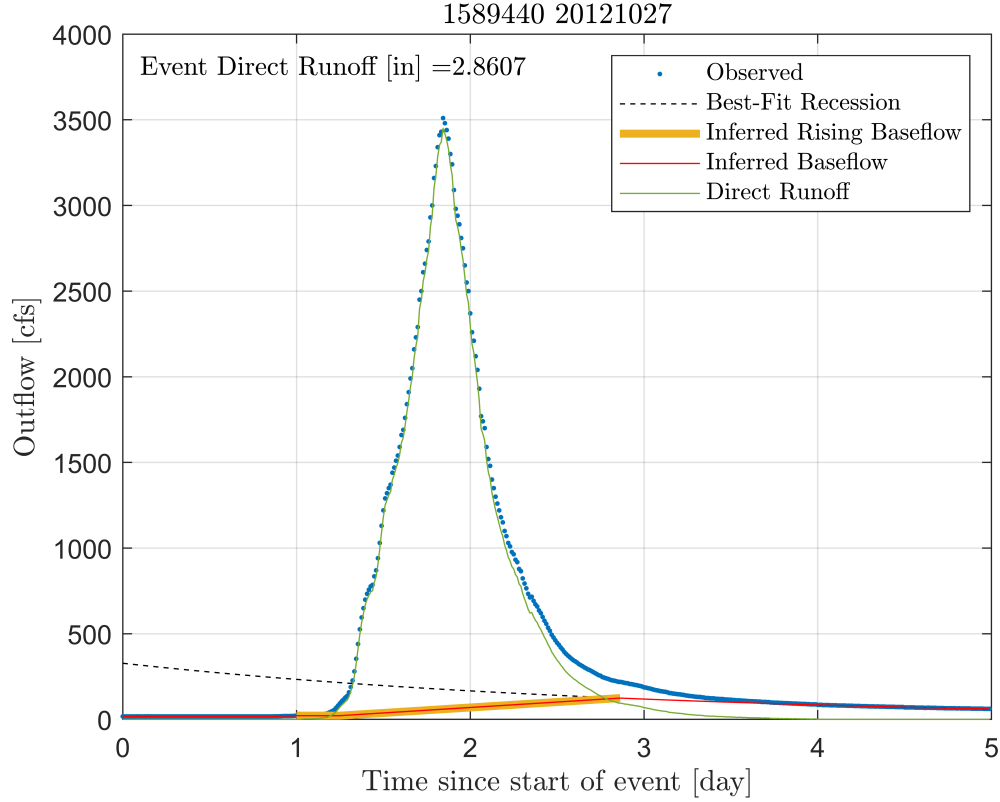


Figure 4.5: Example of Baseflow Separation for Direct Runoff Estimation

4.4.2 Precipitation Excess Estimation

The UH method assumes that the depth of direct runoff equals the depth of excess precipitation. The rainfall excess time series was separated from the event hyetograph using a Phi-index approach. The Phi-index method applies a constant potential loss rate to the event. The actual loss during each time increment is equal to the potential value or the

rainfall rate, whichever is less. An automated solver method is applied to find the value of ϕ for which integrated precipitation excess equals the event direct runoff depth. Figure 4.6 shows the ϕ -index result for the event illustrated in Figure 4.5.

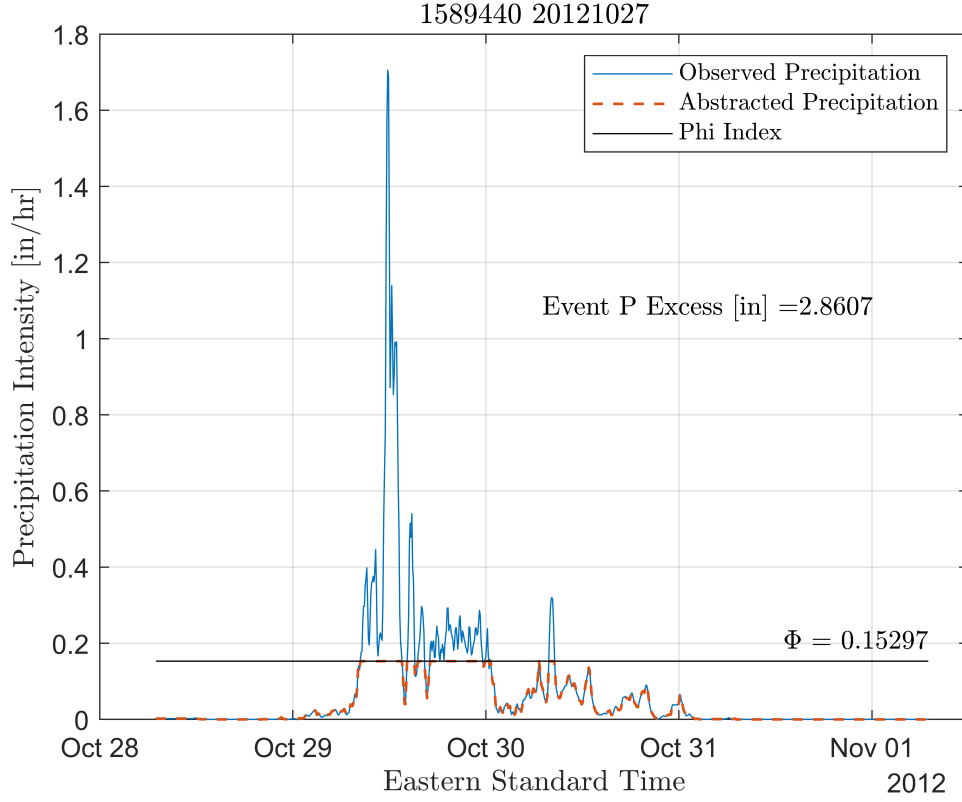


Figure 4.6: Example of Rainfall Excess Estimation for NRCS Unit Hydrograph Analysis

4.4.3 Identification of Unit Hydrograph

The gamma unit hydrograph approach (Equation 4.4) was implemented to transform the precipitation excess to direct runoff in all the selected watersheds.

For a given direct runoff and precipitation excess, the aim is to find the best gamma UH that translates observed precipitation excess to observed direct runoff. The gamma

parameter m was used rather than PRF since m is dimensionless and is more fundamental from the mathematical standpoint. Accordingly, the m parameter can be converted to its equivalent PRF in US Customary Unit System. The (m, t_p) space was systematically searched to determine the combination of parameter values that minimized the Sum of Squared Errors (SSE), where error is defined as the difference between the observed direct runoff hydrograph and the convolution of precipitation excess with the proposed UH. For each candidate (m, t_p) pair, the DUH was numerically integrated (Equation 4.2), q_p was calculated by Equation 4.1, the DUH was converted to a dimensional UH, the precipitation excess was convolved with the candidate UH and the resulting predicted direct runoff hydrograph was quantitatively compared to the observed. An example of a best-fit convolved direct runoff hydrograph is shown in Figure 4.7.

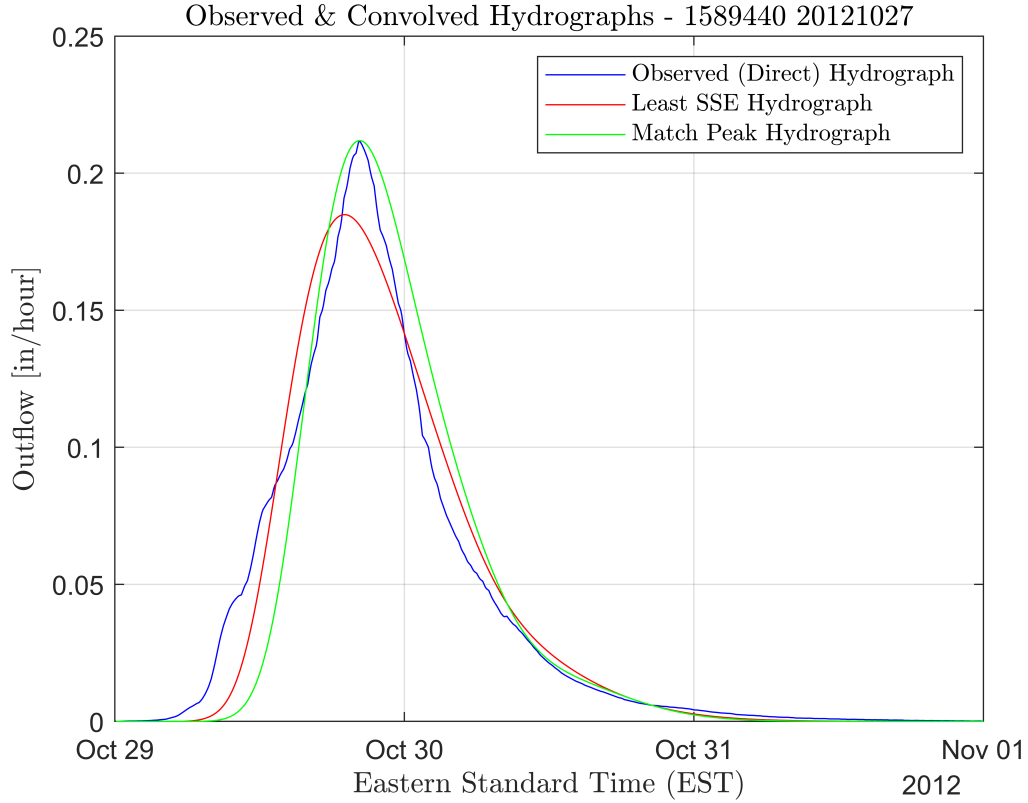


Figure 4.7: Example of Observed & Convolved Hydrographs

4.5 Results

The PRFs range from 49 to 848, as reported in Table 4.3. The overall average for the PRF for both regions including all 20 watersheds is 570. The average PRF in region 1 is 557 and the average PRF in region 2 is 580. Upon the subdivision of Region 2 into two distinct areas, namely the Western Coastal Plain and Eastern Coastal Plain (Delmarva), certain specific watersheds are found within each area. The Western Coastal Plain encompasses Sawmill Creek (1589500), Northwest Branch Anacostia (1651000), Northeast Branch Anacostia (1649500), Western Branch (1594526), Piscataway Creek (1653600), and

Zekiah Swamp Run (1660920). Conversely, the Eastern Coastal Plain includes Tuckahoe Creek (1491500), Beaverdam Branch (1484100), Manokin Branch (1486000), Nassawango Creek (1485500), and Birch Branch (148471320). The average PRF for the Western region in Region 2 is approximately 640, whereas the Eastern region in Region 2 exhibits a lower average PRF, approximately 508. Figure 4.8 is a bar graph of the shape parameter, m , and scale parameter, PRF, of the selected watersheds in Maryland for Hurricane Sandy event.

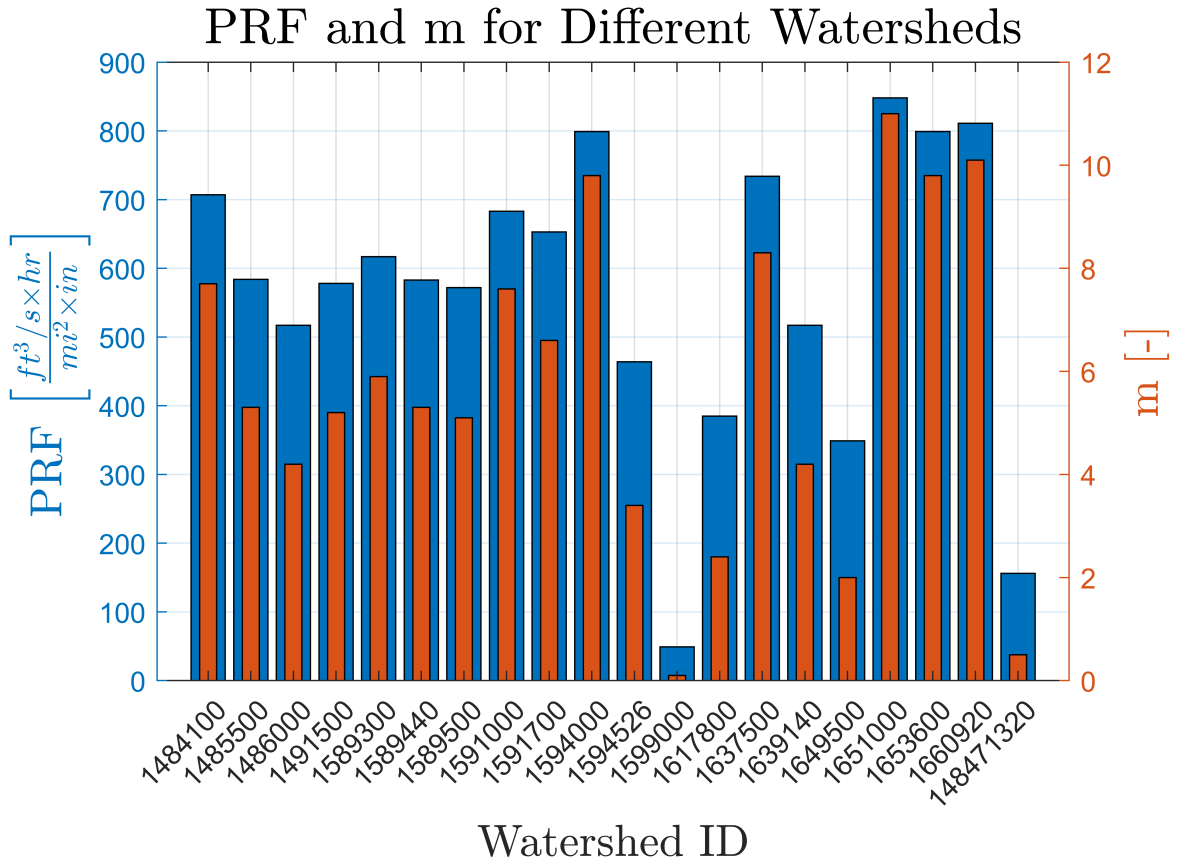


Figure 4.8: Distribution of m and PRF for Selected Watersheds in Maryland and Delaware

Table 4.3: Peak Rate Factor for Watersheds' Response to Hurricane Sandy

Watershed ID	m	PRF	Relative RMSE	Relative Bias
1484100	7.7	707	0.2666	-0.0013
1485500	5.3	584	0.1006	-0.0014
1486000	4.2	517	0.1668	0.000251
1491500	5.2	578	0.0874	0.000873
1589300	5.9	617	0.2935	-0.0157
1589440	5.3	583	0.1977	-9.25E-05
1589500	5.1	572	0.1254	-0.000347
1591000	7.6	683	0.2023	0.000164
1591700	6.6	653	0.172	-0.0045
1594000	9.8	799	0.083	-1.90E-05
1594526	3.4	464	0.1113	-7.64E-06
1599000	0.1	49	0.7521	0.0073
1617800	2.4	385	0.1036	-0.0109
1637500	8.3	734	0.1978	-4.74E-05
1639140	4.2	517	0.3323	-0.0729
1649500	2.0	349	0.205	-0.000172
1651000	11.0	848	0.317	-4.83E-05
1653600	9.8	799	0.1875	-0.000664
1660920	10.1	811	0.2885	-0.0011
148471320	0.5	156	0.2625	-0.00031

4.5.1 Evaluation of Unit Hydrograph Identification

Table 4.3 includes two goodness-of-fit measures describing the match between observed direct runoff and the direct runoff predicted by convolution of precipitation excess with the selected gamma UH. Relative root mean square error is root mean square error (RMSE or S_e) divided by standard deviation (S_y). Therefore, relative RMSE (S_e/S_y) is defined as:

$$\text{Relative RMSE} = \frac{\sqrt{\frac{1}{n-1} \sum_{i=1}^n (y_i - \hat{y}_i)^2}}{\sqrt{\frac{1}{n-1} \sum_{i=1}^n (y_i - \bar{y})^2}} \quad (4.8)$$

where y_i is actual value, $\hat{y}_i$ is predicted value, $\bar{y}$ is sample mean and n is sample size. Values between 0.1 and 0.5 (10% to 50%) are indicative of a good model [McCuen, 2018]. Relative bias (e/y) is defined as:

$$\text{Relative Bias} = \frac{\sum_{i=1}^n (y_{\text{predicted},i} - y_{\text{actual},i})}{\sum_{i=1}^n y_{\text{actual},i}} \quad (4.9)$$

an absolute value of relative bias less than 0.1 (10%) is indicative of a good model [McCuen, 2018].

In Figure 4.9 relative RMSE and relative bias are illustrated. In the relative RMSE bar plots, a fairly consistent trend can be observed ranging from 0.1 to 0.75, which indicates the good fitting between the observed and fitted hydrographs. The high RMSE for watershed ID 1599000, which is 0.75, indicates fitting errors are larger in this watershed than in others. This watershed has a low value of 49 for PRF and it has a fairly large drainage area of 72.4 [mi²]. Based on the trend observed in the fitted hydrographs, the smaller watersheds often produce a better fit than the large ones. The event hydrograph for watershed ID 1599000 has an abrupt surge on the rising limb and an abrupt decline on the recession limb resulting in a shape that is not well matched by UH convolution.

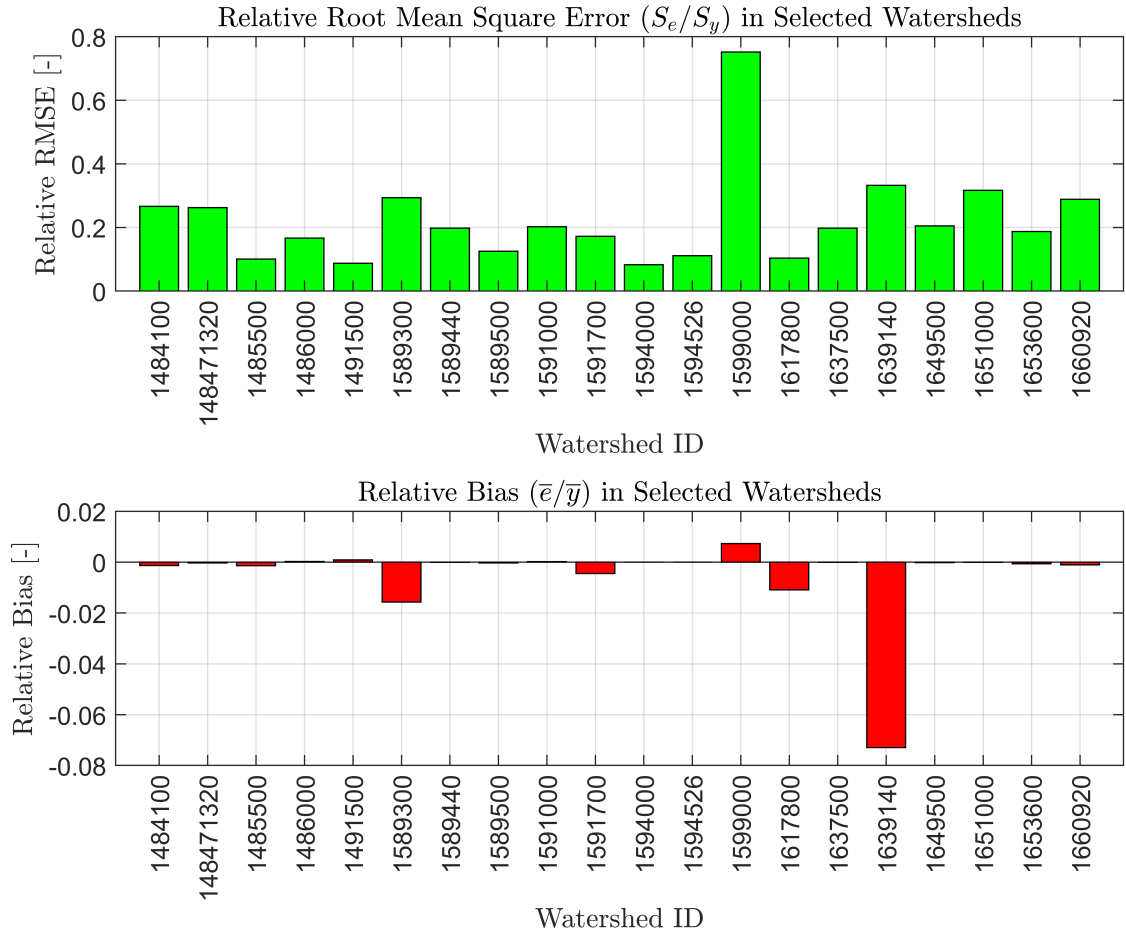


Figure 4.9: Relative RMSE and Bias for Hurricane Sandy Event in 20 Watersheds

The similar consistent trend can be observed in the relative bias bar plots. Most of the watersheds have very small relative errors ranging from -0.02 to 0.02. Errors up to $\pm 10\%$ are considered good [McCuen, 2018]. The error in watershed ID 1639140 is greater than errors in other watersheds reaching to around -7%. This watershed's event hydrograph has a double peak, and its RMSE is the second largest, amounting to almost 0.4 which can be explained by the unusual hydrograph shape and the resulting poor fit.

Georges Creek watershed (1599000, Index 1 in Figure 4.2) was discarded as an outlier.

Its PRF of 49 is lower than any value reported in the literature, and its Relative RMSE is 0.73. It is quite likely that the KLWX radar signal was interrupted by the Appalachian Mountain range, which lies between the radar location and the watershed.

4.6 Association of PRF with Watershed Properties

The remaining 19 watersheds were analyzed using linear regression to assess whether variation in PRF can be explained by selected predictor variables.

In addition to watershed area (Table 4.2), several variables were tested to assess if they could explain variation in the PRF. The predictor variables in this study include: bankfull Froude Number [-], bankfull celerity [ft/s], hydraulic length [mi], slope[ft/mi], land slope [ft/ft], and basin relief [ft]. The values of these predictor variables are summarized in Table 4.4.

Froude number is the nondimensional ratio of the inertial force to the force of gravity for a given fluid flow (AMS 2023) and computed by Equation 4.10.

$$Fr = \frac{v}{\sqrt{gD}} \quad (4.10)$$

where, v is bankfull velocity [ft/s], D is bankfull depth [ft], and g is gravitational acceleration [ft/s²].

Celerity represents the rate at which disturbances within the flow propagate downstream, and occasionally upstream [Beven, 2020]. For open channel flow in a wide rectangular channel, the celerity can be approximated as:

$$C = \sqrt{gD} \quad (4.11)$$

where g is the gravitational acceleration [ft/s²], D is the hydraulic depth (which equals the flow depth for a wide rectangular channel). Celerity represents the propagation speed of a shallow-water gravity wave; thus, the Froude number (Equation 4.10) is the ratio of flow velocity to the wave propagation speed. Hydraulic length is defined as the distance between the most remote point in the watershed and the watershed outlet. The channel slope signifies the inclination of the channel bed, extending from the channel's origin point to its outlet location. Land slope, sometimes referred to as watershed slope, is calculated as the average of all neighborhood slopes determined along the steepest direction of flow for all the pixels in the watershed [Maryland Hydrology Panel, 2020]. Basin relief characterizes the variation in altitude between the apex of the drainage divide and the point of discharge in the basin.

Slope data were gathered from [Maryland Hydrology Panel, 2020]; bankfull depth and velocity were estimated using the regression equations of [McCandless, 2002, 2003a,b]. For several watersheds, the predictor variables were missing; those datapoints are represented with “N/A” in Table 4.4.

The linear relationship between a selected predictor and the PRF was assessed using the coefficient of determination (R^2). The following formula was used to calculate the R^2 .

$$R^2 = \frac{SSR}{SST} \quad (4.12)$$

where SSR is sum of the squared regression (regression-predicted PRF minus mean observed

Table 4.4: Maryland Watersheds Predictor Variables

USGS ID	Bankfull Froude #	Bankfull Celerity [ft/s]	Hydraulic Length [mi]	Channel Slope [ft/mi]	Land Slope [ft/ft]	Basin Relief [ft]
01484100	0.222	6.806	NaN	5.2	0.0073	15.4
01485500	0.170	10.482	15.7	3.1	0.00841	28.54
01486000	0.212	7.329	7.1	6.4	0.00544	16.02
01491500	0.159	11.613	NaN	3.3	0.01189	45.72
01589300	0.483	11.140	15.9	21.2	0.056	198.4
01589440	0.501	10.668	10.6	32.3	0.078	237.5
01589500	0.431	7.370	4.7	31.3	0.036	75.52
01591000	0.479	11.270	13.2	29.8	0.092	259.7
01591700	0.496	10.794	11.2	26.5	0.056	172.3
01594000	0.414	13.448	25	14	0.059	266.5
01594526	0.296	11.709	20.3	7.7	0.055	125.25
01599000	NaN	NaN	19.6	58.5	0.17098	1181.4
01617800	0.487	8.722	10.1	25.5	0.035	149.3
01637500	0.437	12.594	25.3	45.6	0.124	665.5
01639140	0.486	11.069	NaN	17.6	0.042	NaN
01649500	0.304	11.324	17.8	27.5	0.055	201.41
01651000	0.456	11.961	20.5	21	0.065	298.8
01653600	0.330	10.269	15.9	15.8	0.057	184.24
01660920	0.301	11.494	19.3	9.2	0.044	129.92
0148471320	0.206	7.671	NaN	2.5	0.00619	29.61

PRF) and SST is total sum of squares which is the sum of the squares of the differences between the actual and the mean value of the dependent variable (observed PRF).

None of the identified predictors shows a strong association with the calculated PRF (Table 4.5). Overall, hydraulic length, bankfull celerity, and land slope have the highest R^2 (19.97%, 18.65%, 16.98%, respectively). Figure 4.10 illustrates the highest linear association identified (hydraulic length).

Table 4.5: Coefficient of Determination

Watershed Predictor Variables	Coefficient of Determination (R^2)
Drainage Area (mi^2)	0.0874
Froude Number	0.0632
Celerity (ft/s)	0.1865
Hydraulic Length (mi)	0.1997
Channel Slope (ft/mi)	0.0294
Land Slope (ft/ft)	0.1698
Basin Relief (ft)	0.157

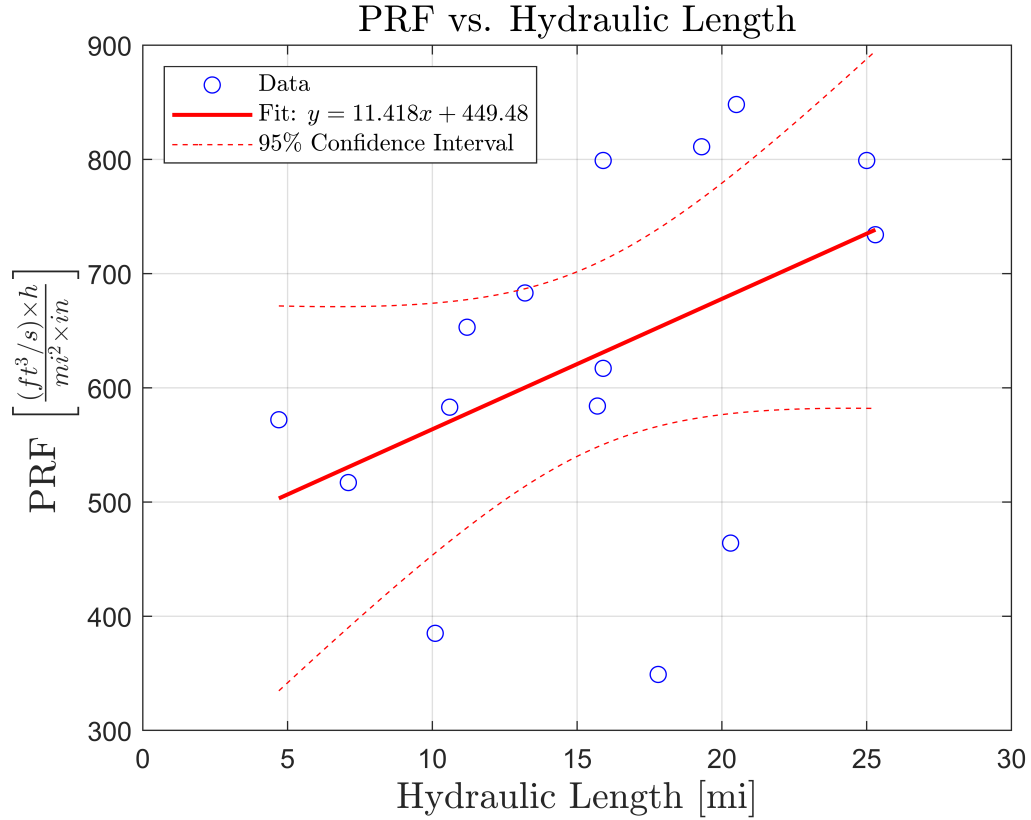


Figure 4.10: Regression Line of PRF vs Hydraulic Length

Table 4.6: Parameters of the Linear Regression of PRF on Hydraulic Length

	Estimate	SE	tStat	pValue
Intercept	449.48	105.14	4.2752	0.0009
x	11.418	6.34	1.80	0.0949

Based on the result provided in Table 4.6, for the intercept: The p-value is 0.0009, which is less than the typical threshold level of $\alpha=0.05$ (5%). Therefore, there is strong evidence to reject the null hypothesis that states the intercept is zero. This means, strong evidence exists to suggest that the intercept is not zero in the regression equation. For the predictor variable: The p-value is 0.0949, which is greater than the threshold level of

$\alpha=0.05$. Therefore, fail to reject the null hypothesis that the coefficient (slope) for the predictor variable (hydraulic length) is zero. This means there is not enough evidence to suggest that the predictor variable (hydraulic length) has an effect on the response variable (PRF).

