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Unit Hydrograph Peaking Analysis for Goose Creek Watershed in Virginia: A Case Study

Nawa Raj Pradhan and Drew Loney   May 2017
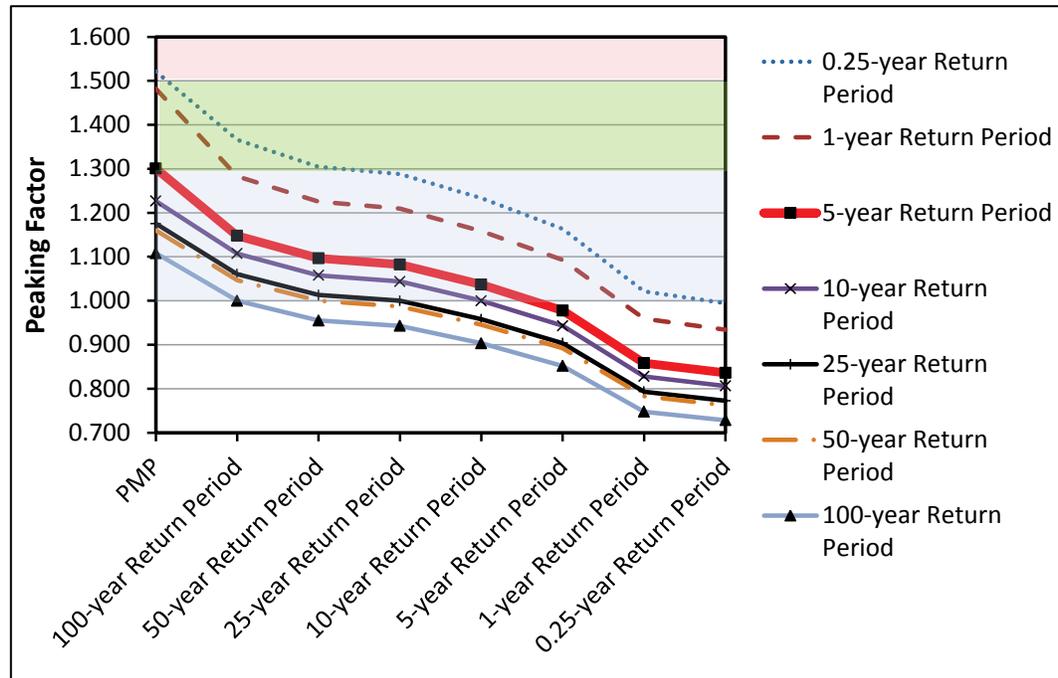
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Final report

Abstract

Existing U.S. Army Corps of Engineers (USACE) policy suggests unit
hydrograph peaking factor (UHPF), the ratio of an observed event unit
hydrograph peak to the modeled one for a catchment, ranges between
1.25 and 1.50 to ensure dam safety. It is pertinent to investigate the impact
of extreme flood events on the validity of this range through physically
based rainfall-runoff models not available during the planning and design of
most USACE dams. The UHPF range was analyzed by deploying the
Gridded Surface Subsurface Hydrologic Analysis model in the Goose Creek,
VA, watershed to develop a peaking factor relationship with excess rainfall
for National Oceanic and Atmospheric Administration rainfall depths across
various return-period events as well as the National Weather Service
probable maximum precipitation event. The effect of statistical uncertainty
on UHPF at Goose Creek was determined by applying the 5% and 95%
confidence interval precipitation depths to the watershed in addition to the
50% value. This study concluded that a design event with a return period
greater than 5-year is required for the unit hydrograph to ensure dam
safety. Goose Creek unit hydrograph was designed from 25-year return
period rainfall and therefore ensures dam safety given the current UHPF
guidance.

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Preface

This study was conducted for the Risk Management Center, U.S. Army Corps of Engineers, under the Project "Unit Hydrograph Peaking Analysis for Dam Safety." The technical monitors were Mr. David Margo (CEIWR-RMC) and Dr. Chandra Pathak (CECW-CE).

The work was performed by the Hydrologic Systems Branch (HF-H), Flood and Storm Protection Division (HF), U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL). At the time of publication of this report, Dr. Pearce Cheng was Branch Chief, CEERD-HF-H; Dr. Cary Talbot was Chief, CEERD-HF. Mr. Jeffrey R. Eckstein was the Deputy Director of ERDC-CHL, and the Director was Mr. José E. Sánchez.

COL Bryan S. Green was the Commander of ERDC, and the Director was Dr. David W. Pittman.

Unit Conversion Factors

Multiply	By	To Obtain
acres	4,046.873	square meters
acre-feet	1,233.5	cubic meters
centistokes	1.0 E-06	square meters per second
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
fathoms	1.8288	meters
feet	0.3048	meters
gallons (U.S. liquid)	3.785412 E-03	cubic meters
hectares	1.0 E+04	square meters
inches	0.0254	meters
knots	0.5144444	meters per second
microns	1.0 E-06	meters
miles (nautical)	1,852	meters
miles (U.S. statute)	1,609.347	meters
miles per hour	0.44704	meters per second
mils	0.0254	millimeters
ounces (U.S. fluid)	2.957353 E-05	cubic meters
pints (U.S. liquid)	4.73176 E-04	cubic meters
quarts (U.S. liquid)	9.463529 E-04	cubic meters
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
square miles	2.589998 E+06	square meters
square yards	0.8361274	square meters
yards	0.9144	meters

1   Introduction

1.1   Background

The U.S. Army Corps of Engineers (USACE) policy contained in ER 1110-8-2 (FR) (USACE 1991) requires that dams "designed, constructed, or operated by the USACE will not create a threat of loss of life or inordinate property damage." For a dam to be considered "hydrologically adequate" and in compliance with the USACE policy, an appropriate embankment, spillway, and regulating outlet, if necessary, must be designed to safely pass an Inflow Design Flood with adequate freeboard. USACE engineering practice utilizes extreme flood events to evaluate and manage flood risk at most dams. Extreme flood events are those events defined by large magnitudes and long return periods. However, it is unclear whether the simplifications employed for selecting design extreme flood events has led to over- or under-estimations of the watershed response and in turn, over- or under-estimations of the required dam size. It is thus pertinent to investigate the impact of extreme flood events through the use of physically based rainfall-runoff models that were not available during the design and construction of most USACE dams to improve dam safety estimates.

In the past, the most common rainfall-runoff models used within the USACE for dam construction assumed both an initial and constant water loss to generate excess precipitation. The unit hydrograph theory, the most frequently used of these methods, transforms excess precipitation to a point runoff hydrograph. According to Sherman (1932), who originally proposed the unit hydrograph (UH) concept, the UH of a watershed is the outflow resulting from one unit of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration. This implies that the ordinates of any hydrograph resulting from a quantity of runoff-producing rainfall of unit duration would be equal to corresponding ordinates of a UH for the same areal distribution of rainfall, multiplied by the ratio of rainfall excess values. For the past 50 years, the USACE practice has been to increase the peak of a UH linearly, in relation to the observed rainfall, by 25% to 50% when the observed rainfall excess is significantly greater than the modeled rainfall excess used to calibrate the UH (USACE 1991). This multiplicative factor is typically referred to as the unit hydrograph peaking factor (UHPF).

Natural variability of environmental systems often results in linear transfer functions (Childs 1958; Minshall 1960; Ding 1974; Kokkonen et al. 2004), as assumed in UH theory, to poorly characterize the range of possible outcomes from the same initial state. Linear transfer functions can be invalid due to multiple factors including differences in the varying antecedent initial conditions such as soil moisture (Pradhan et al. 2012; Hendrickx et al. 2016); runoff generation processes like infiltration excess (Downer and Ogden 2004); saturation excess (Pradhan and Ogden 2010); and hydro-geomorphology, urbanization, and land use change (Ogden et al. 2011). In most extreme flood investigations, the UHPF is rarely analyzed thoroughly in the context of natural watershed variability. Variability ultimately manifests as uncertainty in the relationship between precipitation and flood likelihood. A thorough analysis method relating hydrologic variables to the UHPF is essential to remove the assumption of linear behavior imposed by commonly employed empirical modeling methods. The analysis should also consider and quantify the aforementioned uncertainty to provide confidence information to decision makers.

Physically based hydrologic modeling is capable of testing the linearity and non-linearity of the watershed response (Szilagyi 2007). The Gridded Surface/Subsurface Hydrologic Analysis (GSSHA) model is a distributed-parameter, physically based numerical model that has been used by the USACE in engineering hydrology (Downer et al. 2016); erosion (Downer et al. 2014); contaminant/nutrient fate and transport (Pradhan et al. 2014); as well as storm surge studies (Massey et al. 2013). Deploying a physically based hydrologic model will account for nonlinear watershed hydrological properties (Szilagyi 2007) as well as assist in developing a methodology to analyze/improve the estimation of peaking factor (USACE 1991) and rainfall excess relationships in different climatic regions of continental United States (Wolock et al. 2004).

1.2 Purpose

Existing USACE policy suggests UHPFs between 1.25 (125%) and 1.50 (150%). In other words, although a UH peak is assumed to increase linearly with the increase in the intensity of a precipitation event, the increment would not exceed 1.5 times the designed unit peak. The purpose of this study is to analyze the validity of this UHPF range of the Goose Creek watershed in Virginia via deploying the GSSHA model, a physically based hydrologic model that is capable of determining complex watershed behavior. Goose Creek watershed was selected because of the availability

of existing baseline data from previous studies. This study also investigated the magnitude of a rainfall event associated with a 1.25 or 1.5 UHPF and developed a peaking factor relationship with excess rainfall in the study area.

This study is intended to serve as an example for deploying physically based hydrological models to analyze the UH and the peaking factor on other watersheds. If the UH and the peaking factor analyses for watersheds representing different climatic regions of continental United States are conducted in the same manner, one would develop a regionalized peaking factor relationship with excess rainfall, which potentially could be utilized to evaluate and improve, if needed, the safety of existing dams.

2   Study Area and Data

2.1   Study area

USACE, Baltimore District (NAB) (1963), provides details on the derivation of standard UH in the Goose Creek watershed in Loudoun County, VA (Latitude: 39.019; Longitude: 77.577; Area: 339 square miles [miles^2]). The objectives of this 1963 study were the following:

* to collect a maximum of available information on UH methods and UH data
* to provide standardized methods, terminology, and forms for the presentation of UHs
* to present the accumulated data and information in a form useful to hydrologists.

Goose Creek watershed, shown in Figure 1, was selected as the study area for this pilot study of UHPF using the GSSHA model. The principal reason for selecting this watershed is that the watershed study by USACE (1963) provides baseline information on the UH data along with the standard practice in deriving the UH. The study area is located in the eastern United States, bounded by the Potomac River and Maryland on the northeast. The Goose Creek drains eastward into the Potomac River. The climate of Virginia is diverse and varies from warm and temperate in the eastern coastal areas, which have temperatures moderated by the Atlantic Ocean, to the cooler continental climate of the mountainous counties in the north and west (Sanford et al. 2011). The mean annual temperature is between 54 and 55 °F. The rainfall pattern varies across Virginia and is affected by topography in the north and west and by the presence of tropical moisture systems in the south and east. The mean annual precipitation in the study area is 42 inches. Temperature and rainfall are adequate to support a substantial agriculture industry, with crop and pasture lands evenly scattered between forests of mixed deciduous and evergreen trees across most of Virginia including the watershed (Sanford et al. 2011).

Figure 1. Goose Creek watershed (in Loudoun County of Virginia).
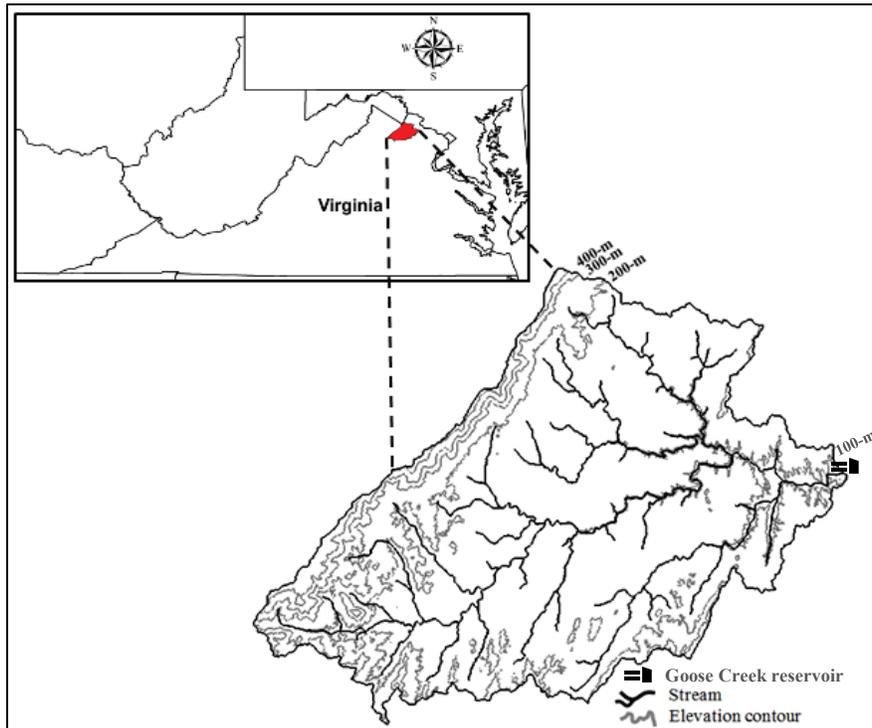
2.2   Data