4.7 Discussion

A major assumption of the UH approach is uniform precipitation over the watershed. To test whether this assumption is satisfied, the rainfall spatial coefficient of variation (COV) was analyzed. When the NEXRAD Level-III DPR data were analyzed, the time-varying spatial variance of precipitation was also calculated in the process of determining the time-varying watershed average. The spatial standard deviation is calculated as the square root of spatial variance. The coefficient of variation (COV) is calculated as:

$$COV = \frac{s_p}{\bar{P}} \quad (4.13)$$

where, s_p , is the spatial standard deviation of precipitation and $\bar{P}$ the spatial average at a given time. For uniform precipitation, both s_p and COV should be zero.

Time series of COV were calculated using Equation 4.13 in the example shown (Figure 4.11), the blue and red plots on the left are the spatial mean and standard deviation of the rainfall, and the blue circles on the right represent COV. In this illustration, COV less than 1 indicates a standard deviation less than the mean. The range of the COV for most of the rainfall analyzed is less than 1; the COV is generally small when the rainfall rate is highest, implying that the spatial uniformity assumption is appropriate during the

dominant flood-producing periods of the events.

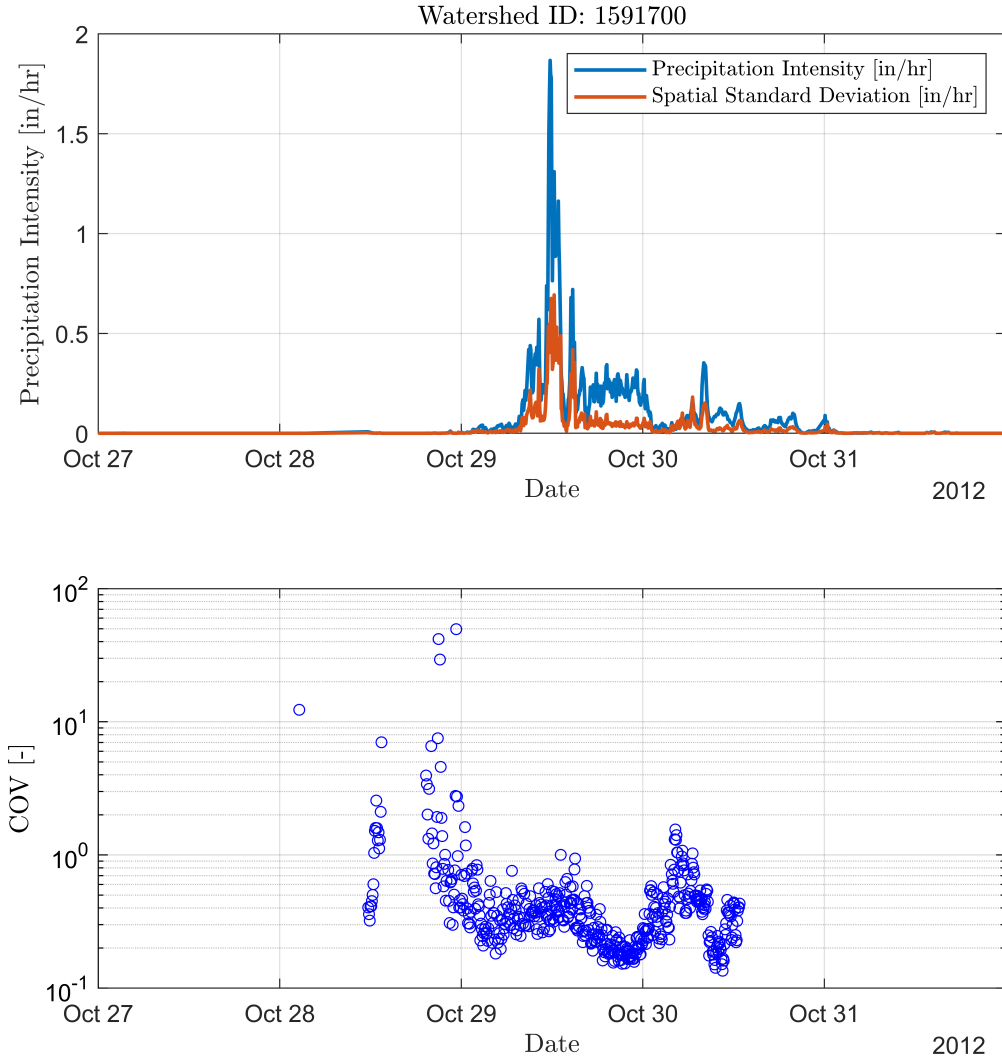


Figure 4.11: Areal mean and spatial standard deviation of watershed precipitation (upper plot) and Coefficient of Variation (lower plot)

The PRF is a parameter or property of a watershed's UH; its applicability is subject to the assumptions of the UH approach including the linear response assumed in the convolution operation, that is, that each increment of precipitation excess multiplies the same

UH response function to produce a contribution to total discharge at the outlet. The DUH identification and PRF estimation in this study make this assumption.

A further assumption in application of the UH approach is that the UH is a property of the watershed and can be applied to any precipitation event. The current study does not address this assumption, because only a single event was examined for each watershed. It is possible that the PRF varies with precipitation magnitude; this might explain the result that the estimated PRFs are greater than the standard NRCS values. This is a matter for further investigation when and where the NRCS curvilinear UH is applied in practice; underestimating the peak discharge of the UH would lead to underestimating event peak discharge, with implications for infrastructure design.

The UH is defined as the direct runoff response to a unit of precipitation excess. Identifying the UH from observed data required separating the hydrograph into direct runoff and baseflow, and the hyetograph into precipitation excess and losses. Despite efforts to make these calculations objective and consistent, both of these analysis processes are subjective and uncertain.

4.8 Conclusions and Recommendations

In this research, the NRCS-recommended gamma DUH, with adjustable shape as reflected by the PRF, was applied. The NRCS unit hydrograph method was used to calculate the PRFs of watersheds using NEXRAD Level-III rainfall data and associated streamflow data. This method was applied to 20 Maryland watersheds (11 Coastal Plain and 9 in Piedmont, Blue Ridge Mountains and Valleys, and Appalachian Plateau) for the

Hurricane Sandy event in October 2012. The average PRF for the 20 study watersheds was 570; excluding one outlier (George's Creek), the average was 597. This result exceeds the standard NRCS PRF of 484. The average estimated PRF for 5 watersheds on the Delmarva peninsula (including Maryland's Eastern Shore or Eastern Coastal Plain province) was 580, which exceeds the PRF of 284 corresponding to the NRCS Delmarva dimensionless unit hydrograph.

A large variation exists in the PRF values, which range from 49 to 848. No clear regional trend in the PRF values was found, although some similarities were observed in adjacent watersheds. Seven watershed characteristics (predictor variables) were analyzed to assess if the variation in PRFs can be explained (using R^2) but no clear association was found between these event PRF values and watershed physical properties. The highest R^2 was found for hydraulic length.

To obtain the direct runoff hydrograph and precipitation excess hyetograph required for UH identification, this study applied, respectively, a modified concave hydrograph separation, and a phi-index hyetograph separation. Further research should examine sensitivity to the selection of separation methods.

The flow events analyzed in this study were 2% to 50% annual exceedance probability peak discharges for their respective gauges. Based on a single event, the conclusion that larger events have larger PRF cannot be demonstrated. Future research should estimate DUH and PRF for a range of events to explore whether the DUH and PRF are functions of event magnitude. Non-dependence on event magnitude is a major assumption in application of the UH approach. Relaxing this assumption could contribute to improved safety and resilience, but proper guidance is required.

Despite conceptual issues and major assumptions, UH methods remain important expedient methods for estimation in ungauged watersheds. Adjusting the UH approach with improved rainfall, streamflow, and watershed data continues to be an important topic for research in applied hydrology.

Future work will continue to investigate the applicability of the UH approach with spatially-varying precipitation, refinement of the PRF estimation procedure, and further efforts to associate PRF with watershed and event characteristics.

Chapter 5: Distributed Hydrologic Model of Jones Falls Watershed

5.1 Introduction

Hydrologic modeling seeks to predict the behavior of watersheds under varying weather conditions. The unit hydrograph (UH) approach is widely used in such assessments. The UH is defined as the graph of watershed direct runoff response to a unit depth of precipitation excess occurring steadily over a defined time increment and uniformly over the watershed. These assumptions, while practical for simplifying complex hydrological processes, may not always align with the inherent variability of real-world precipitation patterns. Such a precipitation event is unlikely to be observed in nature, which makes identification of the watershed's UH a complex process. Numerical simulation provides the opportunity to generate such events and examine their outcomes. According to [Szilagyi \[2007\]](#), physically based hydrologic modeling has the potential to evaluate the non-linear response of a watershed.

The primary motivation behind this chapter was to use a numerical model to create unit precipitation events, simulate the watershed response, and compare it to the NRCS UH. It was hoped that a fully-distributed, physically-based model would provide a digital laboratory to investigate how actual watershed properties affect runoff response to a unit of precipitation excess, and compare the physically generated hydrograph(s) to those

identified in Chapter 3. To test the assumptions of the UH approach, it was necessary to apply a modeling environment that does not use those assumptions. The Gridded Surface Subsurface Hydrologic Analysis (GSSHA), a comprehensive distributed hydrology model, was selected. Ultimately, the effort required was prohibitive. Time and financial resources did not allow the project to attain this goal. However, the extensive effort invested in the horizontally- and vertically-distributed soil model representation provided an opportunity to compare the detailed soil representation to a simpler treatment. This was ultimately the experiment that was performed with the GSSHA model in this chapter.

The UH approach assumes a time-invariant watershed response to a pulse of precipitation excess, assumed to apply uniformly over the watershed. These assumptions are likely violated by time- and space-varying land surface and soil conditions in the watershed. With modern modeling tools and datasets, it is possible to create a watershed model with explicit treatment of unsaturated zone processes and spatially varying soil properties at a fine scale, allowing examination and assessment of the UH assumptions. This research used the GSSHA modeling environment to create a 10 meter resolution model of the Jones Falls watershed (65.3 km²) in the Maryland Piedmont. The model applies the Richards Equation option to simulate the unsaturated zone, using soil data obtained from the SSURGO database [[U.S. Department of Agriculture, Natural Resources Conservation Service \(USDA NRCS\), n.d.](#)]. The watershed contains 71 different soil units, with depths ranging from 100 to 292 cm, and with up to 10 identified horizons. A soil index map was created, assigning an index value to each 10 m $\times$ 10 m pixel soil component. Horizon data were organized into a 3 horizon soil profile for each soil index. Two versions of the model were tested: one run with fully distributed spatially varying soil properties, and another with a single soil

profile applied to the entire watershed. Differences between the output of the two versions were examined, including partitioning of arriving rainfall into direct runoff and infiltration, peak discharge hydrograph shape and components (i.e., surface flow vs baseflow).

5.2 Background and Literature Review

This section provides a brief description of the GSSHA modeling environment, a summary of published research using GSSHA to model watershed runoff, and a review of recent work evaluating the pros and cons of detailed soil process treatment in watershed modeling.

5.2.1 The GSSHA Modeling Environment

GSSHA model is a distributed, two-dimensional, numerically-based model that uses a physical approach. It has been utilized by the US Army Corps of Engineers (USACE) for various applications such as engineering hydrology [Downer et al., 2016], erosion [Downer et al., 2015], contaminant and nutrient fate and transport [Pradhan et al., 2014], and storm surge research [Massey et al., 2013]. According to Szilagyi [2007], using a physically based hydrologic model can capture the non-linear hydrological features of a watershed, while also aiding in the development of a procedure for testing and enhancing the estimation of Unit Hydrograph Peaking Factor (UHPF) [U.S. Army Corps of Engineers (USACE), 1991] and precipitation excess relationships across varying climatic areas within the continental United States [Wolock et al., 2004].

The GSSHA model encompasses several crucial components: 1- Overland Flow: Uti-

lizing a finite volume method, the model represents the watershed's topography through a grid of cells and simulates overland flow by considering precipitation, infiltration, and evaporation. The water flow between cells is determined by mass and momentum conservation equations. 2- Channel Flow: The model employs one-dimensional hydraulic equations to account for the channel's geometry, roughness, and other factors impacting water flow. 3- Infiltration and Soil Moisture: GSSHA estimates soil moisture content and its distribution in response to precipitation, evapotranspiration, and infiltration using either the Green-Ampt or the Richards Equation. 4- Groundwater Flow: The model is capable of simulating confined and unconfined aquifer systems through a two-dimensional finite difference approximation of the groundwater flow equation. 5- Evapotranspiration: GSSHA calculates evapotranspiration according to the Penman-Monteith equation, taking into account variables such as solar radiation, temperature, humidity, and wind speed. 6- Vegetation and Land Use: GSSHA considers the impacts of vegetation and land use on hydrological processes by accounting for factors such as canopy interception, root zone storage, and surface roughness.

The GSSHA model is extensively utilized for a range of applications, including flood prediction and management, water resources planning, erosion control, habitat restoration, and water quality management. Ongoing updates and refinements are made to the model to incorporate novel research findings and enhance its accuracy and functionality.

[Downer et al. \[2002a\]](#) stated that the hydrologic elements are interconnected to maintain a mass balance. In the context of GSSHA, explicit finite volume schemes are utilized to route the flow of water for both 1-D channels and 2-D overland flow.

5.2.1.1 The Richards Equation

In-depth modeling of the soil water profile within the vadose or unsaturated zone is a crucial aspect of the GSSHA model. This dynamic region governs the water flux between the surface and groundwater and is responsible for partitioning rainfall into infiltration, runoff, groundwater recharge, and evapotranspiration (ET). The most comprehensive method for modeling these complex and interconnected processes is by employing the Richards Equation [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki, n.d.](#)]. Although some consider the Richards Equation impractical for hydrologic predictions, it has been effectively utilized in field-scale and watershed-scale models for soil moisture and runoff [[Lappala et al., 1987](#); [Refsgaard and Storm, 1995](#); [Wagenet and Hutson, 1989](#)]. Simpler approaches, such as Green and Ampt (GA) and Green and Ampt with Redistribution (GAR), which are approximations of the Richards Equation, do not offer detailed soil moisture profiles or simulate water movement from the groundwater to the unsaturated zone. Also, appropriate representation of layered soils or soils with a water table is challenging with these estimates [[Short et al., 1995](#)]. To simulate infiltration using the Richards Equation, the INF_RICHARDS option in the GSSHA project file is applied [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki, n.d.](#)].

Unsaturated zone processes are crucial to surface water hydrology. Infiltration, ET, and groundwater recharge are primarily vertically oriented. GSSHA addresses the one-dimensional (vertical direction) head-based formulation of the Richards Equation as follows:

$$C(\psi)\frac{\partial\psi}{\partial t} - \frac{\partial}{\partial z} \left[K(\psi) \left(\frac{\partial\psi}{\partial z} - 1 \right) \right] - W = 0 \quad (5.1)$$

where, C is the specific moisture capacity, ψ is the soil capillary head [cm], z is the vertical coordinate (downward positive) [cm], t is time [hr], $K(\psi)$ is the effective hydraulic conductivity [cm], and W is a flux term added for sources and sinks (cm/hr).

According to [Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki](#) [n.d.] the head-based formulation enables solving the Richards Equation for both saturated and unsaturated conditions [[Haverkamp et al., 1977](#)]. This approach has been effectively applied in field-scale unsaturated flow models such as VS2D [[Lappala et al., 1987](#)], LEACHM [[Wagenet and Hutson, 1989](#)], and SWAP [[Kroes and Van Dam, 2003](#)], as well as MIKE SHE [[Refsgaard and Storm, 1995](#)]. Utilizing a head-based formulation, processes like rainfall, evaporation, and groundwater recharge can be modeled without altering variables. Mass balance issues linked to the head-based formula solution [[Celia et al., 1987, 1990](#); [Pan and Wierenga, 1995](#); [Ross, 1990](#)] have mostly been resolved by the introduction of novel solution techniques [[Kirkland et al., 1992](#); [Pan and Wierenga, 1995](#); [Rathfelder and Abriola, 1994](#)]. Within GSSHA, the Richards Equation is solved using an implicit finite difference method, which directly corresponds to the overland flow finite difference grid. The soil column beneath each overland flow cell is divided into several unsaturated zone cells, as shown in [Figure 5.1](#).

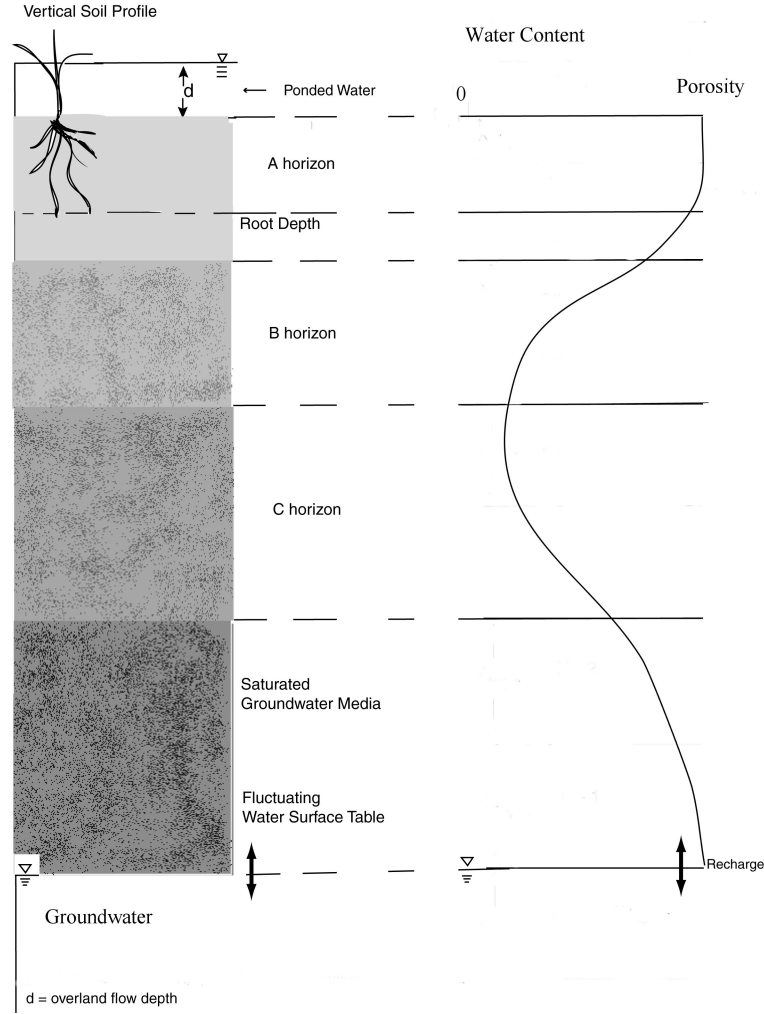


Figure 5.1: GSSHA representation of the unsaturated zone [Gridded Surface Subsurface Hydrologic Analysis (GSSHA) Wiki, n.d.]

5.2.1.2 Surface Water Routing: 1-D Channel Routing

The GSSHA channel routing is illustrated in Figure 5.2. Inter-cell flows, $Q_{i-1/2}$ and $Q_{i+1/2}$ [m³/s] in the longitudinal, x , directions are computed from depths, d , at the n time interval utilizing the Manning formula for the head discharge connection as follow [Downer et al., 2002a]:

$$Q_{i+1/2} = \frac{1}{n} A_i^n (R_i^n)^{2/3} (S_{f_{i+1/2}})^{1/2} \quad (5.2)$$

where n is roughness coefficient, A is the cross-sectional area of the flow (in the channel) [m²], R is the hydraulic radius, and S_f is the friction slope, computed in the x direction as [Downer et al., 2002a]:

$$S_{f_{i+1/2}}^n = S_{0_{i+1/2}} - \frac{d_{i+1}^n - d_i^n}{\Delta x} \quad (5.3)$$

where S_{0x} is the land slope in the direction of x . In the case of negative flow (e.g., flow in the upstream direction), the head in the downstream cell is applied to compute the flow as [Downer et al., 2002a]:

$$Q_{i+1/2} = \frac{1}{n} A_{i+1}^n (R_{i+1}^n)^{2/3} (S_{f_{i+1/2}})^{1/2} \quad (5.4)$$

because the direction might change in the stream at any point, particularly in the ephemeral streams close to the onset of rainfall events, the direction of flow is identified around each node and the locally upstream cell characteristics are utilized to compute the flow.

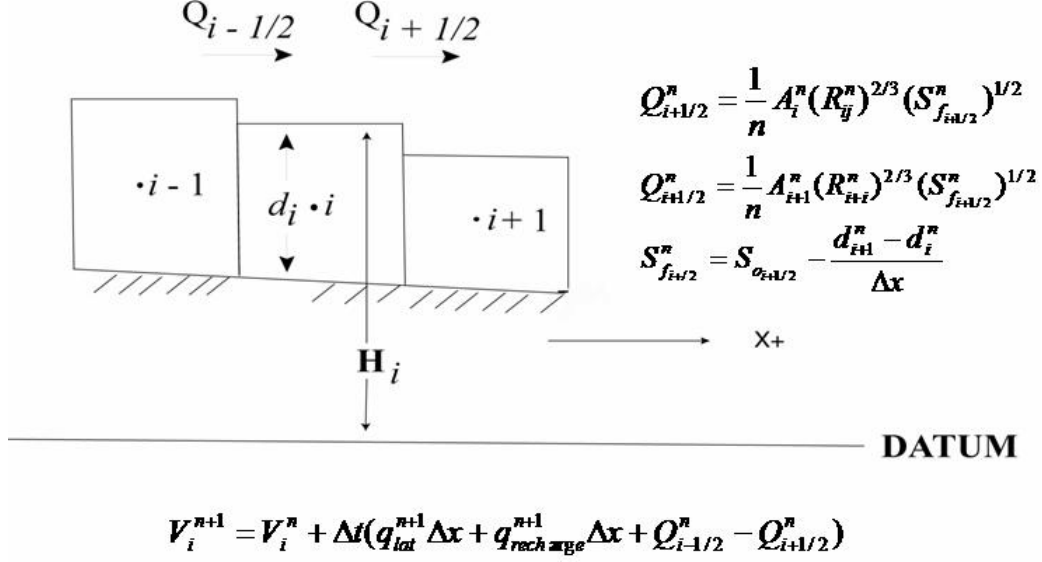


Figure 5.2: GSSHA Explicit Channel Routing Scheme [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki](#), n.d.]

5.2.1.3 Surface Water Routing: 2-D Overland Flow Routing

The 2D overland flow component in GSSHA uses a finite volume method to solve the full 2D shallow water equations, which include the conservation of mass and momentum equations. This approach considers factors like topography, land cover, and soil properties to model the flow pathways and velocities of water over the land surface. Overland flow in 2-D GSHHA model is similar to 1-D channel routing but for each grid cell, calculation is performed on two orthogonal directions at every time step. Further, the watershed boundary is a no-flow boundary for 2-D overland flow routing. In GSSHA, $\Delta x = \Delta y$. Inter-cell fluxes in x and y directions are p and q respectively, and calculated in cell ij from the depth, d_{ij} , at the n_{th} time interval utilizing the Manning formula for the head discharge

connection in the x and y directions, respectively, as [Downer et al., 2002a]:

$$p_{ij}^n = \frac{1}{n} (d_{ij}^n)^{5/3} (S_{f_x})^{1/2} \quad (5.5)$$

$$q_{ij}^n = \frac{1}{n} (d_{ij}^n)^{5/3} (S_{f_y})^{1/2} \quad (5.6)$$

where p_{ij} and q_{ij} are flows in x and y directions.

To calculate the depth in each cell for the $(n + 1)_{th}$ time interval and based on the discharges for each cell [Julien and Saghaffian, 1991]:

$$d_{ij}^{n+1} = d_{ij}^n + \frac{\Delta t}{\Delta x} (p_{ij-1}^n + q_{ij-1}^n - p_{ij}^n - q_{ij}^n) \quad (5.7)$$

Two solution methods were proposed by [Downer et al., 2000] consisting of an Alternating Direction Explicit Scheme (ADE) and an ADE scheme with an extra Predictor-Corrector step (ADEPC).

If the ADE scheme is applied, inter-cell discharges are computed for the x-direction based on Equation 5.5. Updating of the depths in every row takes place in accordance with the discharges for x-direction [Downer et al., 2002b]:

$$d_{ij}^{n+1/2} = d_{ij}^n + \frac{\Delta t}{\Delta x} (p_{ij-1}^n - p_{ij}^n) \quad (5.8)$$

inter-cell discharges in the y-direction are then calculated applying the updated depth [Downer et al., 2002a,b]:

$$q_{ij}^{n+1/2} = \frac{1}{n} \left(d_{ij}^{n+1/2} \right)^{5/3} \left(S_{fy}^{n+1/2} \right)^{1/2} \quad (5.9)$$

Updating of the depths in every column occurs in accordance with the discharges in the y-direction [Downer et al., 2002b]:

$$d_{ij}^{n+1} = d_{ij}^{n+1/2} + \frac{\Delta t}{\Delta x} \left(q_{ij-1}^{n+1/2} - q_{ij}^{n+1/2} \right) \quad (5.10)$$

The 2D overland flow component in GSSHA provides a robust and detailed representation of surface water movement, accounting for complex spatial patterns and hydrologic processes in a watershed.

5.2.1.4 Interception

Based on [Gridded Surface Subsurface Hydrologic Analysis (GSSHA) Wiki, n.d.], in the GSSHA model, the interception of rainfall by vegetation is calculated using the dual parameter method introduced by [Gray, 1970]. An initial amount of rainfall (mm), completely intercepted by the foliage, is denoted by the STORAGE_CAPACITY project card. The proportion of rainfall that remains as intercepted water after fulfilling the storage capacity is denoted by the INTERCEPTION_COEFF card. These two parameter may be specified with ASCII raster maps. Alternatively, both the storage capacity and the interception coefficient can be assigned using a mapping table and an index map based on vegetation.

The interception rate (i) in GSSHA is given by:

$$i(t) = r(t) \text{ if } I < a \quad (5.11)$$

$$i(t) = b \cdot r(t) \text{ if } I > a \quad (5.12)$$

where $r(t)$ represents the rainfall intensity at time t , a is the storage capacity [mm], b is the interception coefficient (dimensionless, ranging from 0.0 to 1.0), I is the cumulative interception depth. Values for storage capacity and interception are typically derived from vegetation or land cover types. Values can be found for storage capacity and interception coefficient in [Gray, 1970] (section 4.6) or [Bras, 1990], page 233. To account for the seasonal variability of interception in temperate zones, the storage capacity parameter can be seasonally adjusted.

5.2.2 Application of GSSHA in Watershed Modeling

Sharif et al. [2010] discussed application of a distributed hydrologic model for a particular flood event in Texas. They argued that findings suggested that choosing a grid cell size in GSSHA larger than 30 meters would be inappropriate, as it resulted in a simulated hydrograph that remarkably underestimated the runoff. That indicates finer resolution might be crucial for model accuracy and accurate runoff estimation, particularly in larger basins. In another paper, Downer and Ogden [2004] illustrated that the model solutions for partial differential equations conserve mass and closely connect the hydrological elements to ensure a full mass balance.