The hourly precipitation data was obtained from Dulles International Airport in Dulles, VA (ID No. Coop – 448903). The discharge data at the outlet in Figure 1 were from U.S. Geological Survey station No. 1644000. Land use data was obtained from the National Land Cover Database (http://www.mrlc.gov/finddata.php). The soil data were obtained from the Natural Resources Conservation Service, Soil Survey Geographic Database (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/survey/geo/?cid=nrcs142p2_053631). The probable maximum precipitation (PMP) estimates was obtained from the Hydrometeorological Report No. 51 (USDoC 1978). Historical precipitation for Goose Creek is well characterized by the Precipitation Frequency Atlas 14, Volume 2, of the National Oceanic and Atmospheric Administration (NOAA). The Precipitation Frequency Atlas is the standard source of historical extreme precipitation information available in the continental United States. The return period precipitation was estimated from the NOAA Frequency Atlas 14 (http://hdsc.nws.noaa.gov/hdsc/pfds/).

3   Methodology

Two different methods were used to derive UHs for the Goose Creek watershed:

1. Utilizing historic discharge and rainfall data
2. Utilizing GSSHA to perform two-dimensional (2D) watershed modeling to derive a UH from the return period events and the PMP event.

3.1   Utilizing historic discharge and rainfall data to derive a unit hydrograph (UH)

As per UH theory, the following steps were taken to derive a UH from the observed discharge:

1. Separate the base flow from the observed streamflow hydrograph to obtain the Direct Runoff Hydrograph QDRH. The steps for base flow separation are as follows:
    a. Identify when direct runoff begins (i.e., end of the base flow-only period).
    b. Estimate the duration of the direct runoff period N using the empirical relationship below:

$$N = A^{0.2} \tag{1}$$

where N is in days; A is the basin area in square miles (Linsley et al. 1975).

2. Draw a line connecting the start and end of the time interval for the direct runoff. This line separates the direct runoff hydrograph from base flow hydrograph. Then, compute the volume of Direct Runoff Hydrograph, V_{DRH}. This volume must be equal to the volume of the Effective Rainfall Hyetograph (ERH).

$$V_{DRH} = \int_t Q_{DRH}(t')dt' \tag{2}$$

where t is the time.

3. Express V_{DRH} in equivalent units of depth, V_{DRHD}:

$$V_{DRHD} = V_{DRH} / A_{basin} \qquad (3)$$

where A_{basin} is the basin area.

4. Obtain the UH by normalizing the Q_{DRH}. Normalizing implies dividing the ordinates of the Q_{DRH} by the V_{DRH} in equivalent units of depth in Equation (3).

$$UH = Q_{DRH} / V_{DRHD} \qquad (4)$$

5. Determine the duration, D, of the ERH associated with the UH obtained in step 4 above.

Figure 2 shows the derivation of the UH for the Goose Creek watershed from the 10 December 2003 observed rainfall and discharge. In deriving the UH in Figure 2, Equations (1) through (4) were employed. Frequently, data are not available for isolated storms of sufficient runoff volume for the derivation of UHs. Because the requisite data are not available, the investigator must develop UHs from the records of complex storms with durations of rainfall excess that differ appreciably from the unit duration desired. In Figure 2, the 6-hour (hr) UH was derived from the 9 hr UH from S-hydrographs (USACE NAB 1963). The guidelines for computation of S-hydrographs and derivation of UHs from S-hydrographs is found in USACE NAB (1963).

Figure 2. Deriving a UH for the Goose Creek watershed.
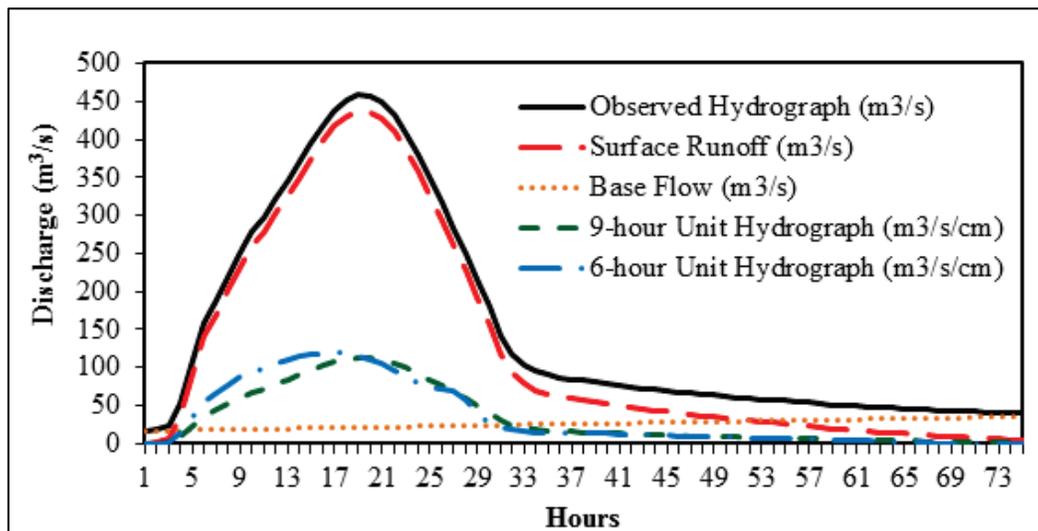
3.2　Utilizing GSSHA to perform two-dimensional (2D) watershed modeling to derive a UH

Hydrologic models play a significant role in transforming extreme return period rainfall events and probable maximum precipitation into corresponding design discharges. In this study, the physically based GSSHA model was deployed to obtain corresponding design discharge from probable rainfall events.

3.2.1　GSSHA model simulation

GSSHA computes one-dimensional (1D) infiltration, 2D overland flow, and 1D stream flow to simulate the overland runoff to streams and channel transport process to catchment outlet. The Green and Ampt infiltration scheme (Green and Ampt 1911) was employed to simulate the partitioning of input precipitation at the land surface. The runoff generated in each computational grid cell flows to adjacent cells through a 2D finite volume numerical scheme. Surface runoff is coupled to channel routing where lateral inflow from surface runoff grid cells is numerically routed through a 1D finite volume scheme.

3.2.2　Infiltration model

The infiltration model partitions the rainfall forcing into subsequent runoff. The rate of infiltration in the infiltration model is governed by the soil physical properties, which vary with the soil type. The soil map in Figure 3 was employed to define these properties. The following are the soil physical parameters specified in the infiltration model:

- effective porosity (cubic centimeter/cubic centimeter)
- field capacity (cubic centimeter/cubic centimeter)
- wilting point (cubic centimeter/cubic centimeter)
- residual saturation (cubic centimeter/cubic centimeter)
- pore size distribution (centimeter/centimeter)
- saturated hydraulic conductivity (centimeter/hour)
- wetting front suction head (centimeter).

Figure 3. Goose Creek soil classification.
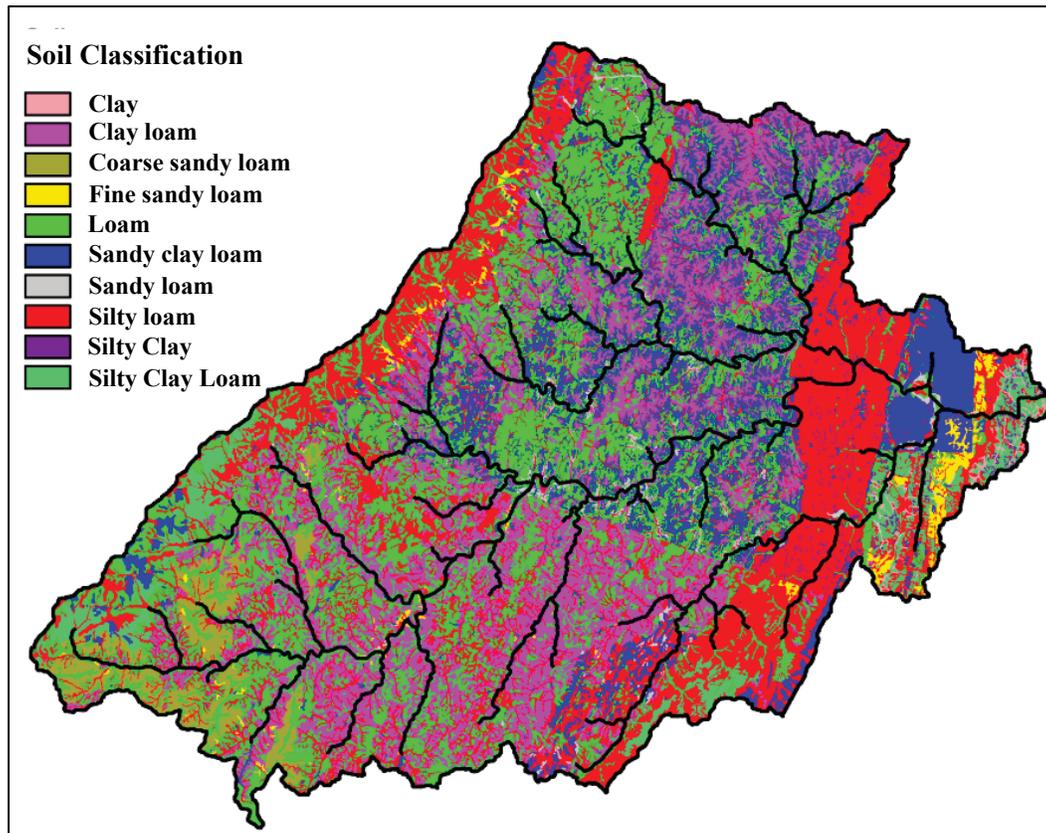
Apart from hydraulic conductivity, the parameters in Table 1 were obtained from Rawls et al. (1983) based on the soil classifications in Figure 3. Hydraulic conductivity in Table 1 was manually calibrated. To account for impervious land, developed areas in Figure 4 with more than 20% impervious land cover were assigned a significantly lower value of hydraulic conductivity, 0.0005 centimeters/hour (cm/hr). This impervious developed area covers less than 3% of the watershed.

Table 1. Soil properties and infiltration parameters.

Soil Type	Hydraulic Conductivity (cm/hour)	Capillary Head (cm)	Porosity (cm³/cm³)	Pore Size Distribution (cm/cm)	Residual Saturation (cm³/cm³)	Field Capacity (cm³/cm³)	Wilting Point (cm³/cm³)
Clay	0.004	31.63	0.380	0.165	0.09	0.379	0.15
Clay loam	0.004	20.88	0.385	0.242	0.075	0.375	0.15
Coarse sandy loam	0.012	11.01	0.390	0.378	0.041	0.375	0.095

Soil Type	Hydraulic Conductivity (cm/hour)	Capillary Head (cm)	Porosity (cm³/cm³)	Pore Size Distribution (cm/cm)	Residual Saturation (cm³/cm³)	Field Capacity (cm³/cm³)	Wilting Point (cm³/cm³)
Fine sandy loam	0.012	11.01	0.390	0.378	0.041	0.370	0.095
Loam	0.005	8.89	0.400	0.252	0.027	0.375	0.117
Sandy clay loam	0.001	21.85	0.380	0.319	0.068	0.375	0.148
Sandy loam	0.012	11.01	0.395	0.378	0.041	0.375	0.095
Silt loam	0.004	16.68	0.410	0.234	0.015	0.375	0.15
Silty clay	0.008	29.22	0.395	0.15	0.056	0.378	0.15
Silty clay loam	0.008	27.3	0.410	0.177	0.04	0.376	0.15

Figure 4. Goose Creek land use.