5.2.3 Comparison of GSSHA and Other Watershed Models

Evaluating the model on a catchment area featuring a variety of runoff methods demonstrates its effectiveness in examining the critical processes that generate stream-flow on a watershed level and ascertaining the contributions from surface water, saturated groundwater, and unsaturated groundwater to the complete water balance. According to [Texas A&M University \[n.d.\]](#),

The Soil & Water Assessment Tool (SWAT) is a small catchment to river basin-scale tool used to model the quantity and quality of surface and ground water and estimate the environmental impact of land use, land management practices, and climate change. SWAT is broadly used in evaluating soil erosion prevention and control, non-point source pollution control and area-specific management in watersheds.

SWAT simulates the hydrological cycle on a daily basis. This includes processes like precipitation, surface runoff, evapotranspiration, percolation, lateral flow, and return flow.

Another model that is widely used in hydrologic modeling is the Storm Water Management Model (SWMM). According to [Environmental Protection Agency \(EPA\) \[n.d.\]](#),

EPA's Storm Water Management Model (SWMM) is applied throughout the globe for planning, analysis, and design pertaining to stormwater runoff, combined and sanitary sewers, and other drainage systems. It is useful to evaluate gray infrastructure stormwater control procedures, such as pipes and storm drains, and it can be used for creating cost-effective green/gray hybrid stormwa-

ter control solutions.

[James \[2009\]](#) discussed that rainfall input can be provided at various time resolutions, often from 1 minute up to 1 day for SWMM. The time steps for calculating runoff in SWMM are set based on the wet weather (with rainfall) or dry weather (no rainfall) condition but should generally be equal to or less than the rainfall time interval.

[Sith and Nadaoka \[2017\]](#) suggested that for high-resolution temporal prediction of streamflow and sediment concentration in a small watershed, the GSSHA model demonstrated marginally superior performance compared to the SWAT model in estimating hourly streamflow. Furthermore, GSSHA delivered more precise results for hourly sediment concentration, especially during intense flooding occurrences. Nonetheless, the SWAT model's effectiveness in predicting sediment concentration using sub-daily erosion remained adequate. Additionally, this investigation unveiled that GSSHA, as a fully distributed model employing structured grid cells, yielded improved predictions of sediment concentration compared to SWAT, a semi-distributed model.

[Brendel et al. \[2021\]](#) compared SWMM, a semi-distributed model and GSSHA models developed for the city of Roanoke, Virginia to evaluate the magnitude, duration, and spatial extent of urban flooding. The result of their research highlights that the both models have distinct strengths. The utility of GSSHA to the City of Roanoke lies in its capacity for forecasting the temporal length and geographical distribution of flood events. This is facilitated by its comprehensive distributed structure, which enables two-dimensional calculation and graphical representation of model results. [Moore et al. \[2017\]](#) demonstrated that the GSSHA model represents the groundwater component with higher precision than

the SWMM model.

[Furl et al. \[2018\]](#) evaluated the performance of multiple satellite rainfall products with respect to gauge corrected ground-based radar estimations for few moderate to high magnitude events across the Guadalupe River system in South Texas. They conducted the experiment across three nested catchments (ranging from 200 to 10,000 km² in area) to capture and measure the effect of the scale on the propagation of the error. Satellite-based rainfall data sets were used to force a fully distributed physics-based model (GSSHA) to examine error propagation through the hydrologic model. In their research, both gauge-corrected and uncorrected satellite products were used, encompassing a variety of latency times, spatial resolutions, and temporal resolutions. They concluded that the location and movement of a storm play a significant role in the maximum flow rate. Specifically, if the intense cells of the storm are concentrated in the middle section of the basin, it can significantly reduce the peak flows while keeping the overall volume of runoff constant. The errors of the precipitation estimates were amplified in the simulated hydrographs. Even though the satellite products provided evenly distributed near-global spatiotemporal estimates, they significantly underestimated strong storm events in spatial scales.

[Jaiswal et al. \[2020\]](#) conducted a study comparing the TANK and Australian Water Balance Model (AWBM) conceptual models to the Soil and Water Assessment Tool (SWAT) model, which was physically distributed but lumped on HRUs scale, for the Tandula basin in Chhattisgarh, India. The TANK model is a conceptual rainfall-runoff model that simulates the hydrological processes using a series of vertical tanks. The AWBM is another conceptual model designed to simulate runoff based on the water balance in different storages within a catchment. The TANK model and AWBM were set up using the

Rainfall Runoff Library (RRL) tool. In the research, three models were used in a basin: two conceptual models (AWBM and TANK), and a physically semi-distributed model (SWAT). Each of these models has its own unique set of parameters and optimization methods. The effectiveness of these models was assessed using several standard fit-quality metrics, such as root-mean-square error, correlation coefficient, Nash-Sutcliffe efficiency, and coefficient of determination, among others. The evaluation of the models indicated that the TANK model outperformed both the AWBM and SWAT models included in the study. The SWAT model, as a semi-distributed model, demonstrated the least effective performance among all the models utilized. However, its ability to use spatially distributed data on landforms, soil, topography, and climate still makes it a valuable tool for evaluating the effects of land use, climate change, and management strategies implemented in the basin. The research further concluded that the adoption of distributed methods could result in more desirable outcomes.

5.3 Study Area and Dataset

The study area for this research is Jones Falls at Sorrento, Baltimore County, Maryland. In previous chapters, the Jones Falls watershed had largest number of events for the Natural Resources Conservation Service (NRCS) unit hydrograph analysis and it was hoped to be able to generate the watershed's unit response and compare to the UHs determined in that study. This served as the foundational rationale for its selection in the analysis. The drainage area of the watershed is 65.4 km^2 (25.25 mi^2). The watershed gauge is located at Latitude: $39^\circ 23' 44.7''$, Longitude: $76^\circ 39' 46.4''$ NAD83. The Jones Falls Watershed is

a significant drainage area in Maryland, United States, and is part of the larger Chesapeake Bay Watershed. Downstream of the gauge, the main watercourse, Jones Falls, flows through Baltimore City before emptying into the Inner Harbor. The watershed is critical for providing drinking water, supporting wildlife, and offering recreational opportunities to local residents. The data for this study obtained from United States Geological Survey (USGS) National Water Information System (NWIS) [U.S. Geological Survey (USGS), n.d.] and Soil Survey Geographic Dataset (SSURGO) [U.S. Department of Agriculture, Natural Resources Conservation Service (USDA NRCS), n.d.]. Figure 5.3 provides the elevation map of Jones Falls Watershed. Additionally, the stream network, defined by links and nodes, is depicted inside the watershed map. The procedure to create links and nodes is discussed in more detail later in this chapter. The input data necessary to run the GSSHA model are summarized in Table 5.1.

Table 5.1: GSSHA Input Variables and Units

Input variables	Unit	Input variables	Unit	Input variables	Unit
Elevation	m	Interception Coeff.	[-]	Links & Nodes	[-]
Roughness	[-]	Water Table	m	Stream Cells	[-]
Precipitation	mm.hr ⁻¹	Aquifer Bottom	m	Mapping Table	[-]
Storage Capacity	mm	Groundwater Boundary	[-]	Soil Index	[-]

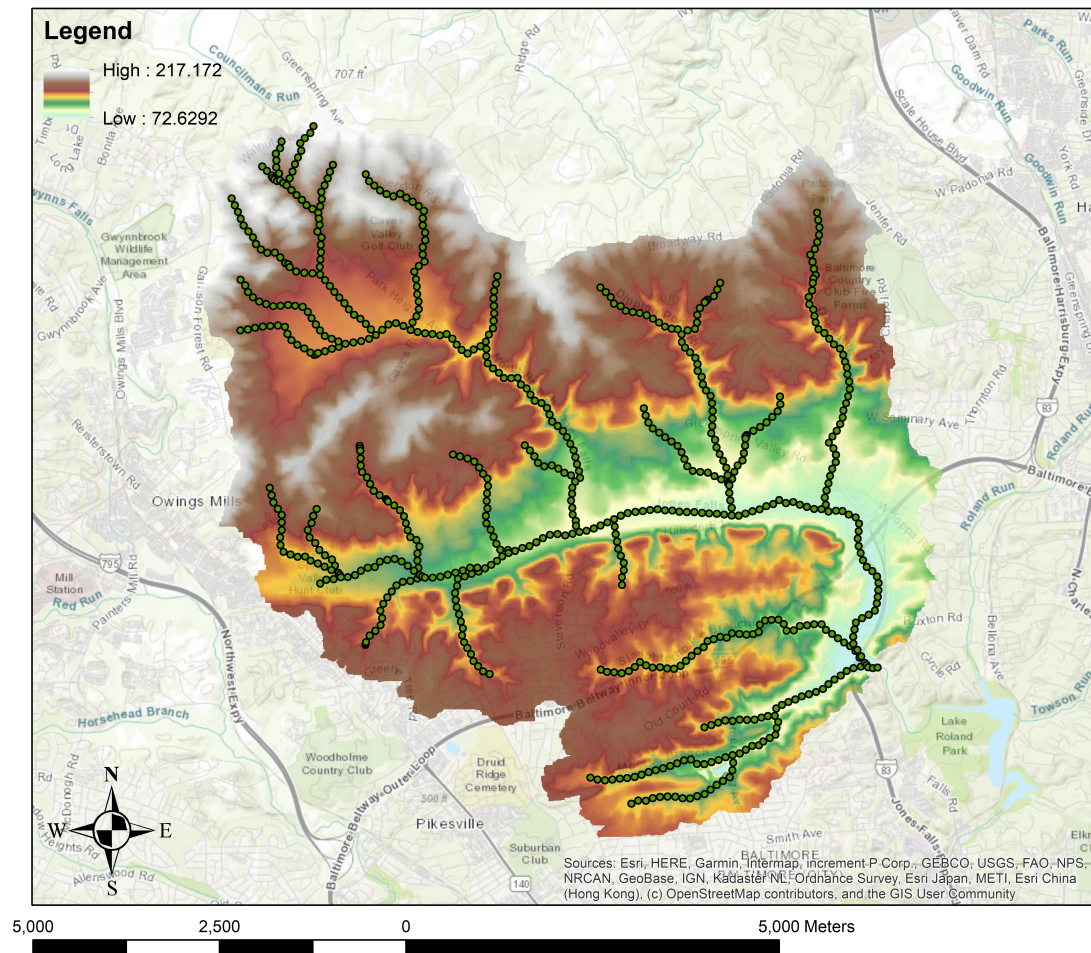


Figure 5.3: Elevation map of Jones Falls Watershed in Maryland

5.3.1 Distributed Soil Parameters

The allocation of distributed soil parameters for the Richards Equation is conducted using an index map and a mapping table. The distinction between parameter assignment in the Richards Equation and other processes lies in the requirement to define three soil layers for each unique soil type present in the index map. Furthermore, parameters and vertical discretization must be designated for all three layers.

The necessary parameters are contingent upon the chosen method for defining the CPS (Capillary Pressure-Saturation) curves (a.k.a Water Retention Curve or Soil Water Characteristics Curve), either Brooks or Haverkamp method. The CPS relationship describes how capillary pressure varies with the degree of saturation in the soil. It represents the relationship between the volumetric water content and the pressure head. Water retention curves by different methods including [Brooks and Corey, 1964] are illustrated in Figure 5.4. The Haverkamp formulation is deemed optimal when laboratory data are accessible for fitting the required parameters. Downer [2002] demonstrated that Haverkamp equations were most effective in defining soil behavior through testing. In the absence of detailed data, which is a common occurrence, the [Brooks and Corey, 1964] approach can be employed. The parameters required for the Brooks and Corey analysis can either be quantified or approximated using soil textures and literature, such as [Rawls et al., 1982].

The assignment of parameters for the Richards Equation is conducted using a raster soil-index map, with a set of parameters assigned to each index value. When employing the Brooks and Corey approach in GSSHA, the following parameters are designated for each of the three layers [Gridded Surface Subsurface Hydrologic Analysis (GSSHA) Wiki, n.d.]:

Hydraulic Conductivity – saturated hydraulic conductivity describes the rate at which water will move through the soil under unit head gradient and saturated conditions (cm/hr).

Porosity – volume of voids/total volume of soil, fraction between 0.0 and 1.0 (m^3/m^3).

Residual saturation – water content of air-dry soil, fraction between 0.0 and 1.0 (m^3/m^3).

Wilting point – water content below which plants cannot take up water from the soil (m^3/m^3).

Depth – thickness of the soil layer (cm). Should be rounded up or down to nearest cen-

timeter.

Lambda (λ) – pore distribution index (cm/cm). Describes the straight line length to the soil water path length.

Bubbling pressure (ψ_b) – pressure at which air enters the soil column (cm). Must be negative.

Delta Z – vertical cell size of the layer (cm). Should be evenly divisible into the depth of the layer.

5.3.2 Soil Data Processing

The soil data for the GSSHA model of Jones Falls were obtained from the [\[U.S. Department of Agriculture, Natural Resources Conservation Service \(USDA NRCS\), n.d.\]](#) Web Soil Survey. A total of 71 different soil groups were identified in the watershed. These were labeled and mapped by index (ID), and all required parameters were estimated for each soil index. As GSSHA necessitates precisely three horizons (H1, H2, and H3) for each soil profile, multi-step processing was performed on the soil data to ensure its compatibility with the required format. This section explains the steps involved in processing the soil data.

The SSURGO database identifies soil horizons (natural layers) as O, A, B, C, and sometimes E or R. When the SSURGO database designated soil horizons as A, B, and C, these labels were retained. In cases where multiple A, B, or C horizons were provided by SSURGO, a weighted average parameter value was calculated over the similar adjacent layers. For instance, if horizons B1, B2, and B3 were given, a weighted average was performed

on all B horizons, and the result was reported as the B horizon.

For the vertical soil column, Saturated Hydraulic Conductivity (K) was determined using the formula:

$$K = \frac{d_1 + d_2 + \dots + d_n}{\frac{d_1}{K_1} + \frac{d_2}{K_2} + \dots + \frac{d_n}{K_n}} \quad (5.13)$$

where d represents the layer thickness.

Thickness-weighted averages were calculated for Lambda (λ), Bubbling Pressure (ψ_b), Residual Saturation (θ_r), and Porosity. Field Capacity was determined as $\theta(\psi_{fc}=337 \text{ cm})$ and Wilting Point as $\theta(\psi_{wp}=15306 \text{ cm})$. These values were estimated using Brooks-Corey equation [Brooks and Corey, 1964] as follows:

$$\theta_{wp} = \theta_r + \left(\frac{15306}{\psi_b} \right)^{-\lambda} (\theta_s - \theta_r) \quad (5.14)$$

$$\theta_{fc} = \theta_r + \left(\frac{337}{\psi_b} \right)^{-\lambda} (\theta_s - \theta_r) \quad (5.15)$$

where θ_r is residual saturation, ψ_b is bubbling pressure and should be negative, λ is pore size distribution, and θ_s is porosity. The same soil parameters were applied across three horizons for Udorthents, urban lands, and water. The depths of these layers were 10, 20, and 70, respectively, with dz values of 1 for the first horizon, 2 for the second, and 5 for the third.

Water Retention Curves

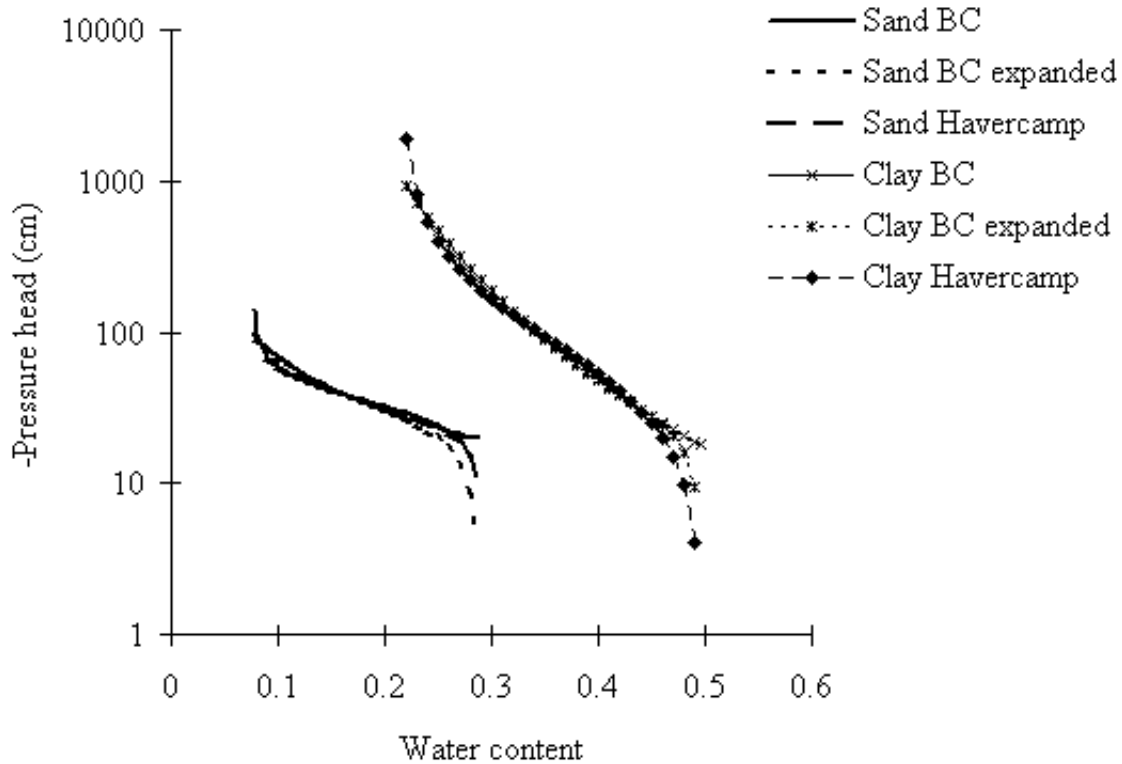


Figure 5.4: Water Retention Curves [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki](#), n.d.]

For H1 and H2 horizons, H2 was divided into two horizons with layer thicknesses of 30 and 135. Dz values were set to 1 for H1 and 2 and 5 for H2 and H3, respectively. All depths were adjusted to represent the actual thickness of the layers. After splitting the horizon, the thicknesses were set to 1/3 and 2/3 of the actual thickness. All depths were corrected, converting the depth column values to thickness. Also, further adjustments made to the layers and horizons for various IDs as follow:

ID 29-31, 35: For cases where only two horizons were present, the second horizon was split in half, with the total depth divided by 2 (e.g., depth $158/2 = 79$ for ID 35), and the dz was adjusted accordingly (e.g., 7.9 for ID 35).

ID 32, 33, 34: The same layer was added for horizons 2 and 3, with depths set to 10, 20, and 70, and dz values set to 1, 2, and 5.

ID 36, 37, 45, 50: The first two horizons were combined.

ID 52: The second horizon was repeated, and the total depth of the horizon was split in half. The dz was adjusted accordingly.

ID 54: Udorthents were repeated three times, with depths set to 10, 20, and 70, and dz values set to 1, 2, and 5.

ID 55, 56: Horizon H3 was added, with parameters identical to H2. The depth of H2 (160) was split into 30 for H2 and 130 for H3, with dz values set to 1, 2, and 5.

These layer and horizon adjustments for different IDs ensured that the soil data complied with the GSSHA model's requirements. By combining, splitting, and calculating weighted averages for the various horizons, the soil data was effectively processed to conform to the necessary format.

Table 5.2: Characteristics of Selected Dominant Soil Profiles

Soil Index	Soil Horizon	Conductivity [cm/hr]	Field Capacity [cm ³ /cm ³]	Wilting Point [cm ³ /cm ³]	Thickness [cm]
24	A	9.17	0.264	0.117	25
24	B	8.51	0.279	0.132	112
24	C	9.17	0.145	0.061	56
28	A	9.00	0.290	0.143	33
28	B	7.70	0.241	0.159	135
28	C	21.20	0.255	0.123	17
30	A	9.00	0.254	0.105	16
30	B	9.00	0.175	0.083	40
30	C	28.00	0.182	0.084	127
31	A	9.00	0.255	0.101	23
31	B	9.00	0.291	0.148	38
31	C	9.00	0.282	0.145	91

5.3.3 Soil Depth and Discretization

Soils within the unsaturated zone must be characterized down to a depth encompassing the soil's active region, where significant fluctuations in water content occur, including the root zone. Typically, soil surveys describe the A, B, and C soil horizons in the study area. In the absence of field measurements, these surveys offer information on suitable thicknesses for each soil layer in the Richards Equation model. Although [Downer \[2002\]](#) demonstrated that the GSSHA model's sensitivity to soil layer depth is minimal, it is advisable to adopt a conservative approach by establishing deeper soil columns than necessary [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki, n.d.](#)]. However, this may increase computation time, so experimenting with soil layer depths to minimize the simulated soil amount, if feasible, could be beneficial. When the saturated zone constitutes the lower boundary, the region between the defined soil column and the water table is as-

signed the parameters utilized in the third, C, layer [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki, n.d.](#)].

Another crucial consideration in the application of the Richards Equation is vertical cell discretization. In essence, smaller cells yield better results, albeit with extended simulation times. Extremely small cells, measuring 1 cm or less, are typically necessary in the top 10 cm of the soil column [[Downer, 2002](#); [Van Dam and Feddes, 2000](#)]. At greater depths within the soil column, larger cells may be employed. The cell size increase between layers should be gradual. The horizontal dimensions of the soil cells are the same as the ground surface cells.

These necessary parameters for generating soil map input files for GSSHA simulations were available in the SSURGO database but required navigating multi-level linkages in the SSURGO database. Several processing steps were carried out using various software tools such as MS Excel Spreadsheet, MATLAB[®], and ArcGIS, to generate soil data input files for GSSHA simulation. The process involved multiple stages of data manipulation, including the integration of spatial and non-spatial data, transformation of raw data into a usable format, and quality checks to ensure the accuracy and consistency of the final product for GSSHA simulation.

5.4 Methods

This section, discusses the methods applied to perform analysis using GSSHA. This research used the free version of GSSHA, which can be downloaded from the official GSSHA website [[Gridded Surface Subsurface Hydrologic Analysis \(GSSHA\) Wiki, n.d.](#)]. Due to

budget constraints, this study was conducted without the proprietary graphical user interface (GUI) called the Watershed Management System (WMS) developed by Aquaveo. As a result, all input files were entirely created manually using text (.txt) files, and the model and simulations were executed through the Windows Command Prompt, also known as CMD. Additional software, including ArcMap, MATLAB®, and Excel were used to assist in creating model input and output files; The overall structure of this chapter is illustrated in Figure 5.5:

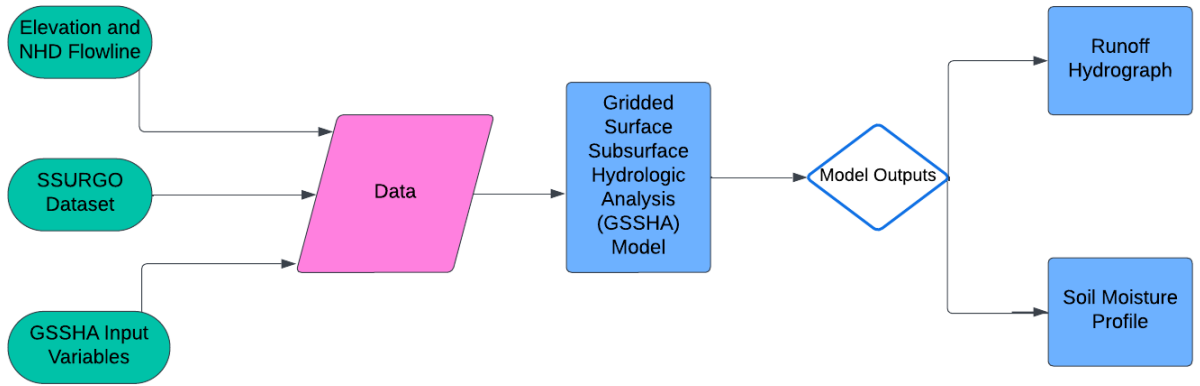


Figure 5.5: Summary of Modeling Steps

5.4.1 Jones Falls (ID: 1589440) Watershed Delineation

To create a fully distributed GSSHA model simulating Jones Falls watershed, the first step taken was downloading digital elevation data in Tagged Image File Format (TIFF) from either USGS National Map [[U.S. Geological Survey \(USGS\), n.d.b](#)] or EarthExplorer [[U.S. Geological Survey \(USGS\), n.d.a](#)]. It was noted that the elevations are in meters and referenced to the North American Vertical Datum of 1988. The elevation data are provided

in Geographic (Latitude-Longitude) coordinates, North American Datum (NAD) 1983, at 1/3 arc-second resolution.

Next, the DEM was clipped to the desired size using the data management tools in ArcMap. The DEM was projected to NAD 1983 Stateplane Maryland FIPS 1900 in meters, and the cell size was set to 10×10. Resampling or interpolation was applied using the “nearest” method, with registration values of x=0 and y=0. To project the raster, the data management tool was employed by using the following functions: Projection and Transformation > Raster > Raster Projection.

The elevation raster for Jones Falls watershed was burned using the ArcMap tools. The streams and rivers inferred from the watershed elevation data might not lie in the correct location. That is why the “burning in” process should be applied. The process of “burning in” streams involves lowering the digital elevation of pixels where streams are known to exist. The stream network, watershed, and subwatersheds derived from the altered DEM should be in better agreement with the known locations and overall are more accurate.

Standard GIS terrain processing steps were applied to delineate the watershed: First, depressions were filled, pixel flow direction was determined from the corrected (filled) elevation raster, and flow accumulation calculated for each pixel. The GIS watershed tool was applied to identify all elevation pixels that drain to the Sorrento gauge location. The delineated watershed with elevation map is illustrated in Figure [5.3](#).

5.4.2 Location of Aquifer Bottom Elevation

GSSHA simulates both saturated and unsaturated subsurface flow. A digital representation of the saturated zone is required. Located between the Blue Ridge and Coastal Plain provinces in Maryland, the Piedmont physiographic region is primarily underlain by crystalline rocks of metamorphic and igneous origin, with a lesser quantity of sedimentary rocks present. This region, subject to historical geological processes, has experienced varying degrees of folding, faulting, and fracturing, and is known as of fractured-rock terrane [Nutter and Otton, 1969].

The demarcation line between the Piedmont and Coastal Plain provinces, known as the Fall Line, distinguishes the robust, fractured rock composition of the Piedmont from the unconsolidated sediments of the Coastal Plain.

Contrary to the Coastal Plain sediments, where ground water permeates the interstices between sediment grains, the aquifers within the Piedmont consist of closely packed mineral grains that resist water transmission. Despite this, fractures that have formed in the rocks over geological timescales are capable of channeling sufficient amounts of water to a well.

The ground water in the Piedmont region of Maryland originates predominantly from local rainfall and snow within the specific watershed where the well is situated. Within the Piedmont, the precipitation infiltrates through the soil and rock strata until it reaches the water table—defined as the depth at which the ground is fully saturated with water. The fully saturated region beneath the water table is referred to as an aquifer.

Predominantly, the Piedmont aquifers are unconfined, or water-table aquifers, imply-

ing they lack an overlying impervious layer to shield the ground water from contamination from surface sources. The water table signifies the upper limit of the unconfined aquifer.

GSSHA requires an aquifer bottom as a boundary condition; this surface represents the depth below which the presence or movement of groundwater does not contribute to the watershed's water balance. The elevation of the aquifer bottom was determined entirely based on the rationale of the typical location of an aquifer from the ground elevation, and as such, it was considered an arbitrary value. Various factors, such as the geology and topography of the area, were taken into consideration at a reasonable estimate for the aquifer bottom elevation. However, it should be emphasized that this value was not determined through direct measurement or empirical data from the existing literature. A 3-D view of the aquifer bottom is depicted in Figure 5.6. The purple color denotes the location of the aquifer, the grey color is watershed elevation, and the blue color is water table initial condition. The aquifer bottom was estimated by constructing a surface 100 meter below the ground surface, then smoothing it using GIS tools.