Land use
- Open water
- Developed open space
- Developed low intensity
- Developed medium intensity
- Developed high intensity
- Barren land
- Deciduous forest
- Ever green forest
- Mixed forest
- Shrub / Scrub
- Grassland / Herbaceous
- Pasture / Hay
- Cultivated crops
- Woody wetland
- Herbaceous wetland
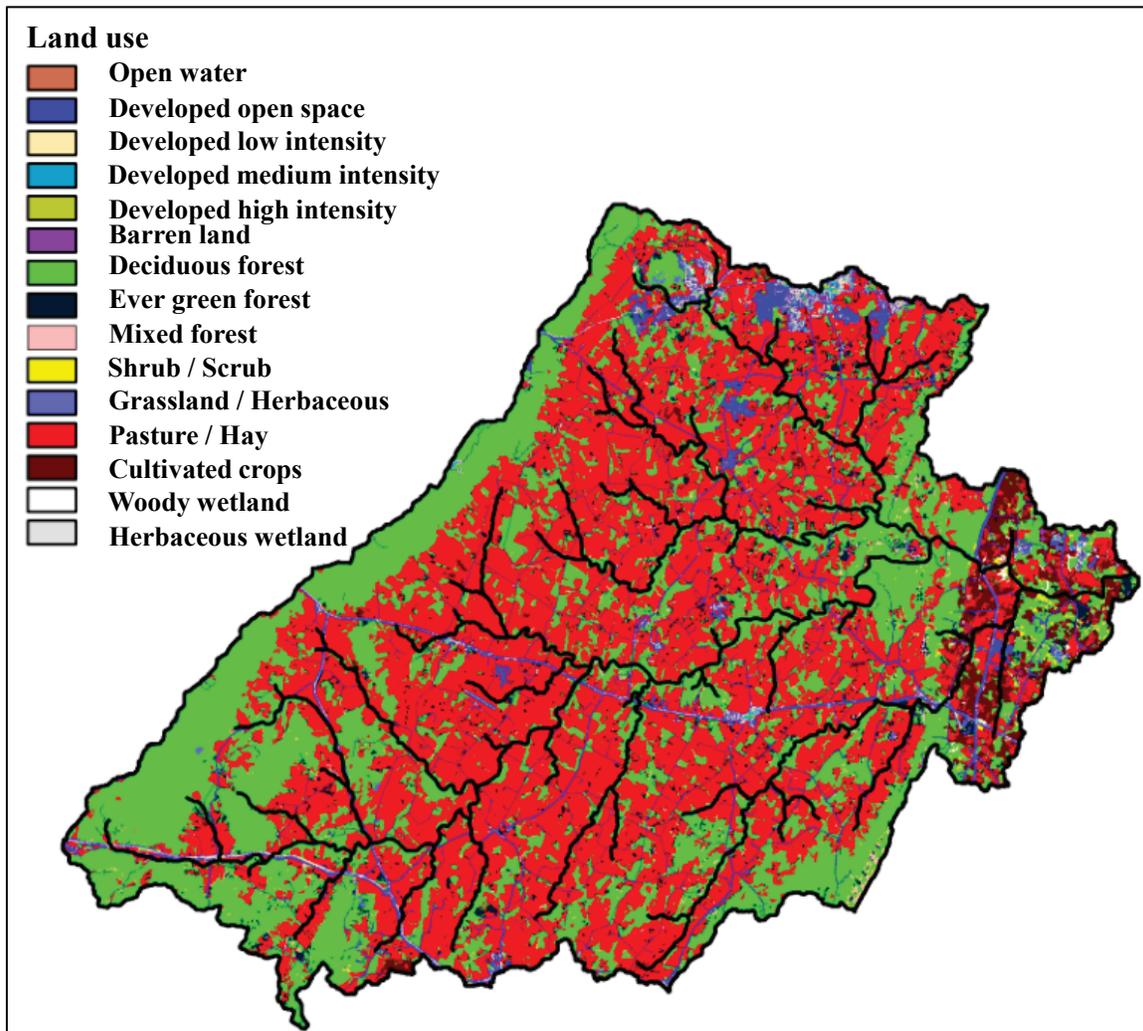
3.2.3 Routing model

GSSHA employs an explicit finite volume scheme to route water for both 1D channel flow and 2D overland flow, where flow rates are computed based on heads, and volumes are updated based on the computed flow rates. The friction slope between one grid cell and its neighbors is calculated as the difference in water surface elevations divided by the grid cell size. Compared with the kinematic wave approach, this diffusive wave approach allows GSSHA to route water through pits or depressions and regions of adverse slope. Manning's equation (Manning 1891) relates normal flow depth to discharge. Hydrological models implement Manning's equation to relate surface roughness to flow rate, in which case the hydraulic roughness is represented by the Manning's roughness coefficient, n. For each land use type shown in Figure 4, a Manning's roughness coefficient, n, was calibrated within the ranges listed on the GSSHA wiki (http://www.gsshawiki.com/Surface_Water_Routing:Overland_Flow_Routing). Table 2 lists the calibrated Manning's roughness coefficients in the study area.

Table 2. Land use type and overland flow hydraulic roughness.

Land use type	Roughness
Water	0.045
Developed (impervious 20%)	0.3
Developed (impervious 20-49%)	0.07
Developed (impervious 50-79%)	0.013
Developed (impervious 80-100%)	0.011
Barren land (vegetation < 15%)	0.4
Deciduous forest	0.492
evergreen forest	0.492
mixed forest	0.492
Shrub	0.48
Grassland	0.45
Pasture	0.45
Cultivated	0.45
Woody wetlands	0.45
Herbaceous wetlands	0.45

3.2.4 Calibration

The model was calibrated to the 9 December 2003 event as shown in Figure 5.

Figure 5. Goose Creek catchment GSSHA calibration (Nash-Sutcliffe Efficiency was 90%).
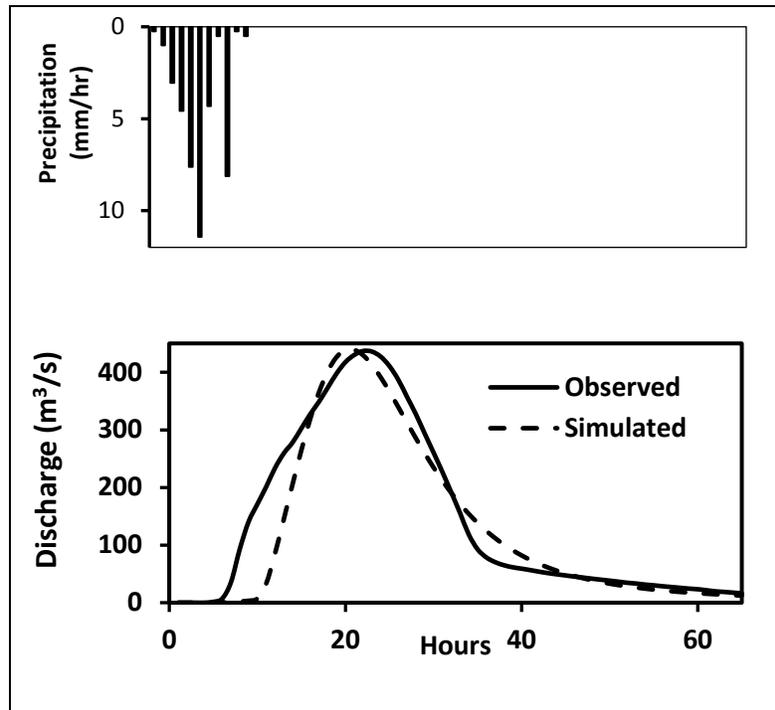
The calibrated model was then deployed to simulate the various return period events.

4   Results and Discussion

The 6 hr historic cumulative event precipitation was obtained from the NOAA ATLAS 14 Precipitation Frequency Data Server (PFDS), http://hdsc.nws. noaa.gov/hdsc/pfds/. The PFDS gives the 1-year, 5-year, 10-year, 25-year, 50-year, and 100-year return period events in the Goose Creek watershed as shown in Figure 6. To derive these series, NOAA collects precipitation measurements from all available sources, performs data correction and outlier removal, and computes extreme event magnitudes with confidence intervals for various durations and return periods. Following an initial data correction process, the annual maximum series is extracted at each measurement site for common durations of interest. A generalized extreme value distribution (GEV) is then fitted to the annual maximum series using L-moments with final distribution parameters obtained by a regional average. The GEV distribution provides the likelihood of a particular magnitude occurring for the extracted duration, which permits the annual exceedance probability to be determined. The return period of an event magnitude can be directly obtained from the inverse of the annual exceedance probability. Common event durations and return periods are summarized in the PFDS, which also provides gridded interpolation between gages.

Additionally, NOAA determines data processing confidence intervals by applying a Monte Carlo procedure to the GEV distribution to obtain the 90% confidence limit (the 5% lower bound and 95% upper bound). NOAA confidence limits only characterize the certainty of the data processing technique used to obtain the return period estimates. Additional sources of uncertainty, such as gage clustering and imperfect historical knowledge, are not represented and would only increase the NOAA-calculated uncertainty limit. However, the user of GEV data must account for the statistical confidence during hydrologic analysis. The NOAA confidence intervals at each return period were applied to the watershed and are given in Figure 6.

NOAA Precipitation Frequency Atlas also provides the temporal event profiles. Event accumulation rates were determined by applying a fixed window at the selected duration to the precipitation series. The deciles of the accumulation rate for each duration are provided graphically from

these events. In addition, the event temporal distributions are further divided into quartiles by magnitude and the accumulation deciles calculated for each magnitude quartile. In Figure 6, design return period events were created by choosing 6 hr accumulation return period precipitation magnitudes in combination with an appropriate temporal distribution provided by the NOAA PFDS, http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html.

Figure 6. NOAA Atlas 14; 6 hr return period precipitation event temporal distribution estimation in the study area.
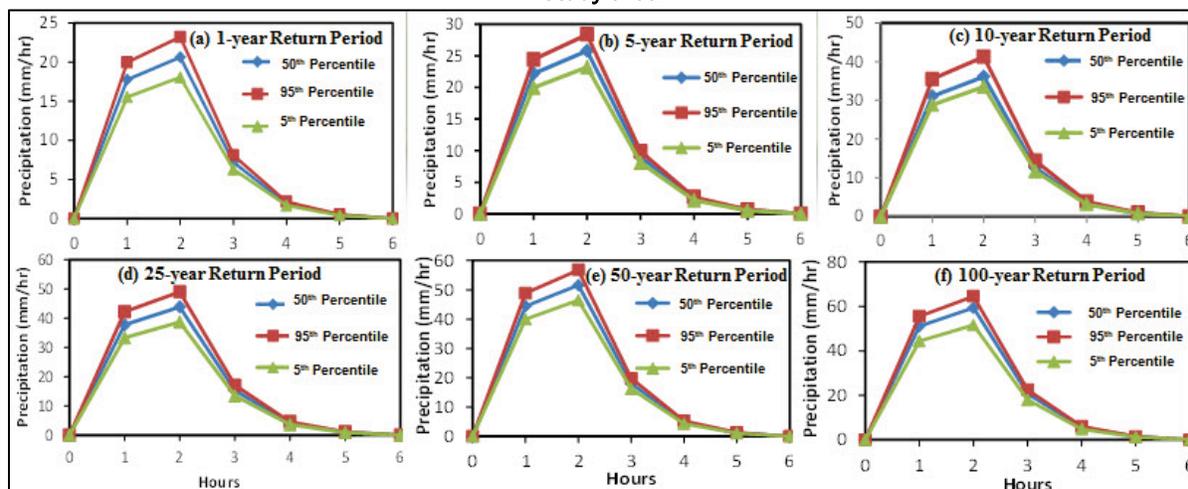
The return period events shown in Figure 6 along with the PMP event are deployed in the calibrated GSSHA model. Figure 7 shows the GSSHA model runoff simulation of the return period events.

Figure 7. NOAA 50th percentile return period discharge from GSSHA simulations.
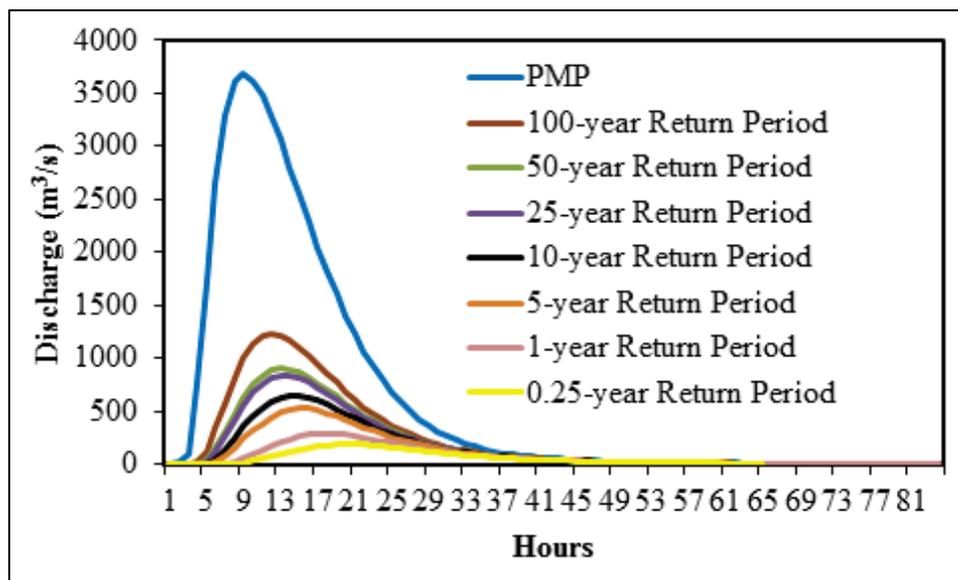
Figure 8 shows the UH obtained from the GSSHA model runoff simulations in Figure 7. The procedure followed in deriving the UH in Figure 8 is described in section 4.3.1 where the GSSHA-simulated runoff was employed instead of observed runoff.