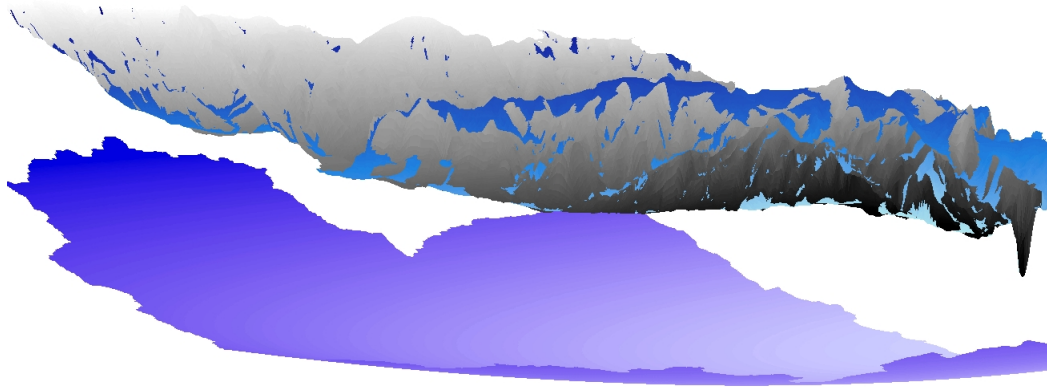


Figure 5.6: Jones Falls Watershed Aquifer Bottom Location in 3D Format

5.4.3 Computation of Water Table

Water table elevation is a distributed, time-varying quantity in GSSHA. Therefore an initial condition is required for the analysis. The minimum water table data in SSURGO database was selected as initial condition. This parameter represents the annual minimum of the water table, indicating the closest observed distance between the water table and the ground surface [[U.S. Department of Agriculture, Natural Resources Conservation Service \(USDA NRCS\), n.d.](#)]. Where the water table depth variable was not available in the SSURGO data, the initial water table was placed 2 meter below the ground elevation.

5.4.4 Richards Equation: Infiltration and Redistribution

The acquisition and preprocessing of soil data from the SSURGO database were integral steps prior to its usage in the GSSHA model for the Jones Falls watershed. The initial raw data, “Muaggatt.txt” file, was downloaded directly from the SSURGO database. Subsequently, this raw data was subjected to a series of procedures executed via an Excel spreadsheet. These operations encompassed a variety of tasks, such as data cleaning, processing, and reformatting to ensure the reliability and consistency of the data. Through these steps, the initial raw data was transformed into a comprehensible and usable format that could effectively integrate with the GSSHA model for the Jones Falls watershed.

5.4.5 Infiltration:Parameter Estimates

Numerous studies have been performed to connect parameters of soil infiltration to texture classification. Among prior research works, [Rawls et al., 1983a; Rawls and Brakensiek, 1985; Rawls et al., 1982, 1983b] are highly relevant literature. In Table 5.3 a summary of Rawls and Brakensiek soil parameter estimates as a function of United States Department of Agriculture (USDA) textural classification is illustrated.

The soil parameters in the Table 5.3 derived from the geometric mean of tests based on multiple soil samples. These values have a high variance, standing for high uncertainty or low correlation between textual classification and soil texture. Nonetheless, those values can be helpful since they provide an initial estimate of infiltration parameters. The variances for those values in Table 5.3 are provided in [Maidment, 1993].

Table 5.3: Rawls & Brakensiek Soil Parameter Estimates [Rawls et al., 1982]

USDA Textural Classification	θ_s (cm ³ /cm ³)	θ_e (cm ³ /cm ³)	θ_f (cm ³ /cm ³)	θ_{wp} (cm ³ /cm ³)	θ_r (cm ³ /cm ³)	ψ_b (cm)	λ (cm/cm)	K_s (cm/h)	ψ_f (cm)
Sand	0.437	0.417	0.091	0.033	0.02	7.26	0.694	23.56	4.95
Loamy sand	0.437	0.401	0.125	0.055	0.035	8.69	0.553	5.98	6.13
Sandy loam	0.453	0.412	0.207	0.095	0.041	14.66	0.378	2.18	11.01
Loam	0.463	0.434	0.27	0.117	0.027	11.15	0.252	1.32	8.89
Silt loam	0.501	0.486	0.33	0.133	0.015	20.79	0.234	0.68	16.68
Sandy clay loam	0.398	0.33	0.255	0.148	0.068	28.08	0.319	0.30	21.85
Clay loam	0.464	0.39	0.318	0.197	0.075	25.89	0.242	0.20	20.88
Silty clay loam	0.471	0.432	0.366	0.208	0.04	32.56	0.177	0.20	27.30
Sandy clay	0.43	0.321	0.339	0.239	0.109	29.17	0.223	0.12	23.90
Silty clay	0.479	0.423	0.387	0.25	0.056	34.19	0.15	0.10	29.22
Clay	0.475	0.385	0.396	0.272	0.09	37.30	0.165	0.06	31.63

5.4.6 Stream Network for Jones Falls GSSHA model

A GSSHA stream network consists of a set of connected links made up of nodes. Within the GSSHA framework, each link must contain a minimum of three nodes. Constructing a link with only two nodes can lead to instability in the model, ultimately resulting in a simulation failure.

Streamlines were created using National Hydrography Dataset (NHD) [[U.S. Geological Survey \(USGS\), n.d.c](#)] database and projected to Maryland State Plane (MSP) coordinates. Two small features (stream lines) were removed from the NHD data. The links for the main stream (Jones Falls at Sorento) and the tributaries were numbered following GSSHA convention, with link 1 being the most upstream link and link 69 being the most downstream (connecting to the outlet).

Each polyline of the projected NHD flowline was split into line segments about 100-meter long, equally spaced. Care was taken to ensure that each link had at least two segments and three nodes. Nodes were numbered and assigned their X and Y coordinates and elevation (Z) using GIS tools.

The stream vertices and their connectivity, as generated in GIS, were exported and manipulated using spreadsheet tools to generate the required stream geometry input file for GSSHA.

The bottom depth and width for each link were calculated using the McCandless regression equation for streams in Maryland Piedmont region [[McCandless, 2002](#)], using the drainage area that belonged to the second last vertex on a link to avoid streamflows of other links contributing. The thalweg elevation was determined by subtracting the bankfull

depth from the elevation for each node, checking that the elevation in each node is lower than the upstream node. If the elevation of the upstream node was lower, a 0.1-meter change in node elevation was enforced.

The GSSHA channel input file was formatted as required for GSSHA's 1-D channel calculation (Section [5.2.1.2](#)).

5.4.7 Associating Flow Accumulation with Points on the Stream Network

GSSHA simulates both overland flow and subsurface flow (baseflow) into the stream segments. It is necessary to specify the connectivity of raster cells to stream segments. For each stream link, the landscape (raster) cells that contribute flow must be identified. GIS polygon, polyline, point, and raster analysis and spreadsheet tools were used to generate the required input text file.



Figure 5.7: 3D view of the physiographical model of Jones Falls watershed on Google Earth Image located in Baltimore County, Maryland

5.4.8 Interception Input

In GSSHA, vegetation's ability to intercept precipitation is simulated using the two-parameter method developed by [Gray, 1970]. The initial amount of rainfall (in millimeters) that is fully intercepted by foliage is defined by the STORAGE_CAPACITY. After the storage capacity is met, the INTERCEPTION_COEFF indicates the proportion of rainfall retained as intercepted water. Storage capacity and interception coefficient were assigned using an index map based on vegetation.

GSSHA employs the interception formula provided below to model the interception of rainfall by vegetation:

$$Interception = a + bP^n \quad (5.16)$$

where a signifies the storage capacity, which has the same dimensions as precipitation (mm or inches), b represents the dimensionless interception coefficient, and n is typically set to 1.0. The a and b values are taken from Bras [1990]. Table 5.4 summarizes the description of each Land Cover (LC) value and Table 5.5 lists the values assigned to each LC type:

Table 5.4: Land Cover (LC) values and their descriptions

LC	Classification Description
11	Open Water
21	Developed, Open Space
22	Developed, Low Intensity
23	Developed, Medium Intensity
24	Developed, High Intensity
31	Barren Land (Rock/Sand/Clay)
41	Deciduous Forest
42	Evergreen Forest
43	Mixed Forest
52	Shrub/Scrub
71	Grassland/Herbaceous
81	Pasture/Hay
82	Cultivated Crops
90	Woody Wetlands
95	Emergent Herbaceous Wetlands

Table 5.5: Interception Coefficients for Various Land Cover (LC)

LC	a	b	LC	a	b	LC	a	b
11	0	0	21	1	18	22	1	18
23	10	18	24	10	18	31	0	0
41	10	18	42	10	20	43	10	18
52	5	15	71	1	8	81	3	10
82	4	5	90	10	18	95	5	18

5.4.9 Surface Roughness Input

To compute the surface roughness for GSSHA input, the land cover and impervious fraction maps were obtained from the National Land Cover Database (NLDC), provided by the Multi-Resolution Land Characteristics Consortium [[MRLC Consortium, n.d.](#)]. The rasters were projected to Maryland State Plane (MSP) and were clipped to the extent of the Jones Falls watershed. The following formula is used to compute surface roughness for each pixel:

$$n_{pixel} = fraction_{imp} \times n_{imp} + (1 - fraction_{imp}) \times n_{pervious} \quad (5.17)$$

The parameter n_{imp} represents a surface roughness factor. If n_{imp} is not a function of the impervious descriptor provided by NLDC ($imp_{descriptor}$), a constant value such as 0.011 can be assigned. If there is a need to incorporate the $imp_{descriptor}$ into the model, appropriate rules will need to be established to capture its influence. The parameter $n_{pervious}$, which represents the roughness factor for pervious surfaces, should ideally be a function of the land cover. This information can be derived from a lookup table linked to the land cover index map. Such a linkage ensures that the variations in land cover are accurately reflected in the $n_{pervious}$ values, thereby improving the overall representation of surface roughness within the hydrological model.

5.5 Precipitation

The present study adopted uniform spatial and temporal precipitation input. An intensity of 80 mm/hr and duration of 10 minutes were employed as the default input precipitation. This is a typical annual maximum 10-minute precipitation intensity for this location [Perica et al., 2018]. Given the complex and inconsistent patterns encountered during the simulation steps, altering the predetermined precipitation intensity and duration proved to be infeasible.

5.6 Model Timestep

The overall duration of the model run was set to 360 minutes. The timestep was set at a minimal interval of 2 seconds.

5.7 Simulations

This study involved two simulations, distinguished by their respective soil profiles - one employing spatially distributed soil type, the other a spatially uniform soil type. The distributed soil map comprised 73 unique soil indices. Each soil profile has 3 horizons as discussed earlier.

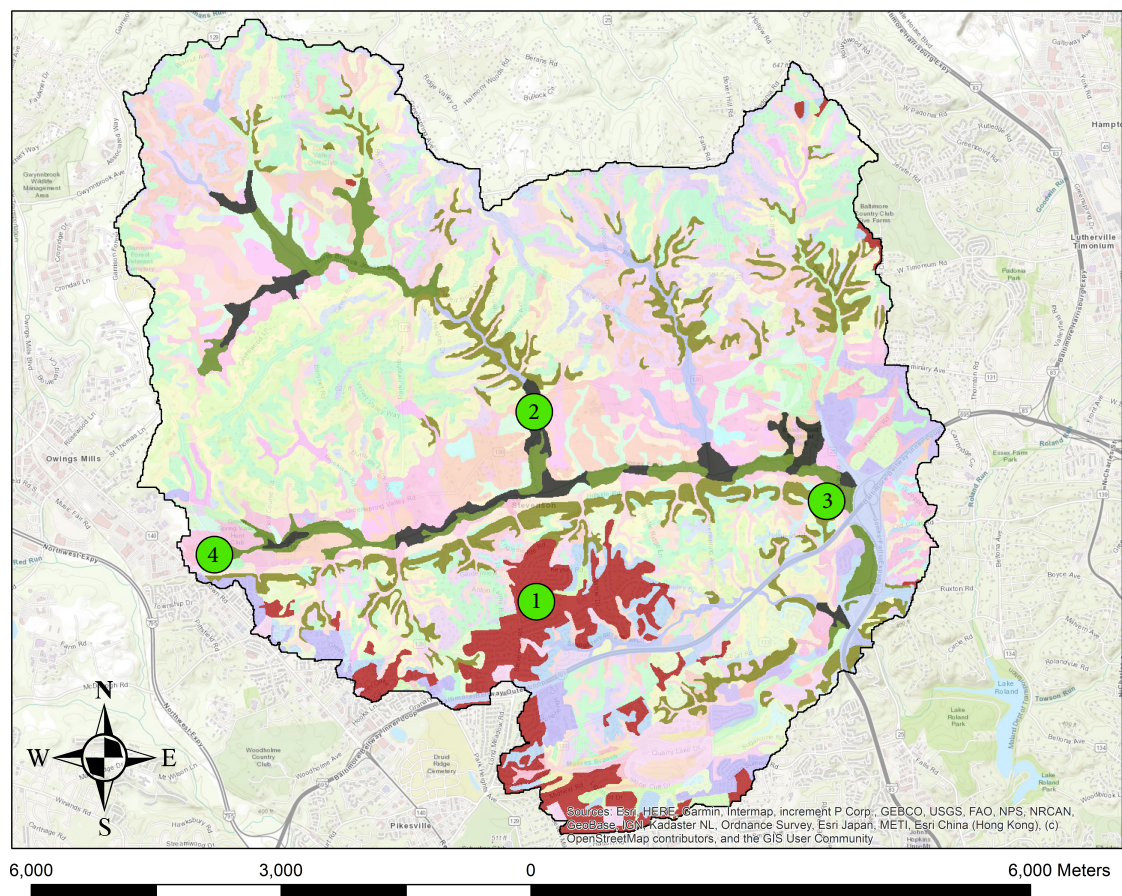


Figure 5.8: Four Dominant Soil Indices in Jones Falls Watershed in Maryland

In contrast, the uniform soil profile applied soil index 28 to the entire watershed. The relative dominance of this soil type throughout the watershed was the basis for the uniform simulation run. All soil profiles were initialized at field capacity, which varies among soils and across soil horizons within a given soil, as specified in the soil properties parameter input file.

GSSHA simulates vertical flow in the soil at each model pixel, solving the Richards equation numerically with the parameters and discretization assigned. The user may provide instructions for the model to provide detailed soil profile output at specified locations.

Four locations were selected for detailed output in the distributed-soil simulation. The selection of the four locations was governed by the dominance of the specific soils in these regions within the watershed. Four dominant soil types in Jones Falls watershed with IDs 24, 28, 30, and 31 were selected as examples for the analysis. The chosen locations are referenced in Figure 5.8, with location 1 linked to soil index 24, location 2 associated with soil index 28, location 3 corresponding to soil index 30, and location 4 matched with soil index 31. In the uniform soil simulation, all four locations were assigned to soil index 28.

5.8 Results

In this section, the simulation output for distributed soil and uniform soil scenarios is presented. The distributed model incorporates 73 different soil types, spatially varying over the watershed. In contrast, the uniform model applies a single soil type (ID 28) consistently across the entire watershed. The outflow hydrographs, simulating flow at the Jones Falls Sorrento gauge, are shown in Figure 5.9. The distributed soil model exhibits a lower peak discharge ($76.6 \text{ m}^3/\text{s} = 2,705 \text{ ft}^3/\text{s}$) than the uniform soil model ($84.5 \text{ m}^3/\text{s} = 2,984 \text{ ft}^3/\text{s}$). Certain unanticipated oscillations in the hydrographs were noted, for which there appeared to be no immediate explanation. Tail of hydrographs do not decline to near-zero baseflow within the time span of the simulation. The peak time for the hydrograph with a uniform soil profile is approximately 98 minutes, while for the hydrograph with a distributed soil profile, it occurs at approximately 94 minutes. Considering a rainfall event duration of 10 minutes, the time span between the end of precipitation and the peak of the hydrograph for the uniform soil profile is approximately 88 minutes, whereas for the distributed soil

profile, it is roughly 84 minutes. This difference in peak discharges highlights the distinct hydrological behavior of distributed and uniform soil systems.

GSSHA allows a detailed examination of water movement into the unsaturated zone. Examples are shown in Figures 5.10 & 5.11. The four locations in the Figure correspond to 4 different soil indices, as described in Section 5.7. The Figure shows that volumetric soil moisture changes continuously for each location and it is different for each soil profile. The depth of each horizon is different for the distributed soil profile.

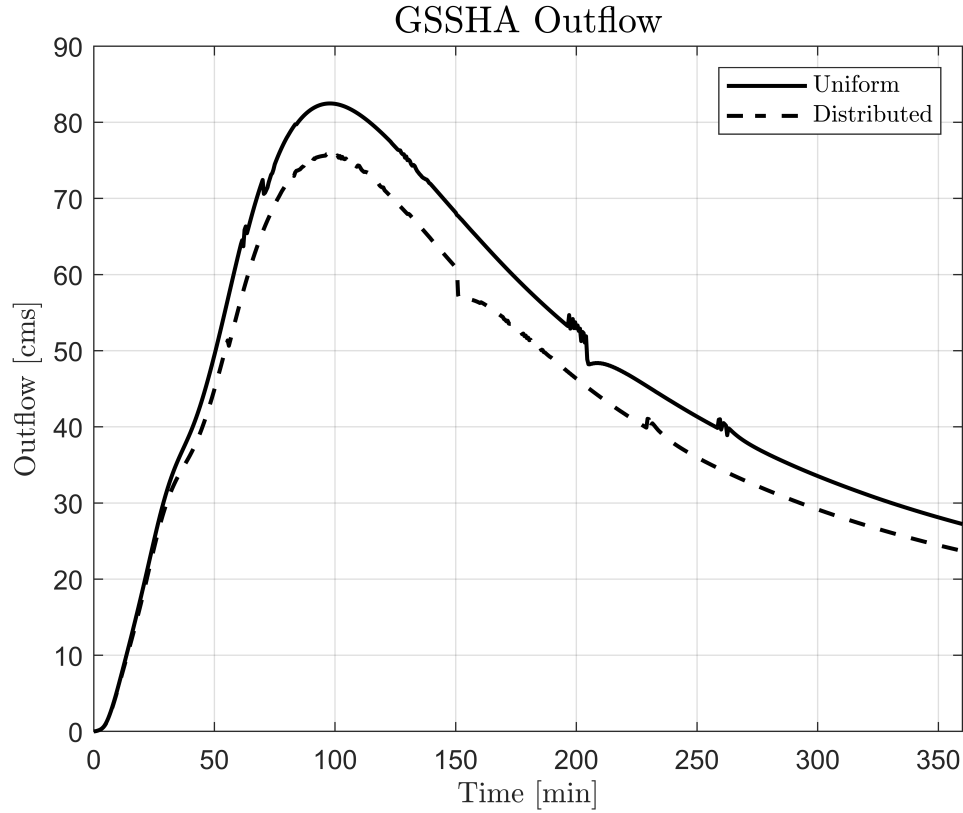


Figure 5.9: GSSHA Outflow for Distributed Soil vs. Uniform Soil

The recording interval was set at 30 seconds (every 15 simulation time steps) to generate soil profiles at each location. Given that the total simulation duration is 360 minutes,

a total of 721 profiles were captured for each location, inclusive of the initial condition (i.e., the simulation at time 0). The sequence of wetting from the surface, subsequent downward redistribution, then a drying cycle after precipitation ceases, can be observed. The soil moisture behavior varies across the different locations, which can be attributed to the variations in soil texture and soil structure at these locations and their respective depths.

Figure 5.11 illustrates the soil profile response at the same four locations, but with all soils uniformly assigned the parameter set corresponding to soil ID 28. Although their parameters are identical, their responses to the precipitation event are different because of their different locations in the watershed. These example results illustrate the level of detail that can be provided by GSSHA and underscore the importance of considering spatial heterogeneity in soil properties when analyzing soil moisture dynamics.

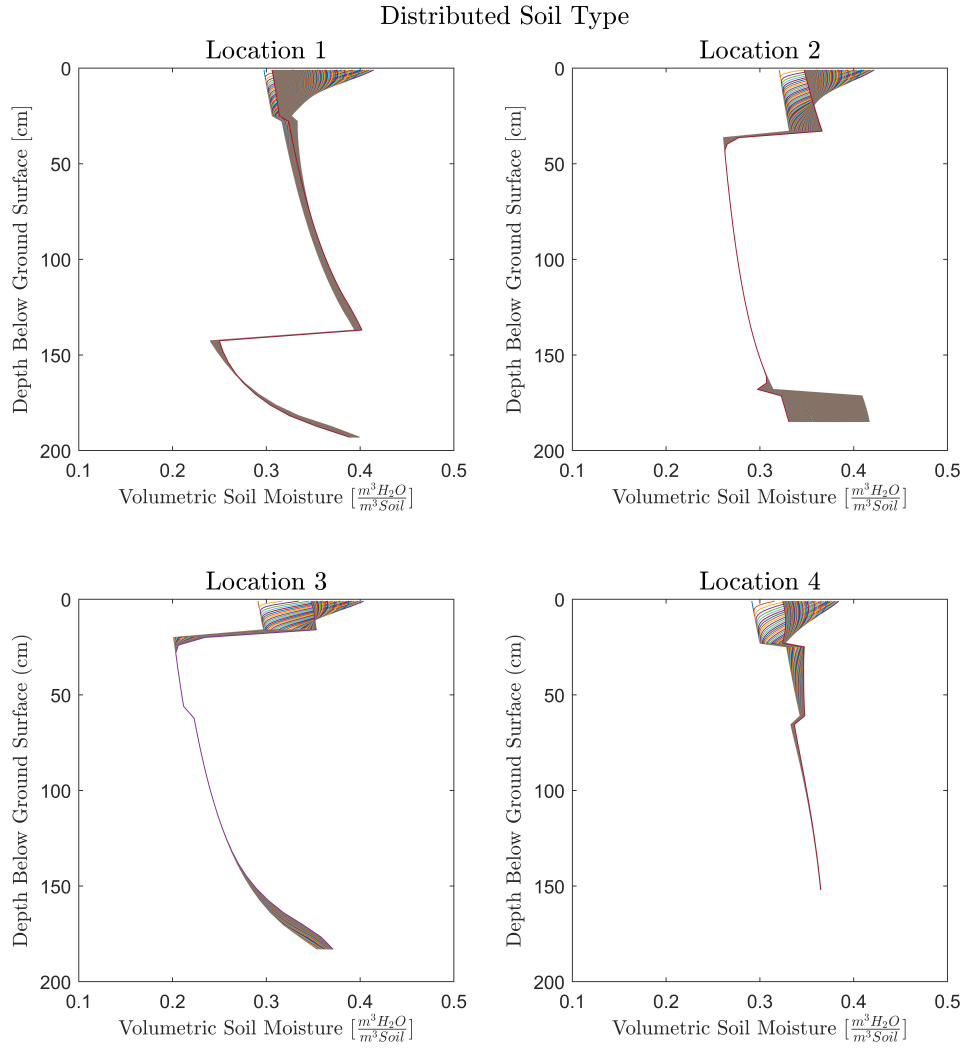


Figure 5.10: Distributed Volumetric Soil Moisture Profile in 4 selected locations in the Jones Falls Watershed (Location 1: Soil Index 24, Location 2: Soil Index 28, Location 3: Soil Index 30, Location 4: Soil Index 31)

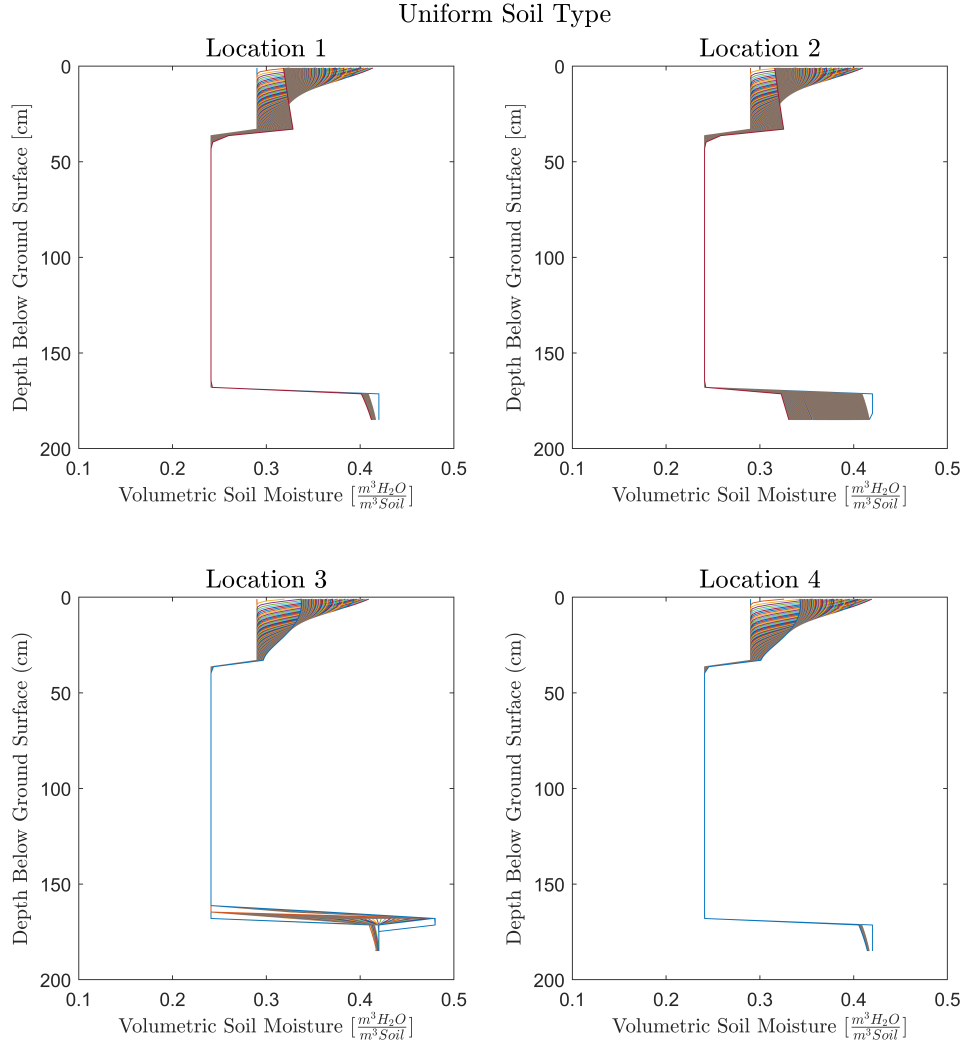


Figure 5.11: Volumetric Soil Moisture Profile in 4 selected locations in Jones Falls Watershed with spatially uniform soil properties

Figure 5.12 illustrates the result from the mass balance output of the GSSHA model. Net Infiltration is larger in the uniform soil simulation than in the distributed soil simulation. Richards Equation mass balance error for the simulation with uniform soil is 1.08% and for the simulation with distributed soil is -1.41%. It is evident that the large portion

of streamflow is generated from groundwater in this simulation. Around 10% of the precipitation volume is intercepted by vegetation and 88% of the precipitation infiltrates into the soil. Therefore, nearly all of the simulated precipitation was intercepted or infiltrated. An attempt was made to increase direct (overland) runoff (DRO) by escalating the uniform precipitation intensity and/or duration, however, numerical instabilities were encountered. A further attempt to boost DRO by rendering the soil initial condition more humid was also unsuccessful. The process was constrained by the extensive time required to procure simulation results.

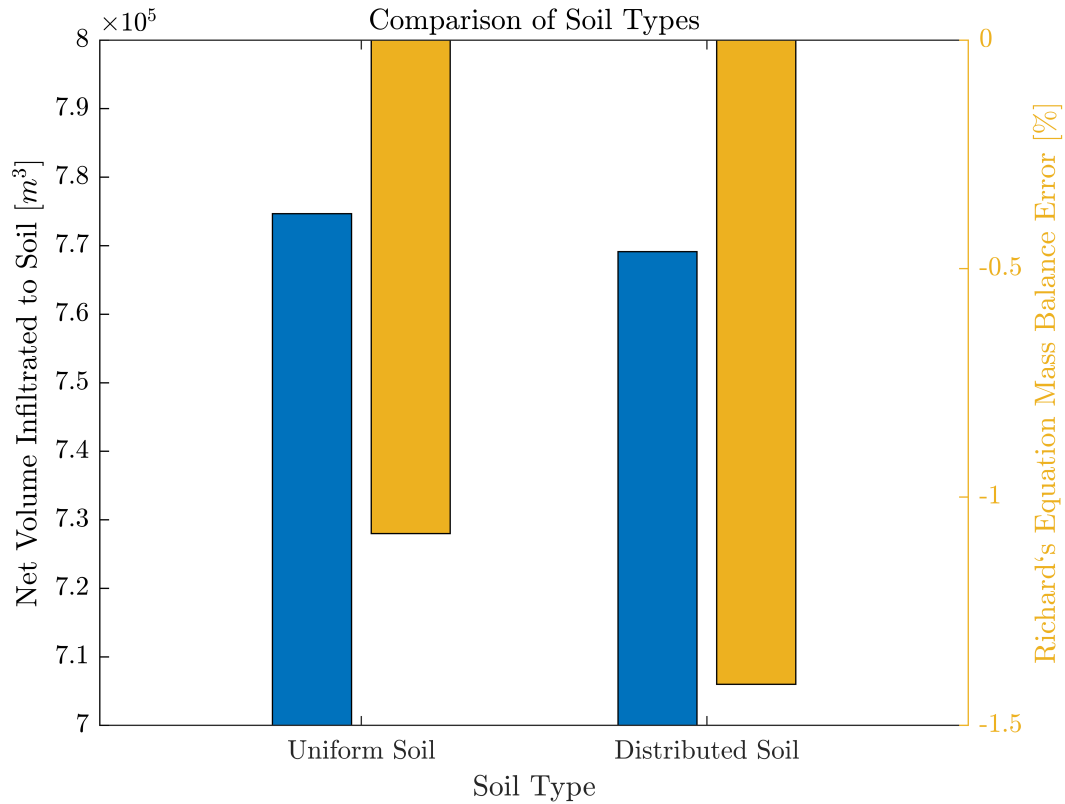


Figure 5.12: GSSHA Summary Comparison Between Uniform Soil and Distributed Soil

5.9 Concluding Remarks

In the course of this research, a fully distributed hydrologic simulation was assembled and executed. The simulation used a spatial resolution of 10 meters and a vertical resolution of 1 centimeter within the soil column. Assembling and running a finely-resolved, fully-distributed coupled surface unsaturated and saturated zone model of the Jones Falls watershed was a major accomplishment. However, given the complexities and simulation time required, the original objective of evaluating the spatially distributed rainfall from a real event on the Jones Falls watershed and comparing the results with hydrographs generated by the NRCS unit hydrograph method was not achieved. Consequently, the focus shifted to assessing runoff hydrographs on the GSSHA model using both uniform and distributed soil data.

The comparison of peak discharges from these two approaches underscored the different hydrological behavior of distributed and uniform soil systems, as exemplified by their respective hydrographs. This observation suggests that incorporating detailed soil data can provide insight into watershed processes and possibly enhance the accuracy of runoff estimation. Also, nearly all of this streamflow comes from the groundwater in this simulation. In summary, while the initial goal was not met, this analysis highlighted the potential benefits of utilizing comprehensive soil information.

GSSHA is available in both Windows and Linux versions. Colleagues at the University of Maryland center of High-Performance Computing Clusters (HPCC) made an effort to install the Linux version on the Deepthought2 cluster, but it was not possible to fully test or optimize the software. In initial tests, running the model with identical inputs gave

slightly different results from the Windows run, and did not significantly improve the run time. Deepthought2 was decommissioned in January 2023 and replaced by the Zaratan system and the dissertation author did not have the opportunity to test GSSHA on the Zaratan system.

The study took advantage of the United States Geological Survey (USGS) digital elevation model and the National Hydrography Dataset (NHD) stream geometry. These resources enabled the linkage of overland and subsurface flow to a distributed, one-dimensional unsteady-flow stream model with a 100-meter resolution.

Soil horizon properties vital to the model were extracted from the Soil Survey Geographic (SSURGO) database, a comprehensive resource that offers detailed information on soil types and properties. The integration of these varied datasets and tools in the model provides the potential for a robust analysis of hydrologic processes within the study area.

The GSSHA model developed in this study has not been calibrated to any specific real-world event. Furthermore, the initial conditions employed for this model, as stated earlier in the chapter, were selected fairly arbitrarily. These factors should be taken into consideration when interpreting the results and their potential implications.

5.10 Future Research

This study was initially designed to investigate the uniform distribution of rainfall over the Jones Falls watershed in Baltimore, Maryland, with the primary goal of evaluating the simplifying assumptions of the unit hydrograph approach, which assumes a spatially uniform rainfall distribution across the entire watershed, as well as linear wa-

tershed response. Distributed models, which do not rely on parameters such as the time of concentration—often defined subjectively in existing literature—may offer more robust and accurate estimations of runoff and peak discharge in response to specific rainfall events. Future research could involve simulating runoff utilizing spatiotemporal rainfall data and comparing the resultant hydrographs with those generated by the NRCS unit hydrograph methodology. This comparison may shed some light on inherent uncertainties associated with the NRCS unit hydrograph approach and contribute to the development of improved hydrological modeling techniques to simulate runoff. Practitioners are likely to continue using expedient methods like UH models for prediction and design – whether the NRCS curvilinear UH or some other model. Detailed process-based modeling could help to provide guidance in the application of those methods.

If time and resources allow, investment in the proprietary Graphical User Interface (GUI) for GSSHA could be useful in assembling spatial and parameter data sets for future modeling efforts.

Chapter 6: Investigating Unit Hydrographs with a Simple Distributed Model

6.1 Introduction

This study set out to determine the physical and geometric properties of watersheds that affect the Peak Rate Factor (PRF) and peak discharge in the unit hydrographs (UH) generated by GSSHA (Gridded Surface Subsurface Hydrologic Analysis). Inferring the UH from actual events (Chapters 3 and 4) requires hydrograph and hyetograph separation; these steps are somewhat subjective and uncertain. Literal unit-hydrograph events (response to 1 inch or 1 cm of rainfall uniformly occurring over the watershed) cannot be found in the record. The attempt to build a distributed model of an actual watershed on which such events could be simulated (Chapter 5) was found to be prohibitively complex and required additional resources. Thus, the strategy employed in this chapter is to create distributed models of highly simplified artificial watersheds whose properties can be controlled, subject them to unit precipitation events, and observe the response hydrographs.

The main goal of this chapter is to test the effects of various parameters (predictors) on both peak discharge and the PRF to identify quantitative associations. The GSSHA model is implemented to simulate the hydrologic behavior of an open book synthetic watershed under various hydrological conditions. By varying parameters such as surface roughness, rainfall pattern (e.g., duration and intensity), grid size, watershed geometry (e.g., side slope

and longitudinal slope) the sensitivity of peak discharge and PRF to these parameters is assessed. This analysis provides insights into the relationships of selected watershed characteristics to PRF and peak discharge, which can be valuable for flood risk management, water resources planning, and environmental management.

The open-book (V-shape) synthetic watershed can provide valuable insight for hydrologic research and analysis. The V-shape geometry of the synthetic watershed represents an idealized scenario for evaluating hydrologic processes and their impact on discharge characteristics. The specific physical characteristics of the channel network within the watershed are not taken into account in this simplified watershed due to the lack of a channel model. Additionally, the complex movement of water through the soil is not modeled because an infiltration model is absent.

In summary, a simplified representation of a real-world watershed is provided by the synthetic open-book watershed model, which omits some features consisting of infiltration model and channel routing. Nonetheless, it can still offer valuable insights into the water balance and hydrological processes of the watershed and be utilized as a useful tool for research and analysis.

6.2 Research Questions

In this section, research questions to assess the impact of watershed physical and geometric properties (e.g., side slope, longitudinal slope, surface roughness) on peak discharges and PRF are discussed.

1. How do gridsize and timestep affect the convergence and stability of the results in

a simple, open-book watershed with temporally and spatially uniform rainfall and no infiltration? How small should the gridsize be to obtain stable and converging results?

2. How do different surface roughness values, representing various land cover types, affect the runoff response, PRF, peak discharge, time to peak, and overall hydrograph shape in a simple, open-book watershed with temporally and spatially uniform rainfall and no infiltration?
3. How do different watershed side slopes affect PRF, peak discharge, time to peak, and overall hydrograph shape in a simple, open-book watershed with no infiltration?
4. How do different watershed longitudinal slopes affect PRF, peak discharge, time to peak, and overall hydrograph shape in a simple, open-book watershed with no infiltration?
5. How does the varying intensity and duration of temporally and spatially uniform rainfall events influence the PRF, peak discharge, time to peak, and overall hydrograph shape in a simple, open-book watershed with no infiltration?

6.3 Method

In this section, an open-book synthetic watershed was generated using a custom algorithm developed in MATLAB®. A series of experiments vary the properties of the simplified watershed and assess the resulting unit hydrograph (UH) as the response to 1 unit (1 cm or 10 mm) of rainfall. This model operates under the presumption of simplified

watersheds, which are designed without any interception or surface or subsurface storage. Consequently, in this model, Precipitation (P) is equated with Precipitation Excess (PEX), indicating that all precipitation falling on the watershed is accounted for as direct runoff. Figure 6.1 depicts the open-book watershed and Figure 6.2 illustrates the contours of the open-book watershed with geometry assigned for the default simulation.

6.4 Properties of Artificial Watersheds

The open-book watershed has dimensions of 5.1×5.1 [km²], with 51 rows and 51 columns which results in a total area of 26,010,000 [m²]. The longitudinal slope is initially set at 20% (represented as $s_0 = 0.20$), and the side slope is defined at 20%. The simulation utilized a grid resolution of 100 meters originally. These values were chosen arbitrarily for preliminary experiments and they were modified based on the simulation preference to test several scenarios.

6.4.1 Surface Roughness

Drawing upon existing literature, a range of surface roughness values was chosen to examine the influence on the unit hydrograph's shape, time to peak, peak discharge, and PRF. According to [Arcement and Schneider, 1989; Chow, 1959], the typical range for Manning's roughness coefficients for surface (sheet) flow spans from 0.1 to 0.4. This selection of roughness values allows for a comprehensive assessment of how variations in surface roughness characteristics may affect the hydrologic response, hydrograph shape, time to peak, peak discharge, and PRF.

6.4.2 Simulated Rainfall

Total rainfall depth is 1 unit (e.g., 1 cm or 10 mm). Rainfall intensity and duration are two experimental variables in the series of simulations. Combinations of varying intensities and durations were selected to produce rainfall depths (volumes) of 1 unit, representing simulated unit hydrographs for the experiments.

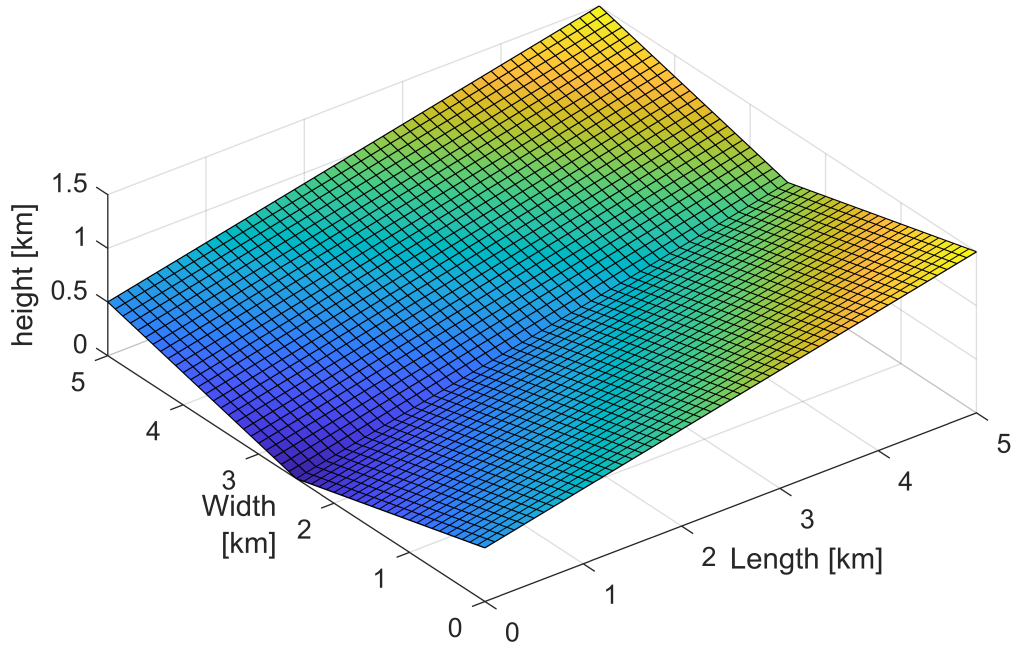


Figure 6.1: Example of open-book watershed created in MATLAB®

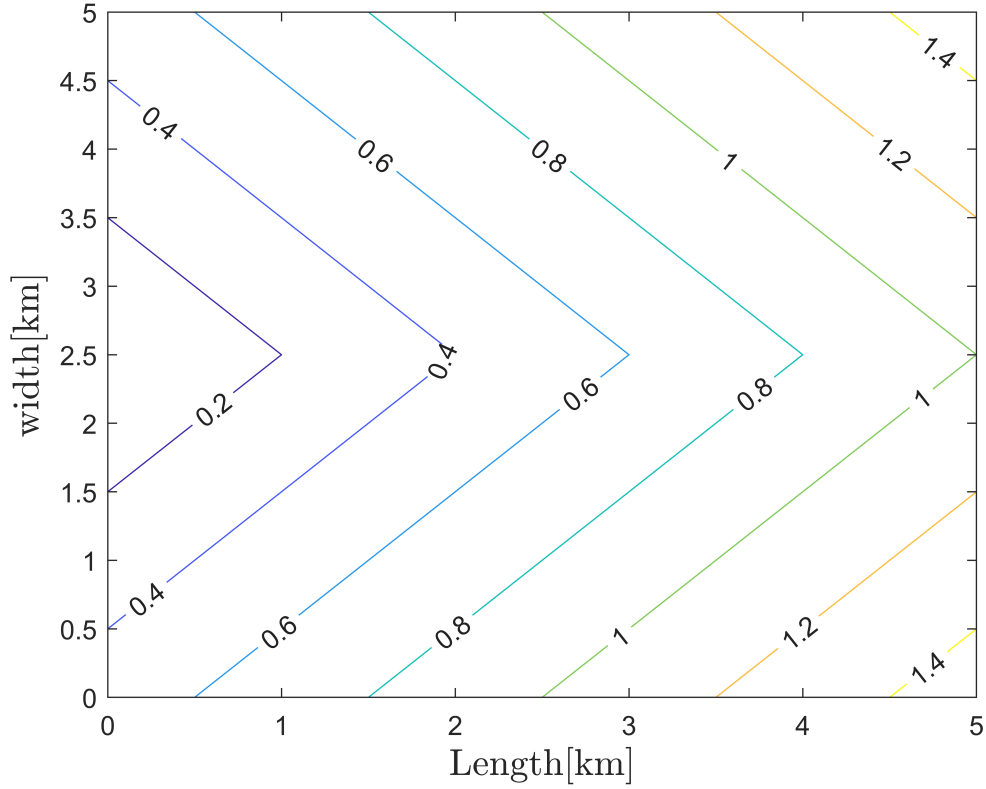


Figure 6.2: Elevation contours of example open-book watershed

6.4.3 Creation of GSSHA Input Files

- **Elevation File:** The .ele file, representing the elevation data, was generated using MATLAB[®]. The output is a text file providing elevation at the center of each grid cell.
- **Watershed Mask:** The watershed file is a binary (0,1) matrix, assigning a value to each grid cell, 1 if the cell is in the watershed, 0 if not. For the rectangular open-book watersheds used in this experiment, the mask contains no zeros.
- **Grid Resolution:** The simulation utilized a grid resolution of 100 meters originally.

This resolution was deemed suitable for the scale of the study area and the level of detail in the available data. Courant Condition analysis is used later in this chapter to assess the stability and convergence of the GSSHA for grid resolution of 100 meters.

- **Simulation Duration:** The total duration for these simulations was set to 28,800 minutes, equivalent to a period of 20 days. This time frame was selected by observing the rise, peak, and recession of the simulated outflow hydrograph. Considering that the simulated experimental hydrographs demonstrate an asymptotic approach to zero, a period of 20 days was determined as the minimum requisite to capture the majority of the receding limbs, or tails, of experimental hydrographs.
- **Model Timestep:** A model timestep of 1 second was employed, offering a high temporal resolution for capturing the dynamics of the system.
- **Watershed Outlet:** The pixel with the lowest elevation in the .ele file was identified as the watershed outlet. This was set to an elevation of zero.

6.4.4 Observation

The artificial watershed's response to a unit precipitation depth is, by definition, a UH. Scaling the ordinates by peak discharge and the abscissae by time to peak gives a DUH. A simulated experimental DUH, the best-fit gamma function, and the standard NRCS unit hydrograph are illustrated in Figure 6.3. The experimental hydrograph does not match the standard NRCS curvilinear UH with PRF 484, and is not well described by a gamma shape. That indicates the gamma-based equations developed to estimate the PRF cannot be applied for experimental hydrographs.

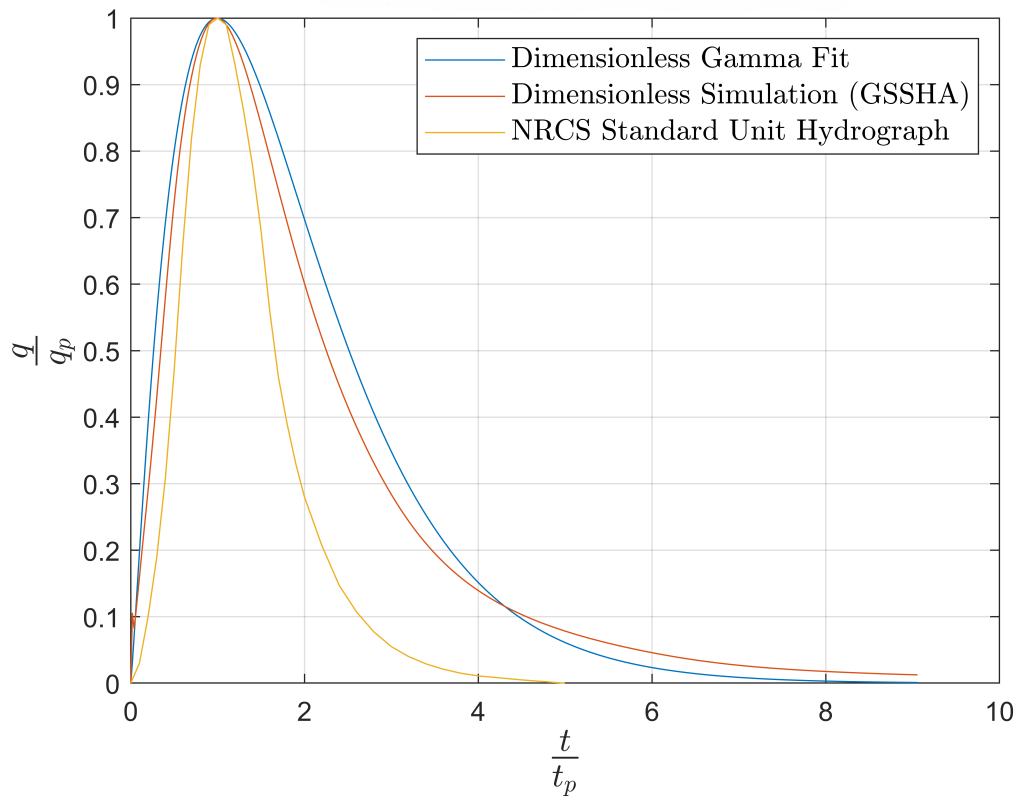


Figure 6.3: Comparison of Dimensionless Unit Hydrographs Generated by Simulation, Gamma Fit, and NRCS Standard Empirical Approach with PRF 484

6.4.5 Peak Rate Factors Derived from Equivalent Triangular Hydrographs and Watershed Parameters

Following [Natural Resources Conservation Service (NRCS), 2007], each curvilinear unit hydrograph is transformed into an equivalent triangular hydrograph by determining the base of the triangle whose area equals the volume of runoff (the integrated curvilinear hydrograph) (Figure 6.4).

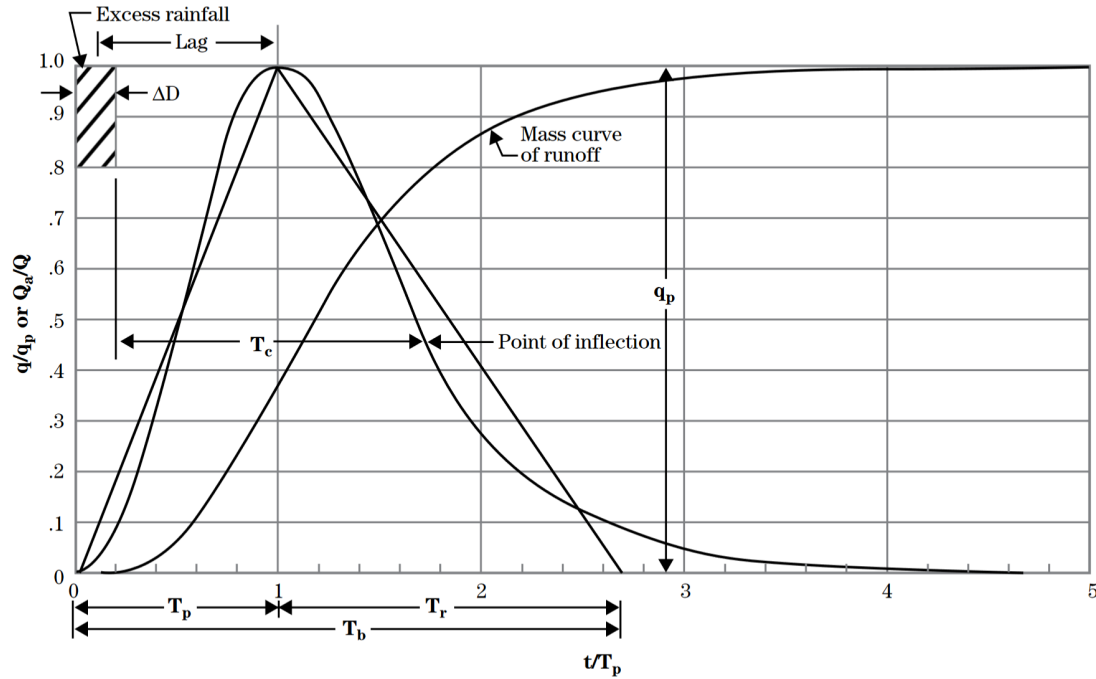


Figure 6.4: NRCS Triangular Hydrograph Based on the NRCS Curvilinear Unit Hydrograph

The geometry of the triangular DUH is determined using a MATLAB[®] algorithm. This algorithm ensures that the total volume of runoff under both the curvilinear and triangular hydrographs remains equal. Figure 6.5 depicts the curvilinear and triangular DUH demonstrating details of the algorithm generated in MATLAB[®]. The same procedure applied in different experiments to obtain PRFs using different watershed parameters in the GSSHA model.

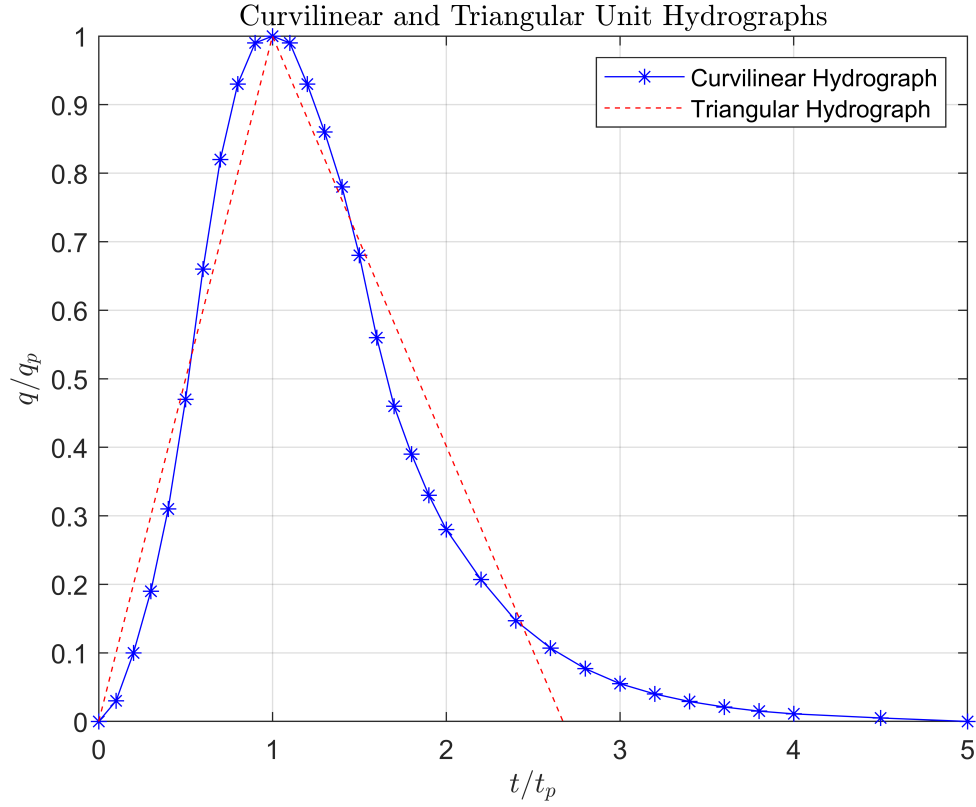


Figure 6.5: Triangular Hydrograph Based on the NRCS Curvilinear Unit Hydrograph

It has been established that:

$$q_p = \frac{645.33 \times K \times A \times Q}{t_p} \quad (6.1)$$

where q_p is peak discharge in $[\text{ft}^3/\text{s}]$, 645.33 is conversion factor for the rate required to discharge 1 inch from 1 square mile in 1 hour, K is nondimensional factor, A is drainage area in square miles $[\text{mi}^2]$, Q is runoff $[\text{in}]$, and t_p is time to peak in hours $[\text{h}]$. Additionally, the “K” factor is given by:

$$K = \frac{1}{\int_0^{t_{*,max}} q_*(t_*) dt_*} \quad (6.2)$$

By construction, the area of the triangle equals the integral in the denominator. The total volume of the water under the triangular unit hydrograph (Figure 6.4) is:

$$Q = \frac{q_p t_p}{2} + \frac{q_p t_r}{2} = \frac{q_p}{2} (t_p + t_r) \quad (6.3)$$

It follows that the peak rate, q_p is:

$$q_p = \frac{2Q}{t_p + t_r} \quad (6.4)$$

In the NRCS DUH, the time to peak has the unit 1, the recession time has the unit of 2/3 or 0.67, and time to base (time to peak + recession time) has the 1.67 unit of time. The K parameter in the NRCS unit hydrograph defined as follows [[Natural Resources Conservation Service \(NRCS\), 2007](#)]:

$$K = \frac{2}{1 + \frac{t_r}{t_p}} \quad (6.5)$$

The equivalent-triangle approach allows estimation of a PRF for an arbitrarily shaped DUH without the need to simulate the entire hydrograph, as long as the peak discharge has been captured. A MATLAB[®] script calculates the base of the equivalent triangle.

The product of the non-dimensional factor, K, and the conversion constant 645.33 is the PRF. The PRF is indicative of the degree of peakedness of the unit hydrograph. For the classic NRCS unit hydrograph PRF is equal to 484, therefore Equation 6.1 can be

rewritten as the widely-used equation to calculate q_p for the UH:

$$q_p = \frac{484AQ}{t_p} \quad (6.6)$$

The same procedure can be applied in different experiments to obtain PRFs using different watershed parameters in GSSHA model.

6.4.6 Experiments

A variety of experiments are conducted to assess the effect of different watershed parameters on unit hydrograph characteristics. Specifically, the impacts of these experiments on the time to peak, peak discharge, the shape of the unit hydrograph, and PRF are assessed.

GSSHA simulations are performed with simplified watershed models, keeping the watershed aspect ratios constant, and varying the following parameters:

- Surface Roughness
- Side Slope
- Longitudinal Slope
- Precipitation Intensity and Duration

for each parameter, PRF is calculated, and its dependence on these parameters is analyzed. Each variable is plotted against the resulting PRFs to determine whether any trends can be observed.