Figure 8. NOAA 50th percentile return period UH from GSSHA simulations.
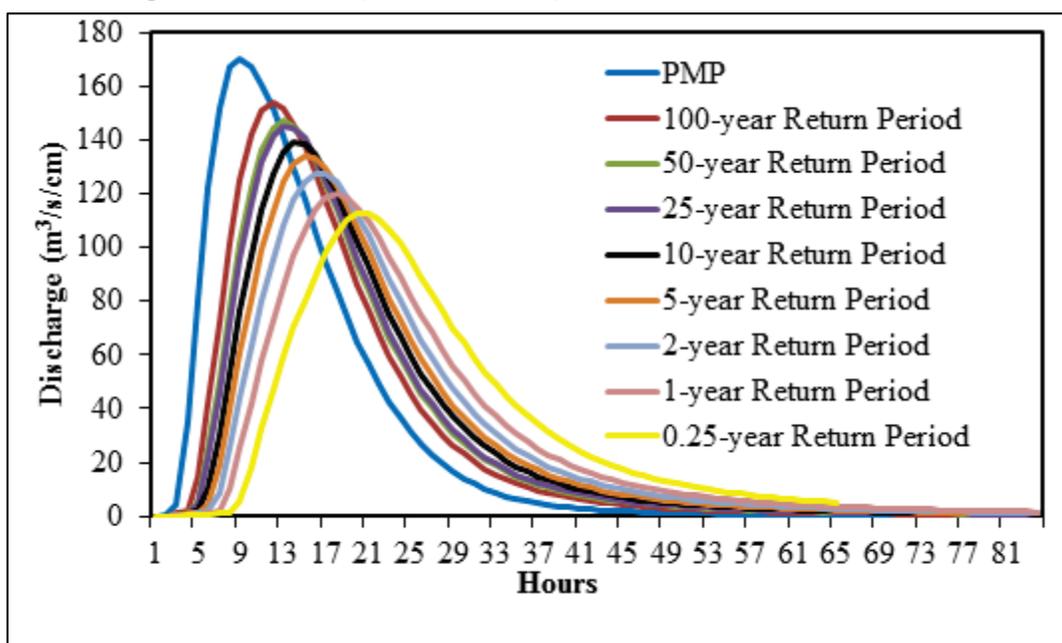
Table 3 shows the derivation of the peaking factor in terms of 0.25-year return period to 100-year return period. In Table 3, the values under the headings "Peaking Factor in terms of," "X-year Return Period," are the peaking factors representing the X-year return period, where X ranges from 0.25 to 100. The X-year return period peaking factors in the rows are the UH peak values of the corresponding event types under heading "Event type" normalized by UH peak value for that X-year return period. UH peak values are defined under heading "Unit hydrograph peak." For example, if the X is 0.25, all the UH peak values, from 0.25-year return period to PMP, are divided by the UH peak value of 0.25-year return period that is 112.6 centimeters/second/centimeter (cm/s/cm) in Table 3. Therefore, Table 3 represents a sensitivity analysis of the peaking factor to the return period used to normalize the peak of the UHs. Figure 9 was developed to illustrate Table 3, where four bands are defined based on the peaking factor value: "Safe" if the peaking factor is less than 1.0; "Moderately Safe" if between 1 and 1.3; "Critical" if between 1.3 and 1.5; and "Risky" if exceeding 1.5. Figure 9 shows that the UHs developed from events at or above the 5-year return period are well within the 1.25 to 1.5

peaking factor range. The UHs developed from 5-year return period and above fall into either the Safe or Moderately Safe band in Figure 9. Table 3 and Figure 9 also show that the peaking factor from the 1-year return period rainfall UH is close to 1.5 at the PMP. In other words, the 1-year return period UH may fall into the Critical band, as shown in Figure 9. Table 3 and Figure 9 show that peaking factor from UH of the 0.25-year event is greater than 1.5 at PMP. That is, a 0.25-year return period UH may fall into the "Unsafe" band, as shown in Figure 9.

Table 3. Derivation of the peaking factor in terms of GSSHA-simulated NOAA 50th percentile 0.25-year return period to 100-year return period in the Goose Creek Catchment.

Event Type	UH Peak (m³/s/cm)	Peaking Factor in Terms of						
		0.25-year Return Period	1-year Return Period	5-year Return Period	10-year Return Period	25-year Return Period	50-year Return Period	100-year Return Period
0.25-year Return Period	112.600	1.000	0.939	0.840	0.811	0.777	0.767	0.732
1-year Return Period	119.900	1.065	1.000	0.895	0.863	0.827	0.816	0.780
5-year Return Period	134.000	1.190	1.118	1.000	0.965	0.924	0.912	0.871
10-year Return Period	138.900	1.234	1.158	1.037	1.000	0.958	0.946	0.903
25-year Return Period	145.000	1.288	1.209	1.082	1.044	1.000	0.987	0.943
50-year Return Period	146.900	1.305	1.225	1.096	1.058	1.013	1.000	0.955
100-year Return Period	153.800	1.366	1.283	1.148	1.107	1.061	1.047	1.000
PMP	170.400	1.513	1.421	1.272	1.227	1.175	1.160	1.108