In the initial phase of the study, the grid spacing and computational time step were carefully assessed to ensure stability and proper convergence of the numerical solutions for the chosen parameters. Throughout this chapter, all variables, except for the one being investigated, remained constant to isolate the effects of the adjusted variable per experiment. The default settings in the model are as follows: rainfall intensity is set at 10 mm/hr and rainfall duration at 60 minutes for all experiments, except where these parameters themselves are under investigation. Similarly, Manning's roughness coefficient is consistently held at 0.22, except in experiments specifically altering this parameter. The side slope is maintained at 0.1 (10%) and the longitudinal slope is kept at 0.025 (2.5%) except in their respective experimental evaluations. The output frequency of the model is fixed at 5 minutes with a total simulation duration of 28,800 minutes, equivalent to 20 days. The computational time step is defined at 1 second, and the spatial grid size is 100 meters. This ensures consistency in the outputs. The aspect ratio of the open-book watershed model remained constant throughout all experiments.

6.5 GSSHA Model Stability

To ensure the stability of the GSSHA model, two distinct grid sizes were evaluated to estimate the unit hydrograph and peak discharge resulting from 1 cm of rainfall excess. As illustrated in Figure 6.6, the model demonstrated consistent and stable results, with minimal difference between the hydrographs simulated with the 50m and 100m grid sizes. Within the model in Figure 6.6, the total simulation duration was set to 28,800 minutes (equivalent to 20 days), with a timestep of 1 second. The Manning's Roughness coefficient

was established at 0.1, and the frequency for generating hydrographs by the model was set to every 60 minutes.

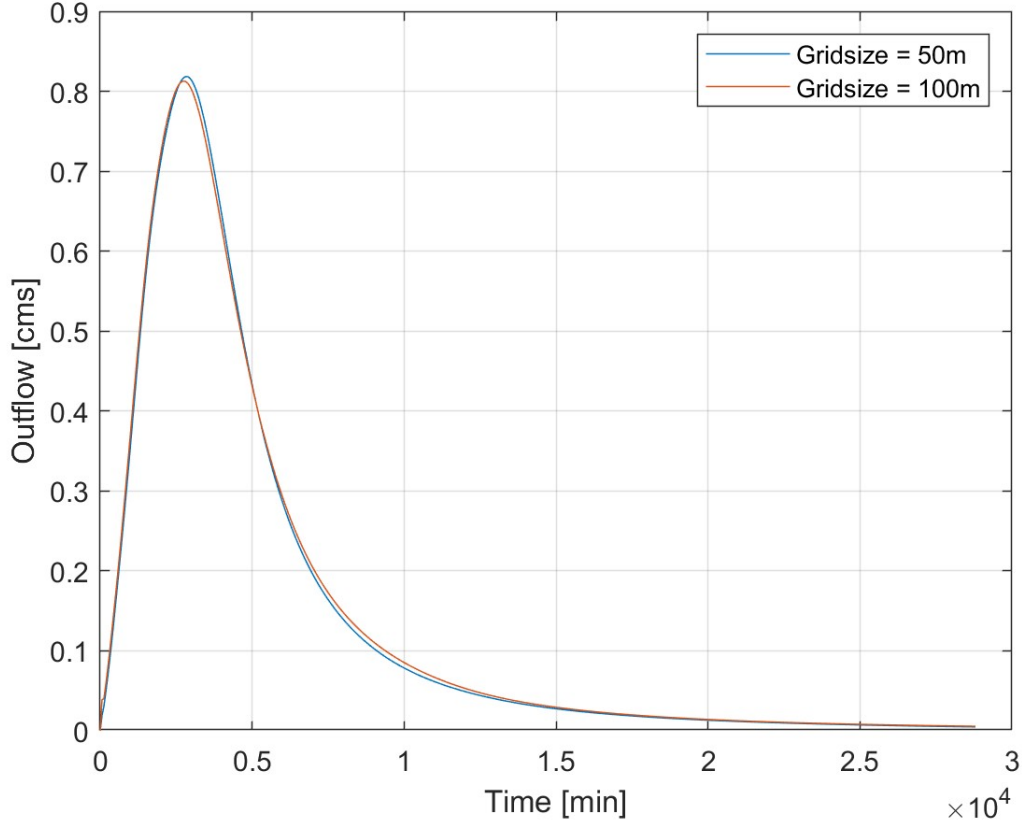


Figure 6.6: Model Stability Experiment

Within the model in Figure 6.7, the total simulation duration was set to 28,800 minutes (equivalent to 20 days), with a timestep of 1 second. The Manning's roughness coefficient was established at 0.01, and the frequency for generating hydrographs by the model was set to every 60 minutes (1 hour).

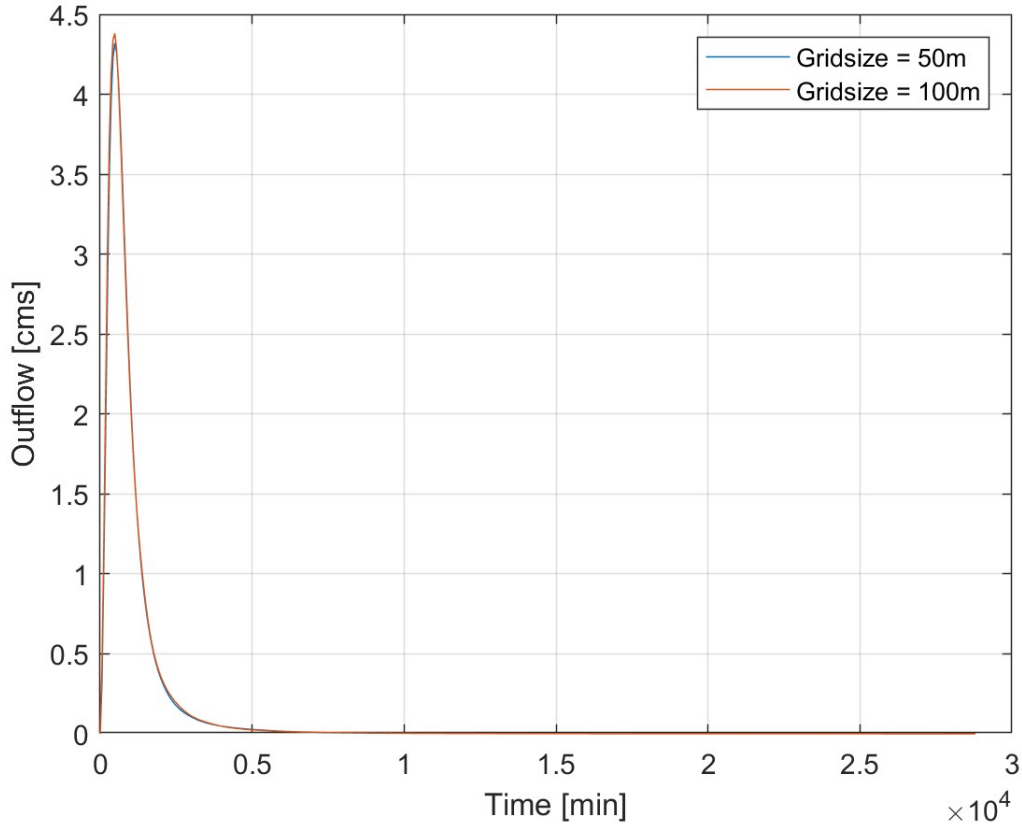


Figure 6.7: Model Stability Experiment

Similar to the Figure 6.6, the adjustment of the Manning's roughness does not change the model stability. Therefore, based on the results obtained, the 100m grid size was selected for conducting various experiments in the subsequent sections. To ensure the stability and accuracy of the GSSHA model with a grid size of 100 meters, an analysis of the Courant-Friedrichs-Lewy (CFL) condition was conducted. The CFL condition is a critical factor in determining the stability of explicit numerical schemes, such as those employed in the GSSHA model. The CFL condition relates the time step and grid size, ensuring that the numerical solution remains stable and convergent. In the subsequent

section, the methodology and results of the CFL condition analysis will be discussed in detail, verifying the suitability of the chosen grid size for the experimental setup.

6.6 Courant-Friedrichs-Lewy Condition Analysis for GSSHA Stability

6.6.1 Introduction

This section presents a detailed analysis of the Courant-Friedrichs-Lewy (CFL) condition with explicit algebraic calculations. The CFL condition is crucial in numerical simulations for ensuring stability and is used in this context to evaluate GSSHA stability for the given timestep and gridsize for running experiments. The Courant number, C , is calculated using the formula $C = \frac{u\Delta t}{\Delta x}$, where u is the average flow velocity in [m/s], Δt is the time step in [sec], and Δx is the spatial grid size in [m]. For the CFL condition to be satisfied $CFL < 1$,

6.6.2 Timestep

In the GSSHA model, the time step is set to 1 second, thus the time increment, Δt , in the Courant-Friedrichs-Lewy (CFL) condition is equal to 1.

6.6.3 Average Speed Calculation (u)

The average speed is calculated as the speed that the water is moving. This can be tested for extreme scenarios using Manning's Equation as follows:

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \quad (6.7)$$

where V is the flow velocity measured in [m/s], n is Manning's roughness coefficient, which is dimensionless [-]¹, R is the hydraulic radius measured in [m], defined as the area of the water cross-section, A , divided by the wetted perimeter, P , ($R = A/P$), and S is the hydraulic slope, typically approximated by the slope of the channel bed (longitudinal slope) and is dimensionless [-].

6.6.4 Courant Number Calculation (C)

In the conducted experiments, the highest longitudinal slope is 0.4 ($S_0 = 0.4$), and the lowest Manning's roughness coefficient (n) used is 0.02. The interpretation of this condition is that the distance physically traveled by water in a time step ($u\Delta t$) is less than the spatial separation used in the numerical solution. In other words, the physical water flow is not outpacing the numerical calculation of flow. Therefore, with a time step (Δt) of 1 second and a spatial grid size (Δx) of 100 meters, the average flow velocity (u) must not exceed 100 m/s to satisfy $CFL < 1$. To be able to calculate the average velocity using Manning's equation, the hydraulic radius should be estimated. Since no channel is created for the open-book watershed and all the flow is considered overland/sheet flow, the hydraulic radius (R_h) is typically assumed to equal the water depth (y) in wide rectangular assumption. Furthermore, the velocity should be estimated for overland/sheet flow using the maximum slope (side slope in the experiments).

To estimate the CFL, a reasonable range should be selected for y which is the depth of the overland/sheet flow. Therefore, hydraulic radius (R_h), average velocity and Courant

¹ n can be treated as unitless if units are assigned to the constants in the USCS and SI forms of the Manning's equation. **USCS:** $V \left[\frac{ft}{s} \right] = \frac{1.486}{n} ft^{1/3} / s (R_h[ft])^{2/3} S_0^{1/2}$, **SI:** $V \left[\frac{m}{s} \right] = \frac{1}{n} m^{1/3} / s (R_h[m])^{2/3} S_0^{1/2}$

Number can be estimated. For y , a value of 10 cm (0.1 m) is selected. Given, $y = 0.1$ [m], $n = 0.38$ [-], $S_0 = 0.2$ [-], average velocity (V) is calculated as:

$$V = \frac{1}{n} R_h^{\frac{2}{3}} S_0^{\frac{1}{2}} = \frac{1}{0.38} (0.1)^{2/3} (0.2)^{1/2} = 0.25 [m/s] \quad (6.8)$$

and accordingly, Courant number is:

$$C = \frac{u \Delta t}{\Delta x} = \frac{0.25 [m/s] \times 1 [sec]}{100 [m]} = 0.0025 [-] \quad (6.9)$$

since the condition $CFL < 1$ is met, this confirms that the numerical model with a time step of 1 second and a spatial grid size of 100 meters is stable.

6.7 Experiments for Surface Roughness

This section presents a series of experiments designed to assess the effect of surface roughness, n , on the peak discharge and the PRF. Literature suggests that a reasonable range for surface roughness lies between 0.01 and 0.40 and the surface roughness for non-channel areas would generally be larger than 0.1. A total of 15 experiments were carried out in this section, with calculations were made for the entire V-shaped watershed at the model's 1-second time step. The outflow hydrograph was reported at 5-minute intervals. Peak discharge was determined from the outflow hydrograph. For these experiments, surface roughness ranges from 0.1 to 0.38. The event hydrographs are shown in Figure 6.8. The maximum point on each curve is the peak discharge. The peak discharge is highest at $n = 0.1$, approximately 0.27 cms. This experiment also exhibits the shortest time to peak

(t_p). With increasing values of surface roughness, there is a noticeable decrease in peak discharge and a corresponding increase in t_p , which results in more flattened hydrographs. These observations clearly demonstrate that higher surface roughness is directly associated with lower peak discharge and prolonged time to peak within the watersheds.

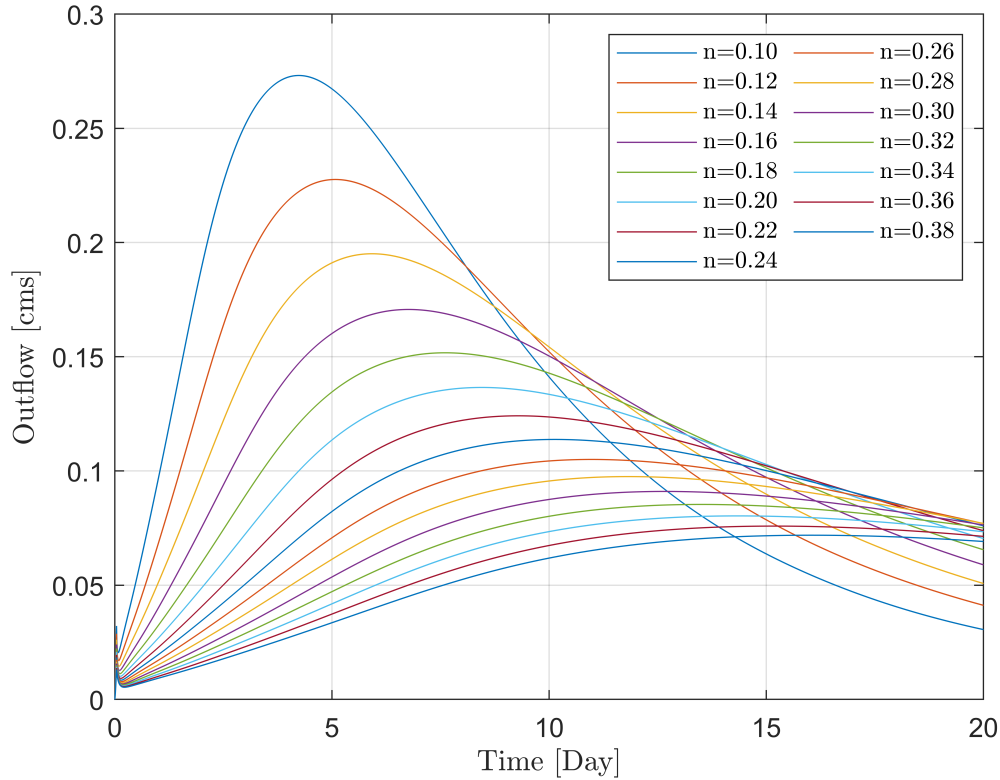


Figure 6.8: Unit Hydrograph Experiments with Varying Surface Roughness

Figure 6.9 depicts the relationship between peak discharge and Manning's roughness coefficient (n). As the roughness coefficient increases, the peak discharge decreases. This trend is consistent with the hydraulic principle that a greater surface roughness reduces the velocity of water flow, leading to a lower discharge rate. The result also displays a rapid initial decrease in discharge as roughness increases from low values, which then tapers off,

suggesting a diminishing effect of roughness on discharge at higher values of the coefficient.

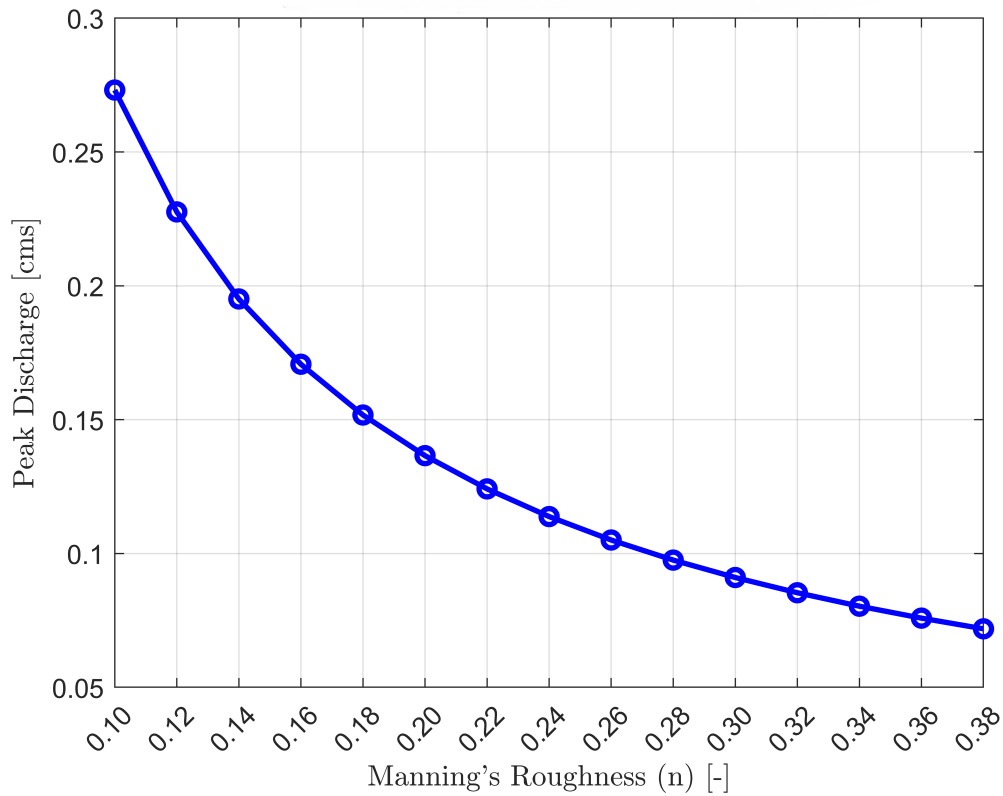


Figure 6.9: Peak Discharge vs. Manning's Roughness

Figure 6.10 presents a positive correlation between the time to peak (t_p) and Manning's roughness coefficient (n). As Manning's roughness increases, the time to peak also increases. This suggests that rougher channel surfaces slow the flow of water, thus delaying the time at which the peak discharge is reached. The linear relationship indicated by the straight line through the data points signifies that this delay in peak discharge is proportional to the increase in roughness. This behavior aligns with hydrologic understanding that increased surface roughness impedes water flow velocity, resulting in a more gradual rise to peak discharge levels.

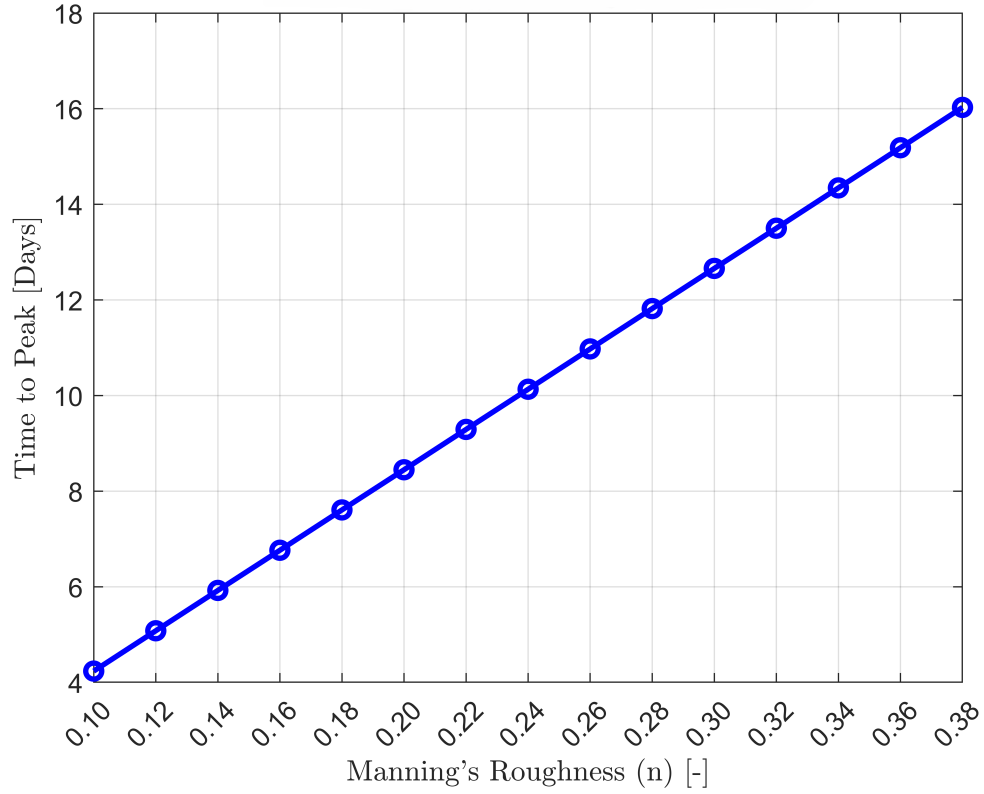


Figure 6.10: Time to Peak vs. Manning's Roughness

In the next step, the association between Manning's roughness and PRF is established. Since both peak discharge (q_p) and time to peak (t_p) are identified in generated UHs from the distributed model, the PRFs can be estimated by inverting the NRCS peak discharge Equation 4.1 as follows:

$$PRF = \frac{q_p t_p}{AQ} \quad (6.10)$$

where q_p is the peak discharge in [ft^3/s], t_p is the time to peak in [hours], A is the watershed area in [mi^2], and Q is the direct runoff depth (or precipitation excess depth) in [inches].

Since the time to peak is in days and the peak discharge generated by the distributed model is in $[m^3]$, all units are converted to appropriately compute PRF in U.S. Customary System units using Equation 6.10.

Figure 6.11 illustrates the relationship between the PRF and surface roughness (n). As the surface roughness increases from 0.10 to 0.38, the PRF gradually decreases, demonstrating a negative correlation between the two variables. The PRF starts at approximately 248 for the lowest surface roughness value of 0.10. As the surface roughness increases, the PRF gradually declines, reaching about 246.9 at a surface roughness of 0.38. This smooth, downward trend indicates that higher surface roughness values result in lower PRF values. The curve stabilizes towards the higher end of the surface roughness range, showing more gradual changes in PRF. Notably, the change in PRF based on the surface roughness range is not significant, as the PRF drops from 248 to 246.95, which is a reduction of only about 1 unit. Overall, the plot demonstrates that increased surface roughness reduces the PRF, although the overall change is quite small. This finding is significant for understanding the impact of surface characteristics on hydrologic modeling and runoff estimations.

Table 6.1 provides a overview of the experimental outcome with the impact of variable of interest, Manning's roughness coefficient, on time to peak, peak discharge, and the PRF.

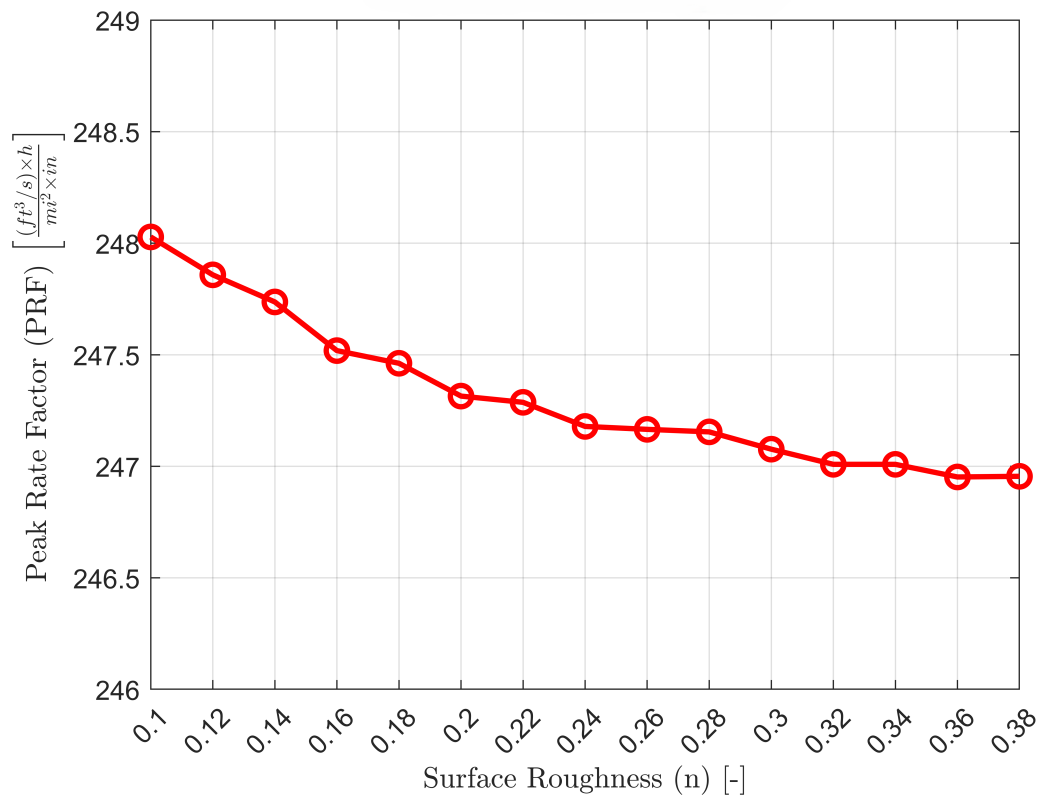


Figure 6.11: Peak Rate Factor vs. Surface Roughness

Table 6.1: Surface Roughness, Time to Peak, Peak Discharge, and PRF Values

n	Time to Peak [days]	Peak Discharge [cms]	PRF $\left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right]$
0.10	4.24	0.27	248.0
0.12	5.08	0.23	247.8
0.14	5.92	0.20	247.7
0.16	6.76	0.17	247.5
0.18	7.61	0.15	247.4
0.20	8.45	0.14	247.3
0.22	9.29	0.12	247.2
0.24	10.13	0.11	247.1
0.26	10.98	0.11	247.1
0.28	11.82	0.10	247.1
0.30	12.66	0.09	247.0
0.32	13.50	0.09	247.0
0.34	14.34	0.08	247.0
0.36	15.18	0.08	246.9
0.38	16.03	0.07	246.9

6.8 Experiments for Side Slope

This section presents another set of experiments designed to evaluate the impact of side slope on time to peak, the peak discharge and the PRF in simplified open-book watersheds. A reasonable range of side slope values, from 4% representing less steep side slopes to 18% representing very steep side slopes, was selected for investigation. These values were chosen to cover a wide spectrum of realistic side slope scenarios in the open-book watershed.

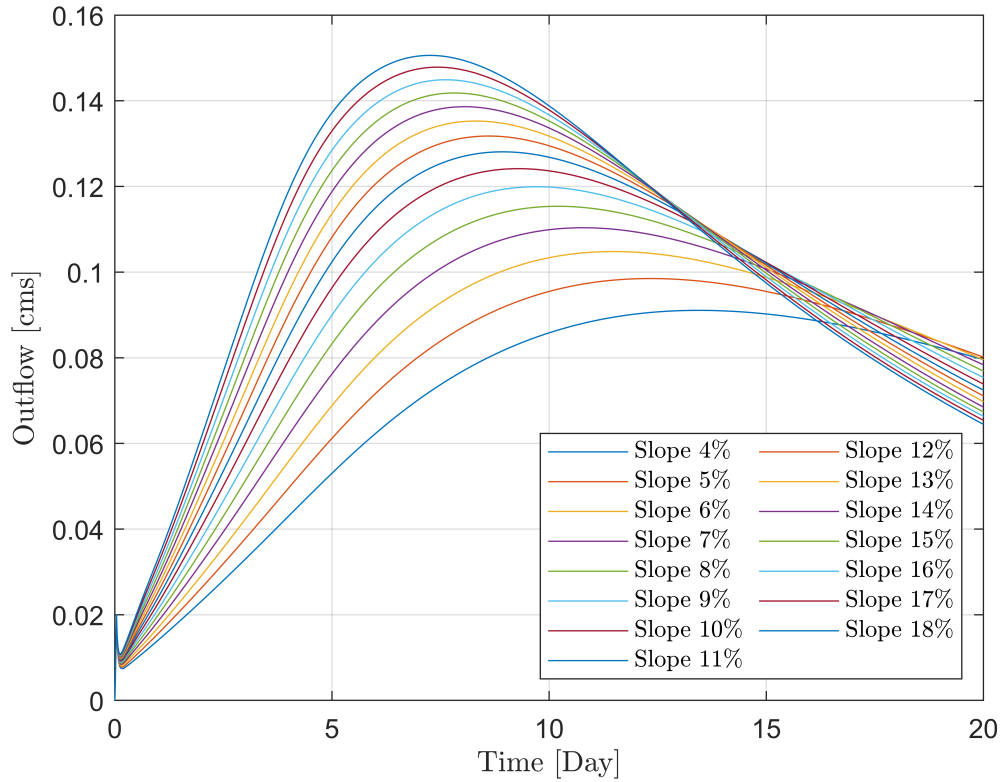


Figure 6.12: Side Slope Experiments for Simplified Watershed

Similar to other experiments, all other variables remained constant throughout this series of simulations. Fifteen experiments were conducted to investigate the impact of side slope on the UH characteristics. The side slope values were varied within a selected range, while keeping other parameters unchanged. Figure 6.12 illustrates the results of these experiments. The highest peak discharge, approximately 0.15 cms, was observed for the experiment with a side slope of 18%. This experiment also exhibited the shortest t_p . As the side slope values decreased, a noticeable trend emerged: the peak discharge decreased, accompanied by a corresponding increase in t_p .

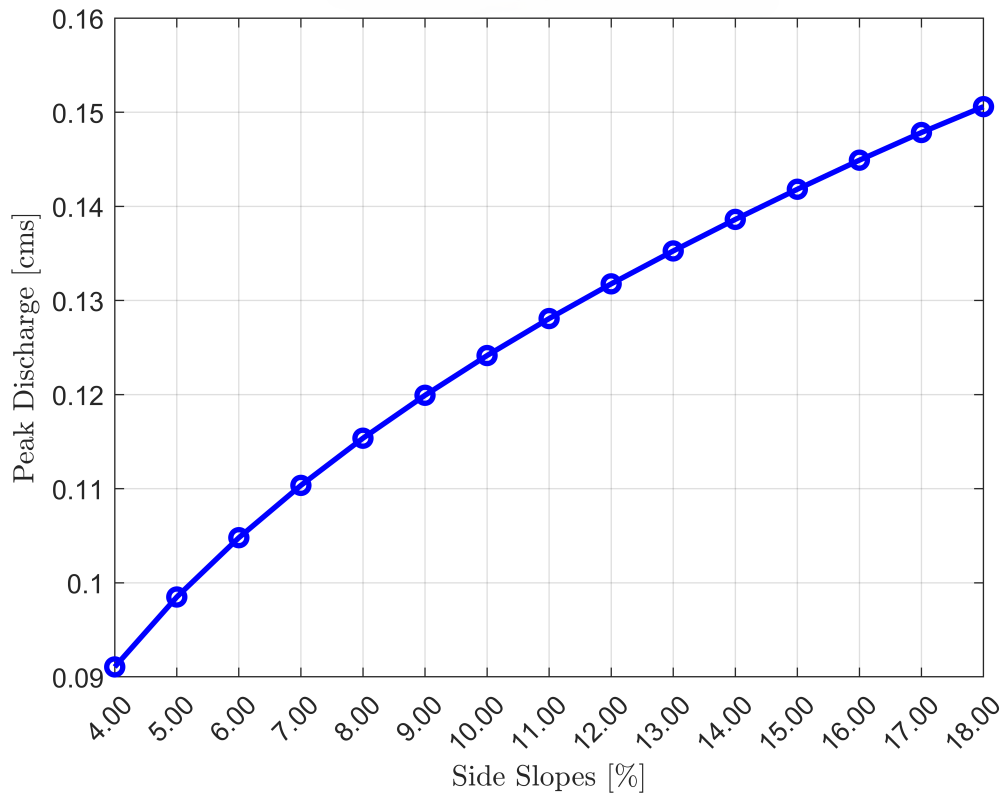


Figure 6.13: Peak Discharge vs. Side Slope

This trend resulted in more peaked hydrographs for higher side slopes. Interestingly, the rate of change in peak discharge and t_p with respect to side slope variations was smaller compared to the experiments focusing on surface roughness. These observations clearly demonstrate that within the context of the simplified watersheds used in this study, higher side slopes are directly associated with higher peak discharges and shorter times to peak.

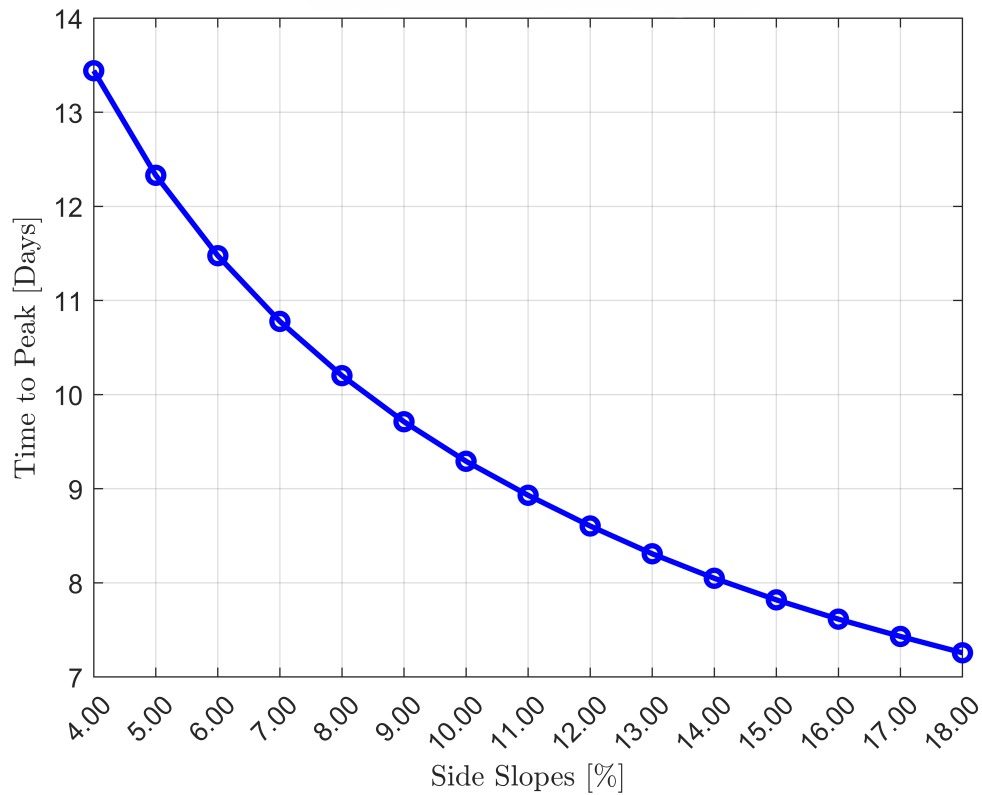


Figure 6.14: Time to Peak vs. Side Slope

Figure 6.13 illustrates a clear positive correlation between the peak discharge of the unit hydrograph and the side slope of the open-book watershed. As the side slope becomes larger, the peak discharge increases significantly. This trend suggests that flatter side slopes attenuate the peak flow, due to slower runoff velocity as no infiltration or storage was modeled in this simulation. The curve exhibits a non-linear relationship, with the rate of increase in peak discharge becoming less pronounced as the side slope increases. This may indicate that the influence of side slope on peak discharge is reduced at higher gradients. The steepest increase in peak discharge occurs between side slopes of 4% and 5%, implying that this range of slopes has the greatest impact on moderating peak flows

in the simulation. This implies the higher sensitivity of the peak discharge to changes in side slope at flatter side slopes.

Figure 6.14 shows a clear negative correlation between the time to peak discharge of the UH and the side slope of the simplified watershed. The time to peak discharge decreases as the side slope becomes steeper. This trend suggests that steeper side slopes intensifies the concentration of runoff, thereby diminishing the time it takes for the peak flow to occur. The curve also indicates a non-linear relationship, with the rate of decrease in time to peak diminishing as the side slope gets larger. This implies that the time to peak discharge becomes less sensitive to changes in side slope at larger gradients. Further analysis and simulations would be needed to quantify the precise mathematical relationship and determine if there is any physical thresholds.

Figure 6.15 displays the relationship between side slopes and the estimated PRFs computed from Equation 6.10. It illustrates a clear negative correlation between the side slope and the PRF, indicating that as the side slope increases, the PRF decreases non-linearly. This trend suggests that steeper side slopes result in a lower PRF, which quantifies fraction of runoff volume that is delivered in the rising limb of the hydrograph. The PRF relates the maximum rate of discharge with watershed area, depth of direct runoff (1 unit for UH), and time to peak. The negative relationship can be attributed to hydrological dynamics where steeper slopes may accelerate surface runoff, thereby reducing the duration of peak discharge. This association is crucial in understanding watershed management as it impacts flood estimation and the design of hydraulic structures by highlighting how varying terrain slopes influence runoff characteristics.

Table 6.2 provides a overview of the experimental outcome with the impact of variable

of interest, side slope, on time to peak, peak discharge, and the PRF.

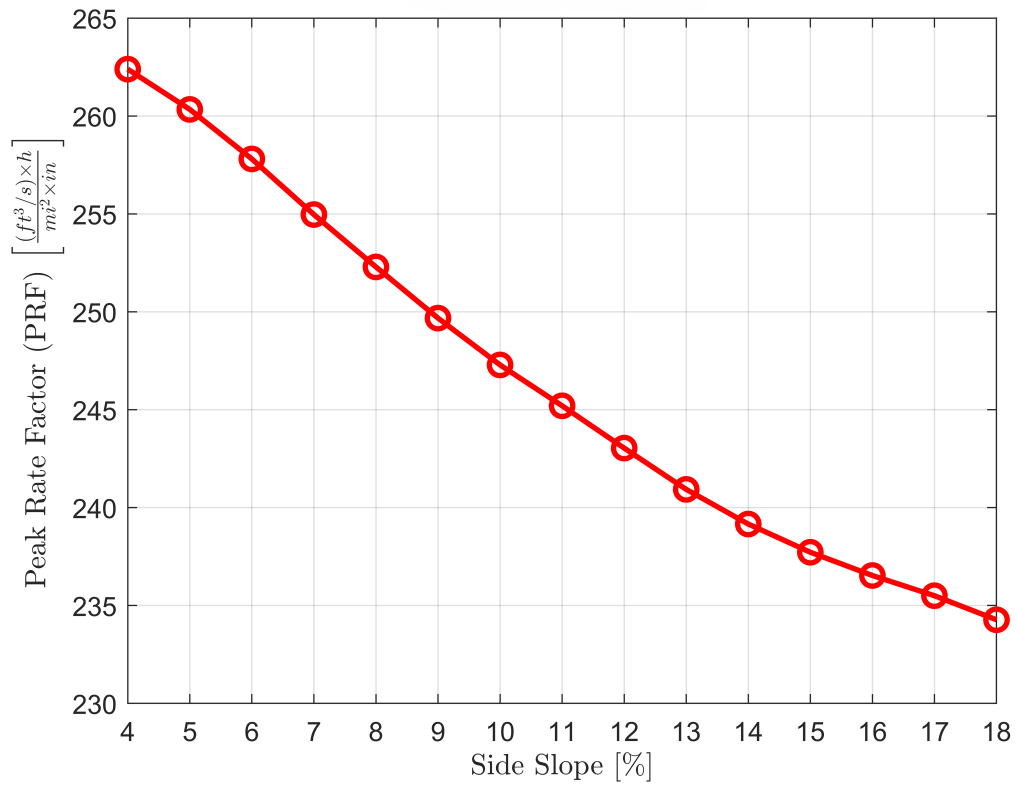


Figure 6.15: Peak Rate Factor vs. Side Slope

Table 6.2: Side Slope, Time to Peak, Peak Discharge, and PRF Values

Side Slope [%]	Time to Peak [days]	Peak Discharge [cms]	PRF $\left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right]$
4	13.440	0.091	262.39
5	12.329	0.098	260.34
6	11.475	0.105	257.81
7	10.777	0.110	254.97
8	10.201	0.115	252.29
9	9.711	0.120	249.68
10	9.292	0.124	247.29
11	8.930	0.128	245.20
12	8.604	0.132	243.04
13	8.309	0.135	240.93
14	8.049	0.139	239.16
15	7.819	0.142	237.72
16	7.615	0.145	236.53
17	7.431	0.148	235.50
18	7.257	0.151	234.27

6.9 Experiments for Longitudinal Slope

This section describes a series of experiments designed to assess the influence of longitudinal slope on time to peak, peak discharge and the PRF in the open-book watershed. A spectrum of longitudinal slopes was selected, spanning from 0.01 (1%), indicative of gentle slopes, to 0.038 (3.8%), indicative of steep slopes. This selection was made to include a broad array of realistic longitudinal slope conditions within the open-book watershed. The range of slope values was determined to ensure a comprehensive evaluation across different scenarios in this experiment.

Figure 6.16 represent a series of unit hydrograph experiments that show how the outflow from the open-book watershed changes over time for different longitudinal slopes. As the slope increases, the peak outflow also increases and occurs slightly earlier, signifying

a faster response to rainfall. This suggests that steeper longitudinal slopes lead to quicker and more intense runoff. Conversely, flatter slopes (e.g., Slope 0.01) show slightly delayed and more prolonged outflow. After the peak, all the hydrographs demonstrate a gradual recession, with steeper slopes resulting in slightly quicker recession.

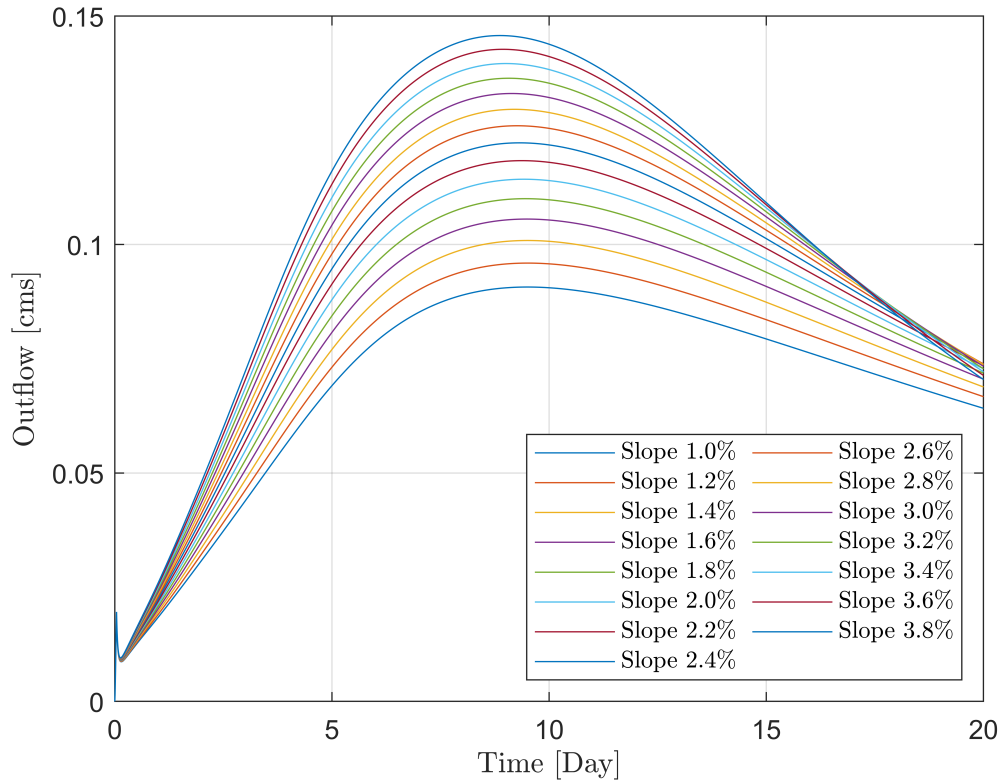


Figure 6.16: Longitudinal Slope Experiments for Simplified Watershed

Figure 6.17 illustrates the relationship between the longitudinal slope of the open-book watershed and the peak discharge. The graph indicates a clear positive correlation: as the longitudinal slope percentage increases, the peak discharge also increases. Furthermore, the relationship between longitudinal slopes and peak discharges exhibits a non-linear trend. However, the degree of non-linearity in this relationship is less pronounced than that

observed between side slopes and peak discharges. This distinction suggests varying hydrodynamic responses to changes in slope geometry, warranting further investigation to understand the underlying mechanisms and their implications on hydrological modeling.

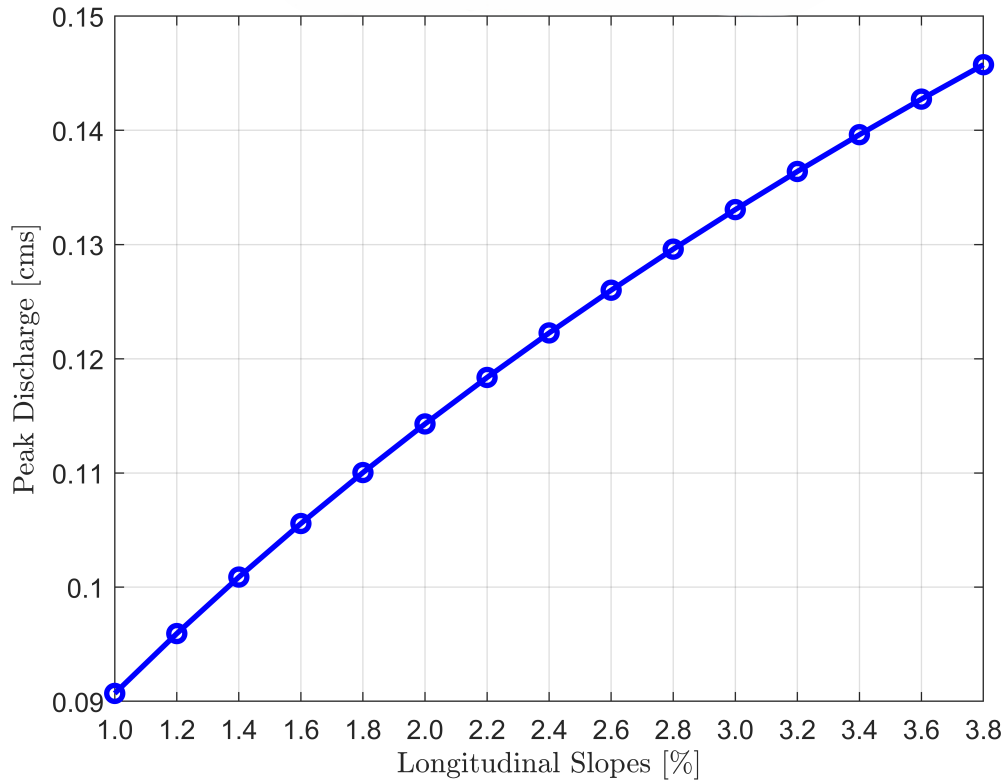


Figure 6.17: Peak Discharge vs. Longitudinal Slope

Figure 6.18 illustrates the relationship between longitudinal slope and time to peak in an open-book watershed. The data shows that for lower longitudinal slopes, the time to peak changes minimally, suggesting a low sensitivity to changes in these smaller slope values. As the slope increases, the time to peak decreases more markedly, indicating a heightened sensitivity at higher slope values. This non-linear relationship underscores a stronger hydrological response as the slope becomes steeper, reflecting more pronounced

hydrodynamic effects at higher gradients. Further research is needed to fully understand the dynamics of this relationship and its implications for watershed management.

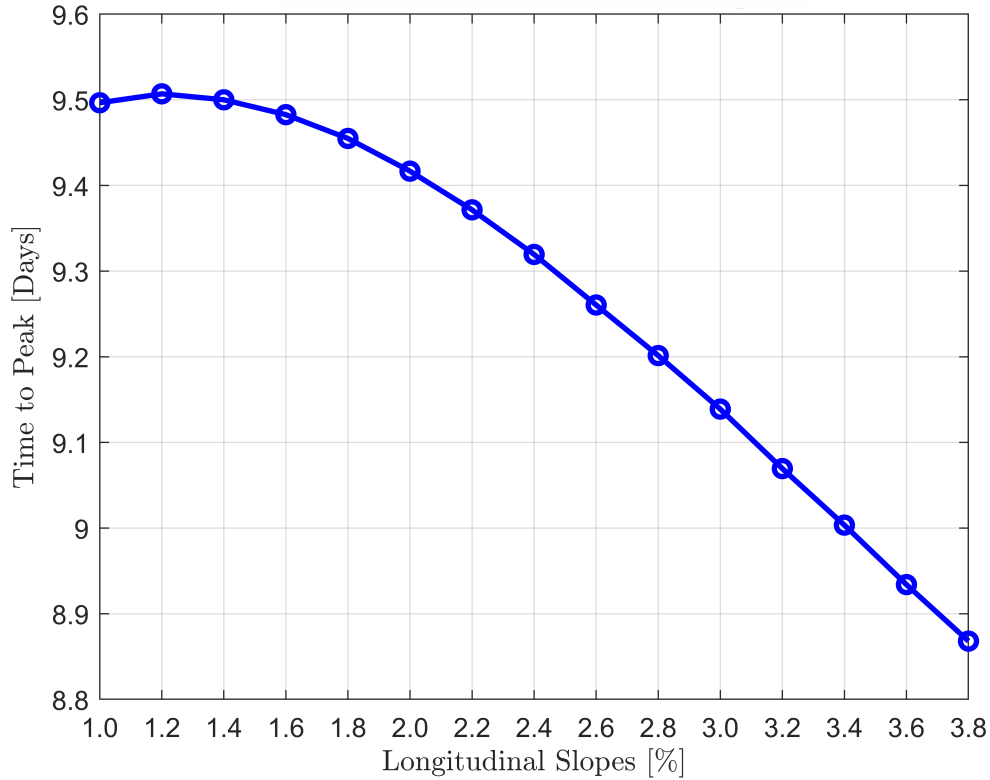


Figure 6.18: Time to Peak vs. Longitudinal Slope

Figure 6.19 displays the relationship between longitudinal slopes and the estimated PRFs. It shows a clear non-linear relationship between the longitudinal slope and the PRF. As the longitudinal slope increases from 1.0% to 3.8%, the PRF also exhibits a consistent upward trend from approximately 185 to 280. This indicates that the PRF increases with the steepness of the longitudinal slope. The positive correlation suggests that as the slope becomes steeper, the runoff generated by rainfall is likely to be more rapid, thus increasing the PRF. The non-linear pattern in the data points, supports a strong proportional non-

linear relationship between longitudinal slope and PRF. This relationship is important for hydrologic analysis and flood modeling.

Figure 6.15 illustrates a negative correlation between side slope and PRF, suggesting that as the side slope becomes steeper, the peak rate of flow decreases. Conversely, Figure 6.19 shows a positive correlation between longitudinal slope and PRF, indicating that steeper longitudinal slopes lead to higher peak rates of flow.

Table 6.3 provides a overview of the experimental outcome with the impact of variable of interest, longitudinal slope, on time to peak, peak discharge, and the PRF.

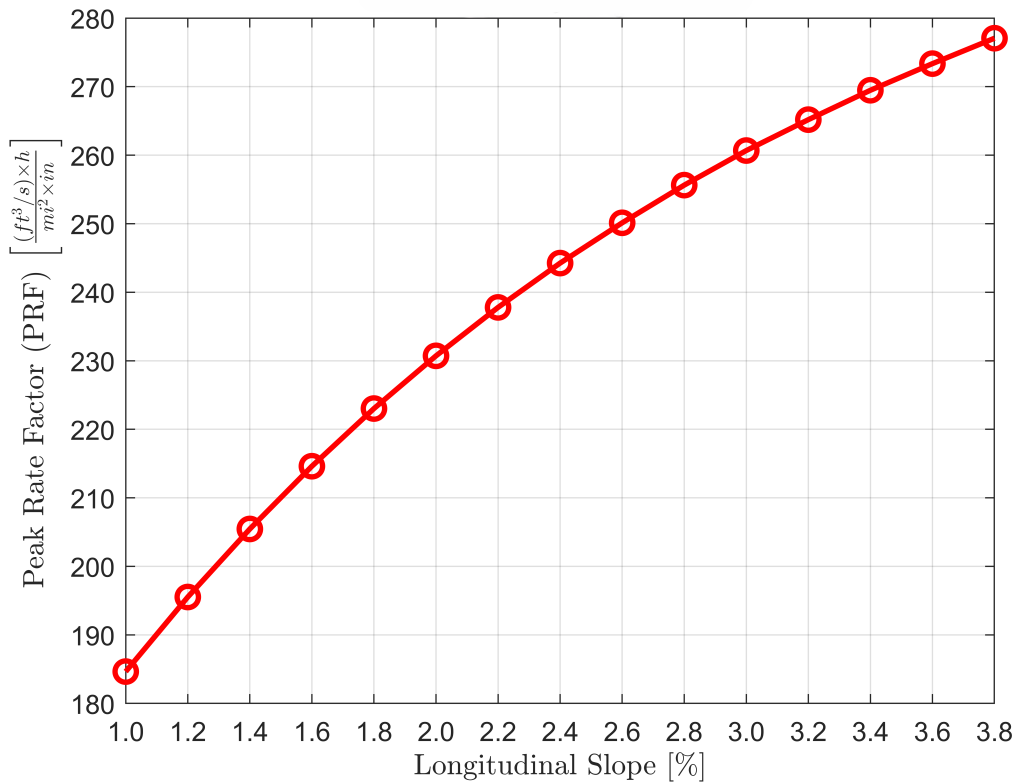


Figure 6.19: Peak Rate Factor vs. Longitudinal Slope

Table 6.3: Longitudinal Slope, Time to Peak, Peak Discharge, and PRF Values

Longitudinal Slope [%]	Time to Peak [days]	Peak Discharge [cms]	PRF $\left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right]$
1.0	9.500	0.091	184.64
1.2	9.507	0.096	195.52
1.4	9.500	0.101	205.45
1.6	9.483	0.106	214.61
1.8	9.455	0.110	223.03
2.0	9.417	0.114	230.72
2.2	9.372	0.118	237.79
2.4	9.319	0.122	244.25
2.6	9.260	0.126	250.13
2.8	9.201	0.130	255.63
3.0	9.139	0.133	260.68
3.2	9.069	0.136	265.18
3.4	9.004	0.140	269.47
3.6	8.934	0.143	273.36
3.8	8.868	0.146	277.07

6.10 Experiments for Rainfall Intensity and Duration

This section details experiments designed to evaluate the influence of varying rainfall intensities and durations on peak discharge and the PRF in a simulated open-book watershed. The experiments were structured around a set of intensity and duration combinations that produced a total rainfall volume of 10 mm (1 cm), equivalent to a unit hydrograph. The experimental parameters ranged from a low intensity of 0.41667 mm/hr with a long duration of 1440 minutes (24 hours) to a high intensity of 120 mm/hr for a small duration of 5 minutes. These values were selected to consistently result in 10 mm (1 cm) of runoff, with all variations falling within this specified range to ensure the unit hydrograph volume (i.e., depth). The selection was made to ensure an evaluation across a variety of scenarios. In this experiment, a rainfall index from 1 to 15 is used for every combination of rainfall

intensity and duration. The intensity and duration associated with each rainfall index are listed in Table 6.4. Index 1 corresponds to the lowest intensity and longest duration, while Index 15 corresponds to the highest intensity and shortest duration. Figure 6.20 illustrates the 15 experiments conducted to assess the effects of varied rainfall intensities and durations on the resulting unit hydrograph, time to peak, peak discharge, and PRF as simulated by the GSSHA distributed hydrologic model.

Table 6.4: Rainfall Index, Rate, and Duration

Rainfall Index	Intensity [mm/hour]	Duration [minutes]
1	0.41667	1440
2	1	600
3	2	300
4	4	150
5	5	120
6	6	100
7	8	75
8	10	60
9	20	30
10	30	20
11	40	15
12	50	12
13	60	10
14	75	8
15	120	5

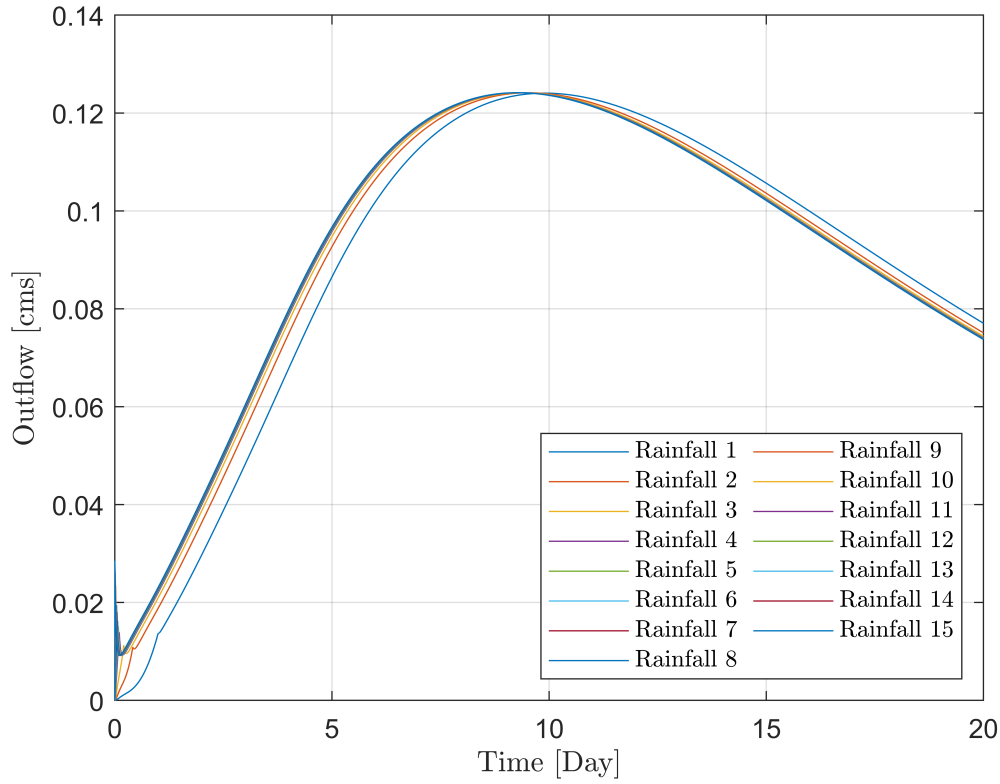


Figure 6.20: Rainfall Intensity & Duration Experiments for Simplified Watershed

Figure 6.20 depicts that variations in rainfall durations and intensities do not significantly affect the peak discharge and time to peak when employing the GSSHA distributed model in the open-book watershed. The simulation results indicate that, provided the depth of uniform rainfall remains at one unit (e.g., 1 cm or 1 inch), the shape of the resulting unit hydrograph produced by the GSSHA distributed model, remains largely unchanged, despite considerable variations in both intensities and durations across all simulations within this section.

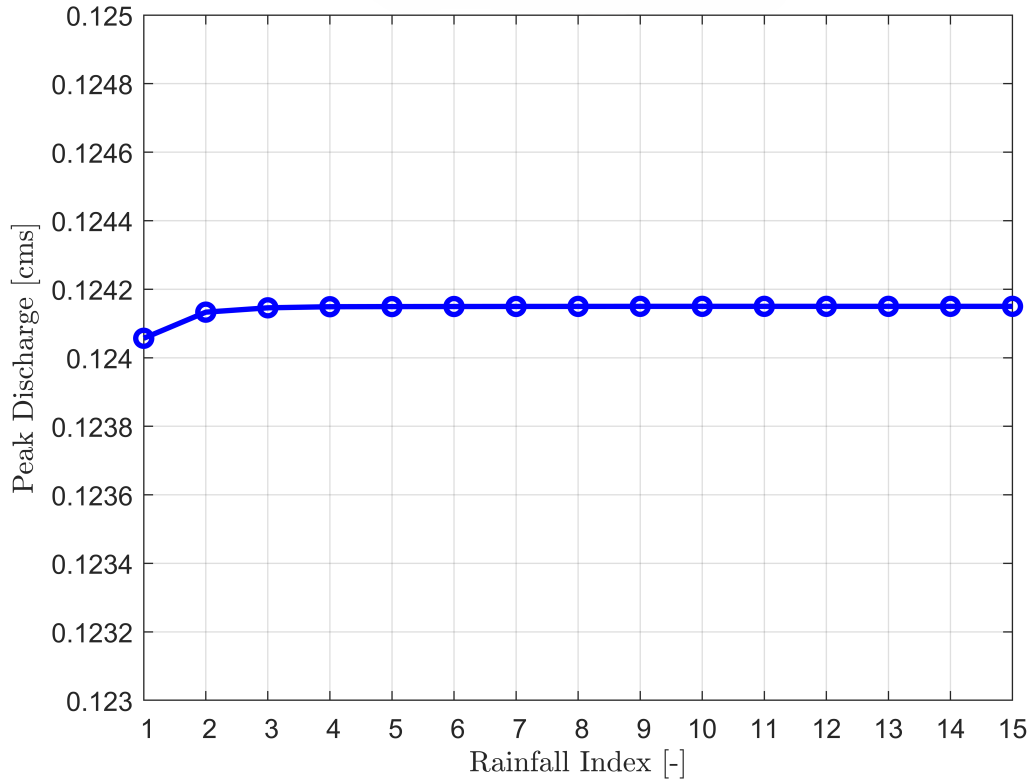


Figure 6.21: Peak Discharge vs. Rainfall Index

Figure 6.21 illustrates the relationship between rainfall index and peak discharge as simulated by the GSSHA distributed model for the open-book watershed. The peak discharge values increase only slightly after the first index, without showing substantial rise, indicating that beyond a certain threshold, changes in rainfall intensity do not have a significant impact on the peak discharge. Specifically, beyond index 2, the peak discharge stabilizes around 0.1242 cms. This stabilization persists across the varied intensities and duration, showing little to no fluctuation even as the intensity increases to 120 mm/hr for a duration of 5 minutes. It can be concluded that the GSSHA model predicts a threshold effect, in which intensity variations above a certain minimum do not significantly alter

the peak discharge in this specific watershed, suggesting that the watershed's response to rainfall is highly inelastic in terms of peak discharge once it reaches saturation. In Figure 6.21, the x-axis representing rainfall index, implicitly incorporates intensity and duration. This is because, while intensity ranges from 0.41667 mm/hr to 120 mm/hr, the corresponding duration inversely varies from 1440 minutes to 5 minutes. This inversely proportional relationship ensures that each simulation maintains a constant rainfall depth of 1 cm (10 mm) across the entire watershed. Thus, the x-axis effectively conveys a composite metric of both intensity and duration, which collectively influence the peak discharge outcomes.

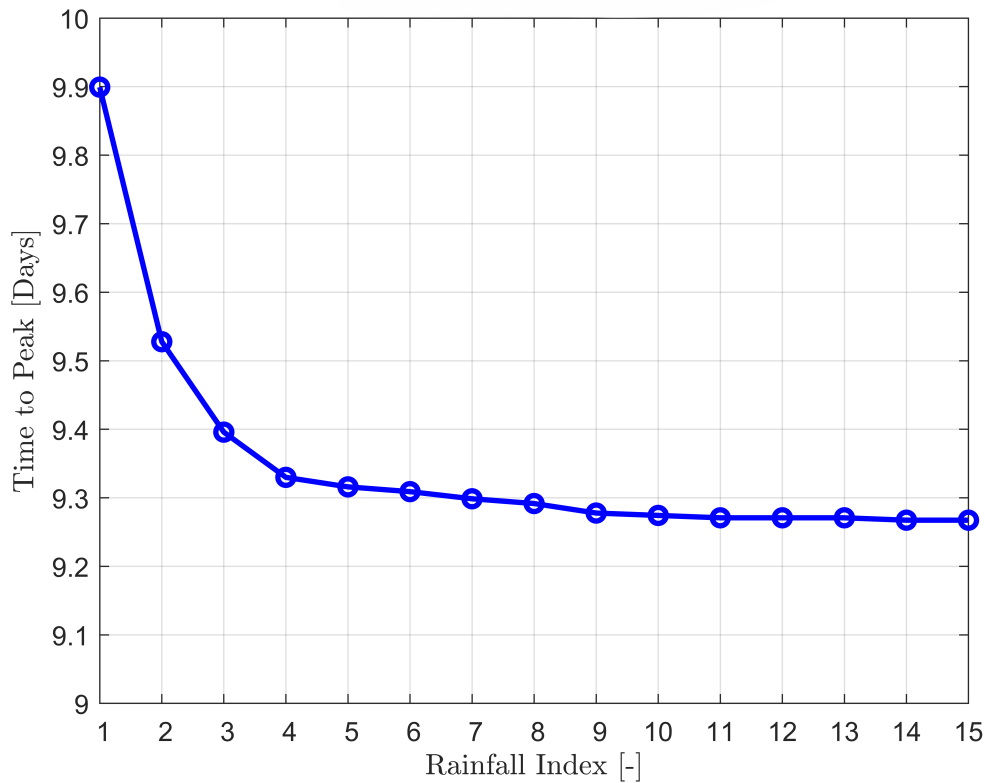


Figure 6.22: Time to Peak vs. Rainfall Index

Figure 6.22 illustrates the relationship between rainfall index and time to peak in the

open-book watershed. The abscissa shows rainfall index, and the ordinate shows time to peak in days. As intensity increases from 0.41667 mm/hr (index 1) to 4 mm/hr (index 4), the time to peak sharply decreases from around 9.9 days to 9.3 days. After this point, increasing the intensity for a shorter duration does not significantly reduce the time to peak, which stabilizes at around 9.3 days. This pattern suggests that once rainfall intensity exceeds 4 mm/hr, the watershed's response in terms of the timing of peak discharge does not change, indicating a limit to how rainfall intensity affects the rate of the open-book watershed's response.

Figure 6.23 illustrates the correlation between the rainfall index and the estimated PRFs calculated using the NRCS equation. Notably, the PRF decreases from approximately 263 to 248 from index 1 to 4. From index 5 onward, there is a marginal but consistent decline in PRF values as the rainfall index increases. This pattern is also observed in the t_p plot (Figure 6.22). Despite these changes, the overall variation in the estimated PRFs remains relatively insignificant after the initial few experiments. This observation indicates that, with a consistent rainfall depth of 1 cm, variations in intensity beyond a certain threshold exert a minimal influence on PRF.

Table 6.5 provides a summary of the experimental outcome with the impact of variable of interest (rainfall intensity and duration) on time to peak, peak discharge, and the PRF.

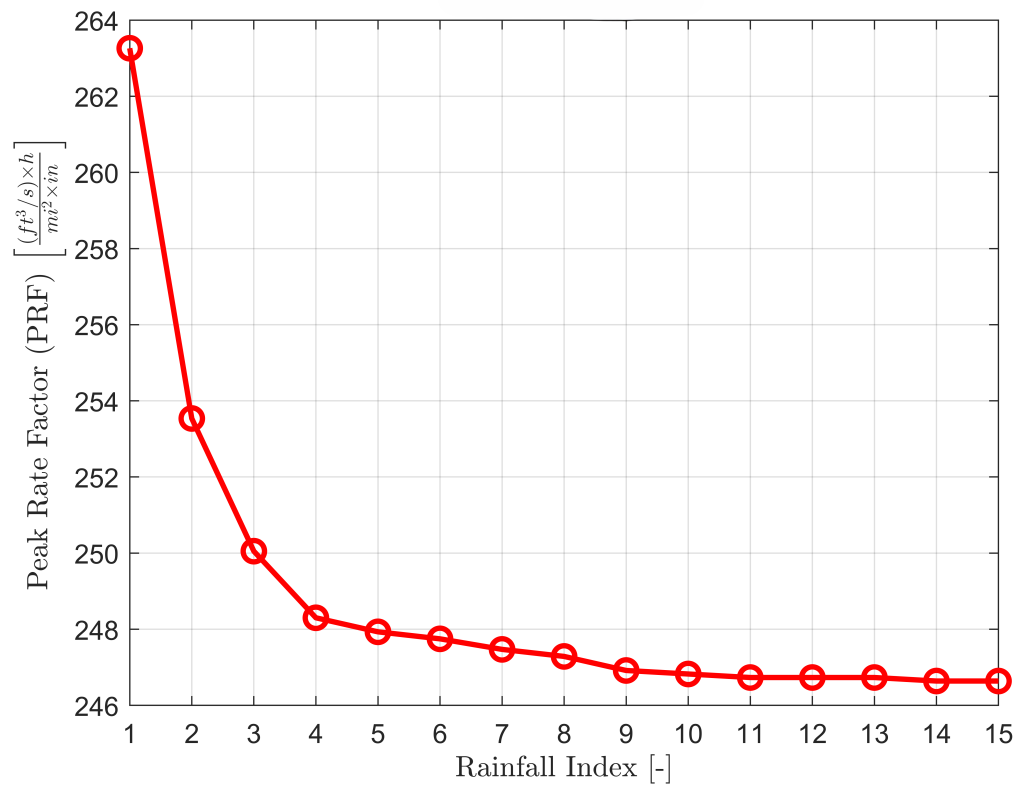


Figure 6.23: Peak Rate Factor vs. Rainfall Index

Table 6.5: Rainfall Intensity & Duration, Time to Peak, Peak Discharge, and PRF Values

Rainfall Index	Intensity [mm/hour]	Duration [minute]	Time to Peak [day]	Peak Discharge [cms]	PRF $\left[\frac{(ft^3/s) \times h}{mi^2 \times in} \right]$
1	0.41667	1440	9.900	0.1241	263.26
2	1	600	9.528	0.1241	253.54
3	2	300	9.400	0.1241	250.05
4	4	150	9.330	0.1241	248.30
5	5	120	9.316	0.1241	247.93
6	6	100	9.309	0.1241	247.75
7	8	75	9.299	0.1241	247.47
8	10	60	9.292	0.1241	247.29
9	20	30	9.278	0.1242	246.92
10	30	20	9.274	0.1242	246.83
11	40	15	9.271	0.1242	246.73
12	50	12	9.271	0.1242	246.73
13	60	10	9.271	0.1242	246.73
14	75	8	9.267	0.1242	246.64
15	120	5	9.267	0.1242	246.64

6.11 Conclusion

Among all 4 parameters tested in this chapter, the hydrograph shapes and peak discharge have the highest sensitivity to the surface roughness and the lowest sensitivity to the varying rainfall intensities and duration (depth of 1 cm) for the selected simulations. The PRFs calculated in these experiments show different behavior for each variable of interest. In summary, the PRF results collectively highlight that PRFs are influenced by a complex interplay of hydrological and physical landscape variables. The varying nature of PRF responses underlines the importance of considering a multi-variable approach in hydrological modeling to understand and predict runoff more accurately. Given that PRF is a mathematical factor to estimate runoff or a tuning factor to generate a watershed's UH, the results of this chapter and chapter 4 raise questions about its usefulness and accuracy

in estimating peak discharge.

This chapter has explored the effects of various watershed characteristics on the peak discharge and PRF using the GSSHA model across simplified, synthetic watersheds. The systematic investigation into surface roughness, side slope, longitudinal slope, and rainfall intensity and duration has provided insights into their hydrological impact.

6.11.1 Key Findings

1. **Surface Roughness:** There is a large sensitivity of hydrograph characteristics to surface roughness. As roughness increases, peak discharges are significantly reduced, demonstrating the critical role of surface texture in moderating hydrological responses.
2. **Slope Variations:** Both side and longitudinal slopes significantly influence the timing and magnitude of peak discharges; however, the time to peak is less affected by longitudinal slopes compared to side slopes. Steeper slopes lead to quicker and more intense runoff, underscoring their importance in watershed management and flood mitigation strategies. The relationship between slopes and runoff peak discharge is important as it helps to predict how changes in the slope of a watershed can affect the runoff flow rate, which is critical for flood forecasting, water resources management, and hydraulic and infrastructure design. The trend between slopes and peak discharge also suggests that steeper slopes may result in faster and higher peak water flows, which can have significant implications for erosion and stream restoration.
3. **Rainfall Characteristics:** The influence of rainfall intensity and duration on peak

discharge appears relatively inelastic beyond certain thresholds.

In addition to the key findings, the range of PRFs observed in these experiments varied significantly. The lowest PRF recorded was 184.64, associated with the experiment involving a longitudinal slope of 1%. Conversely, the highest PRF amounting to 277.07, was observed with longitudinal slope of 3.8%. A smaller PRF corresponding to the lower slope is consistent with current practice of applying PRF 284 instead of the standard 484 in the DelMarVa peninsula, where the land is very flat. By contrast, an increase in side slope decreased the PRF. When these results are compared with the PRFs estimated for Hurricane Sandy in Chapter 4, which span from 49 to 848, it is evident that the NRCS peak discharge equation yields a considerably constrained range for this parameter in hydrographs generated by the distributed model in simplified watershed.

6.11.2 Implication of the PRF Findings

1. **Modeling Limitations:** The results suggest that current modeling of open-book watershed may be overly simplistic or lack the necessary complexity to capture the factors that influence the peakedness of hydrographs.
2. **Need for Comprehensive Parameters/Models:** The varying results for PRF prediction from basic watershed parameters underscore the potential need for integrating more comprehensive hydrological processes in distributed models (e.g., GSSHA). That may also indicate that using PRF to estimate peak discharge has uncertainties due to the assumptions implicit in its mathematical definition. Further analysis can include considering aspects such as detailed soil layer interactions, sub-

surface flow dynamics, channelization, and the complex interplay between vegetation cover and hydrological response, which are not typically modeled in simplified setups like the open-book watershed.

6.12 Benefit and Contribution

The time to peak, peak discharge, and the shape of the unit hydrograph are vital parameters in hydrologic practice. These parameters are influenced by a multitude of factors in a real watershed. To the best of the author's knowledge, no distributed model has investigated the impact of watershed characteristics on peak discharge and PRF for a unit hydrograph generated within a simplified open-book watershed. A deeper understanding of the watershed system as a whole can be gained by comprehending how each experiment impacts these parameters, potentially laying the groundwork for future research and development.

6.13 Recommendations for Future Research

1. **Incorporating More Realistic Features:** Future studies should aim to include more realistic watershed features and complex interactions in hydrological models. This could involve the integration of land use, soil data, and detailed channel network.
2. **Empirical and Field-Based Studies:** There is a need for empirical studies that involve extensive field/experimental data to validate the PRFs predicted by models. Such studies can provide critical insights into the actual behaviors of watersheds under varied hydrological conditions and help refine the predictive accuracy of hydrological

models.

3. **Advanced Computational Techniques:** Utilizing advanced computational techniques, such as machine learning algorithms, could provide new ways to analyze and predict the impacts of various watershed characteristics on PRFs. These techniques can handle more complex and non-linear relationships that traditional models may not adequately capture.

Chapter 7: Concluding Remarks and Recommendations

This dissertation focuses on the estimation of the Natural Resources Conservation Service (NRCS) Unit Hydrograph (UH) and its parameters consisting of time of concentration (t_c) and peak rate factor (PRF) from radar precipitation data and gridMET data. The subjectivity of t_c in hydrology stems from the different definitions in the literature, methods employed in its estimation, the assumptions inherent in these methods, and the parameter's sensitivity to input data and watershed features. These methods consider various watershed attributes, including topography, land cover, soil type, and flow path length. Each of these methods possesses unique advantages and disadvantages, rendering them more appropriate for specific circumstances or watershed types. The selection of a suitable method typically hinges on data availability, quality of the data, watershed properties, and the intended application. Among all these methods and all other empirical equations discussed in this literature for various geographical locations, it appears that NRCS UH method is perhaps the best and most robust method in current hydrologic practice.

The results from Chapter 3 clearly demonstrate that a unique UH and t_c do not exist for a given watershed; instead, they vary among different storm events. Additionally, no correlation was found between the PRF and storm event size, and this parameter alone cannot explain the variations in PRF values. This chapter further explores the evidence

that t_c is not an invariant property of a watershed and fluctuates with each storm event.

Chapter 4 of this research focuses on PRF estimations for Hurricane Sandy events in selected Maryland watersheds. The variations in PRFs could not be explained by the selected watershed characteristics, and no regional or spatial trends were observed.

Chapter 5 of this research concentrates on using a distributed model to estimate hydrographs in the Jones Falls watershed. The first research question to assess the assumptions of NRCS UH using a fully distributed model could not be addressed due to the complex model components and lack of time and resources. However, the second research question to assess the role of the detailed soil profile on the runoff estimation was addressed. The role of the detailed soil profile in runoff estimation is highlighted by subtle yet meaningful variations in simulated peak discharges from the Jones Falls watershed, underscoring differences that warrant further examination.

Chapter 6 of this study evaluated the impact of watershed characteristics on the time to peak, peak discharge, and peak rate factor (PRF) in a simplified open-book watershed. This research demonstrated that in such a model, the PRF marginally decreases in response to higher Manning's roughness coefficients. The PRF consistently shows a downward pattern with steeper side slopes. Conversely, the PRF exhibits a consistent upward trend with steeper longitudinal slopes. Additionally, during the rainfall intensity and duration experiments, the PRFs sharply decreased from the first rainfall index (lowest intensity and longest duration) to the fourth, and then gradually diminished through to the fifteenth index (highest intensity and shortest duration). Therefore, the variability observed in PRFs within this study's open-book watershed model cannot be solely explained by fluctuations in these parameters. This suggests that each of these parameters plays a role in influencing

PRF values under the specific conditions tested.

The overall results lead to the conclusion that there are uncertainties involved in estimating t_c and PRF. If high-quality data and resources are available, using a fully distributed model should be preferred, as these models do not rely on t_c or a UH for runoff modeling. Furthermore, the usefulness of the PRF to estimate runoff and peak discharge is unclear, as this parameter does not exhibit consistent behavior across different watershed characteristics. Due to its mathematical nature, it can be estimated using triangular approximation or NRCS peak discharge equation to any generated unit hydrograph, regardless of the approach employed. Using high-quality and adequate data with a fully distributed model containing comprehensive components might shed some light if application of PRF is justifiable in the field of hydrology. There is a very limited body of research concerning the PRF in hydrology. The current understanding of this area remains constrained due to the relatively small number of studies, indicating a pressing need for more comprehensive and rigorous research to better understand, quantify, and validate the parameter and justify its use in practice. Furthermore, the findings of this study suggest PRF is not exclusively limited to the NRCS UH; rather, it is a mathematical property inherent to a hydrograph and can also be applied to any UH, including those generated using distributed hydrologic models. This factor is not a physical characteristic of the watershed itself but instead serves as a mathematical descriptor of the watershed's response to precipitation events, which is one of the reasons its predictability and variability is very complex. Blind application of PRF 484 might result in erroneous peak discharge estimation.

7.1 Novelty and Contributions

This research has provided several contributions:

- **Less Subjective Estimation of Time of Concentration:** In this study, a more objective approach was employed to estimate the time of concentration using a mathematical inflection point method. This less subjective methodology enhances the accuracy of t_c estimation compared to conventional methods that often rely on subjective judgments. By utilizing a more objective approach, this research contributes to improving the precision and reliability of hydrological modeling.
- **Advanced Precipitation Data Analysis and Modified Baseflow Separation Method:** This research utilized high-resolution precipitation data (6-minute intervals) to capture the intricate dynamics of rainfall events. Additionally, a modified concave method was applied for baseflow separation, resulting in more accurate estimations of the direct runoff component using an exponential decay function. The combination of fine-resolution precipitation data and the enhanced baseflow separation technique improves the accuracy of the results, leading to more reliable hydrological predictions.
- **Parameter Testing for Unit Hydrographs:** A novel aspect of this study is the testing of watershed parameters on the PRF of unit hydrographs generated by a distributed model without imposed time parameters, particularly t_c , which adds a great deal of uncertainty to rainfall-runoff models. This innovative approach provides a more flexible and nuanced model that can be adapted to various scenarios. By

eliminating the reliance on time parameters, this research offers a more versatile tool for hydrological modeling.

- **Extending Peak Rate Factor beyond 484 and NRCS DUH:** This research extends the applicability of the PRF beyond the limitations of the NRCS PRF of 484 in dimensionless unit hydrograph. By demonstrating the effectiveness and estimation of the PRF in non-NRCS unit hydrographs, this study broadens the scope of existing hydrological models that relate PRF to NRCS DUH. The extended applicability of the PRF enables its use in a wider range of watersheds and scenarios using different models.
- **Impact of Detailed Soil Data on Runoff:** This study investigates the influence of detailed soil data on runoff hydrographs and peak discharge. By incorporating comprehensive soil data into the distributed model, this research provides insights into the significance of soil characteristics in determining runoff and peak discharge behavior. The findings emphasize the importance of considering detailed soil data in hydrological studies, as it can impact the accuracy of runoff predictions and the understanding of watershed response to precipitation events.
- **First to Use Fully Distributed Model to Generate Unit Hydrograph:** To the best of the author's knowledge, this research is the first to investigate the impact of watershed characteristics on peak discharge and PRF for a UH generated within a simplified open-book watershed using a distributed model. This approach offers a unique perspective on the relationship between watershed properties and hydrological response, providing valuable insights regarding PRF usefulness for future studies and

its applications in hydrologic practice.

7.2 Limitations

The research presented in this dissertation encompasses several limitations. First, although a less subjective method was utilized to estimate the time of concentration, there remains a lack of global consensus regarding its definition. Despite employing an innovative approach for baseflow separation, the complexities and inherent uncertainties of watershed behavior persist in every baseflow separation technique. In addition, the uncertainty associated with the Phi-index approach is intrinsic to the method itself. While alternative methods such as the NRCS Curve Number exist, each comes with its distinct set of benefits and limitations.

Chapter 5 faced challenges due to the absence of a graphical user interface (GUI), time and available resources and the complexity of various model components, which necessitated a revision of the original objectives. Rather than assessing the distributed rainfall impact on the assumptions of NRCS UH, the focus shifted towards developing GIS and spreadsheet methods to assign distributed watershed parameters, and to evaluating the impact of a detailed soil profile on runoff peak discharge in the Jones Falls watershed.

In Chapter 6, the use of an overly-simplified open-book watershed to study the relationship between watershed characteristics and both peak discharge and PRF was a significant limitation. The model did not include critical components such as channel networks, infiltration, and interception. Moreover, the parameter ranges used in the experiments were arbitrarily selected, lacking real data to support these experimental designs.

7.3 Future Work

Based on the results and conclusions provided by this research, several recommendations for future work are as follows:

- **Advanced Testing of Unit Hydrograph Assumptions with Distributed Models Using Spatially Distributed Rainfall:** Future studies should focus on using fully distributed hydrological models to critically assess the foundational assumptions of UH theory, especially in the context of spatially distributed rainfall. Traditional UH methods assume a linear and time-invariant response to rainfall, which may not accurately reflect the complex interactions and non-linear behaviors exhibited by real watersheds under variable meteorological conditions. By employing fully distributed models, researchers can simulate the hydrological response of watersheds to rainfall with high spatial and temporal resolution, thereby uncovering the limitations and potential inaccuracies of the UH assumptions. This approach would involve detailed simulations where rainfall inputs are varied across different sections of a watershed to observe how changes in the spatial distribution of rainfall affect the resulting hydrographs. Such studies are essential for understanding the scale and impact of heterogeneities in watershed properties, such as soil saturation, land use, and topography, on runoff generation and routing. The findings could lead to significant enhancements in flood forecasting and watershed management, providing a more robust theoretical basis for hydrological modeling.
- **Estimating Peak Rate Factors Using Fully Distributed Models with Com-**

prehensive Real Data: Future research should focus on estimating PRFs using fully distributed models that incorporate comprehensive real data and all relevant watershed components. This approach seeks to address the limitations identified in traditional hydrological modeling, which often simplifies and overlooks complex interactions within the watershed. By integrating real data from soil characteristics, vegetation patterns, land cover/use, channel network, infiltration, and interception into fully distributed models, researchers can achieve a more nuanced understanding of how various watershed characteristics influence runoff hydrographs and obtain more realistic results.

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