Figure 9. Peaking factor sensitivity analysis chart.
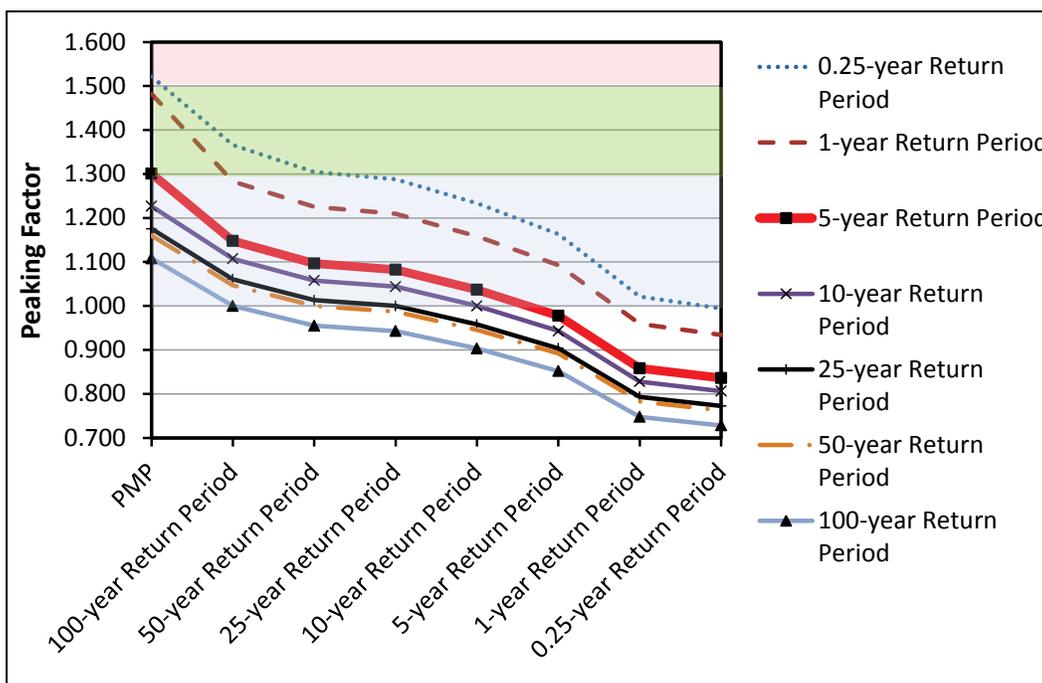
This case study in the Goose Creek watershed found that the upper limit of the UHPF for the UH peak published in the USACE NAB (1963) report was 0.9 and that for the UH in Figure 2 was 1.4. The average rainfall published in the USACE NAB (1963) report was 135 millimeters (mm) in 36 hr. This rainfall is equivalent to 25-year return period rainfall as per the PFDS report. The peak of the derived 6-hour UH for the USACE NAB (1963) published report was 189 cubic meters/second/centimeter (m³/s/cm). The rainfall for Figure 2 was 42 mm in 12 hr, which is equivalent to 1-year return period rainfall as per the PFDS report.

Figure 10 shows the peaking factor and effective rainfall relationship as defined by Equation (5).

$$y = 2.486x^{-0.179} \qquad (5)$$

The relationship defined with Equation (5) had the coefficient of determination of 0.9406.

Figure 10. Peaking factor and effective rainfall relationship.

$$y = 2.486x^{-0.179}$$
$$R^2 = 0.9406$$
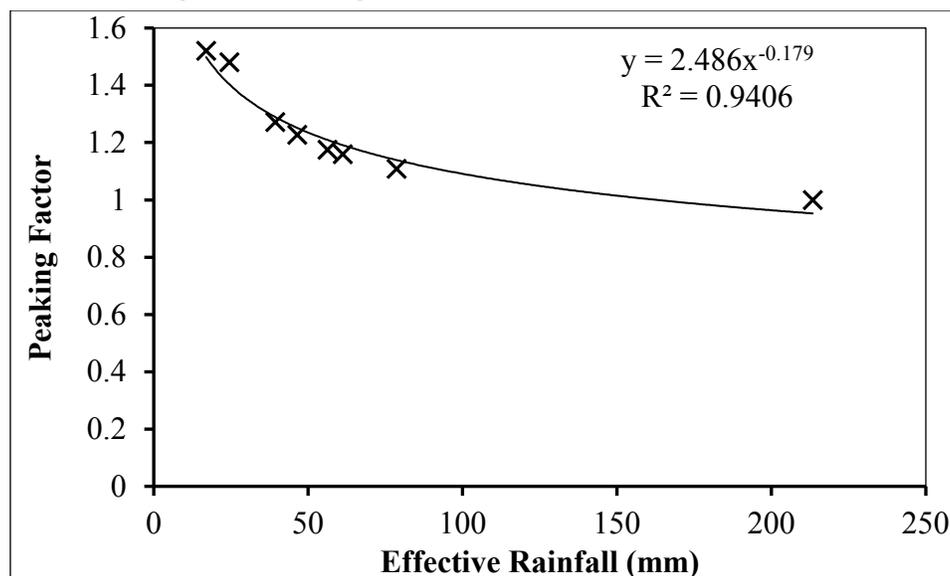
The use of a single, deterministic reference hydrograph to establish an acceptable peaking factor implies that the full flood and precipitation frequency curves are known with certainty. As complete knowledge is rarely available; the use of probabilistic techniques can be used to capture the various sources of uncertainty to establish a peaking factor confidence interval. The range and maximum value of the interval can then be incorporated as part of a dam safety assessment as a measure of risk. Uncertainty can be introduced from both the input data as well as the data analysis procedure. The present analysis is confined to the latter introduced by the NOAA precipitation analysis procedure; the uncertainty associated with input data and other processes can be readily incorporated through a joint probability analysis.

NOAA Atlas 14 provides three precipitation depths occurring at the 5th, 50th, and 95th confidence intervals that equally satisfy the return period as a result of its Monte Carlo data analysis procedure. Selection of any depth that satisfies the confidence intervals would be an acceptable event to represent the return period of interest. The confidence interval range that is produced may be quite large in many cases. The range in the calculated precipitation depth data analysis confidence intervals is a proxy for the amount of risk accepted by an installation if data processing uncertainty is not considered when calculating the peaking factor. If a precipitation depth and resulting flood at a low confidence interval is utilized as the design event, the peaking factor may be underestimated, resulting in an unsafe design. Similarly, if a precipitation depth and flood

at a high confidence interval is utilized as the norming event, a design may be overly conservative leading to increased construction/maintenance costs.

The effect of data processing uncertainty on peaking factors at Goose Creek was determined by applying the 5% and 95% confidence interval precipitation depths to the watershed in addition to the 50% value. Figure 11 shows the variation in the peaking factor at each return period, normalized to the 5-year, 50th percentile value. The 50th percentile is given by the black line; the 95th percentile is given by the upper error bar and the 5th by the lower error bar. The difference between the 5th and 95th percentile is as large as 0.052 for the 50-year return period, 4.7% of the 50th percentile value. While not evident directly at the Goose Creek watershed, this difference in the selected precipitation depth due to data processing uncertainty alone is sufficiently large that it could cause some reference events to become inadequate at maintaining a peaking factor below 1.5 at other watersheds. The large difference between the percentiles supports greater conservativeness in choosing the precipitation depth confidence level used to establish peaking factors.

Figure 11. Peaking factor as a function of return period and data processing uncertainty interval for the precipitation depths specified by NOAA Atlas 14 that were applied to the hydrologic model. The black line gives the 50th percentile peaking factor with lower and upper bars giving the 5th and 95th percentiles, respectively.
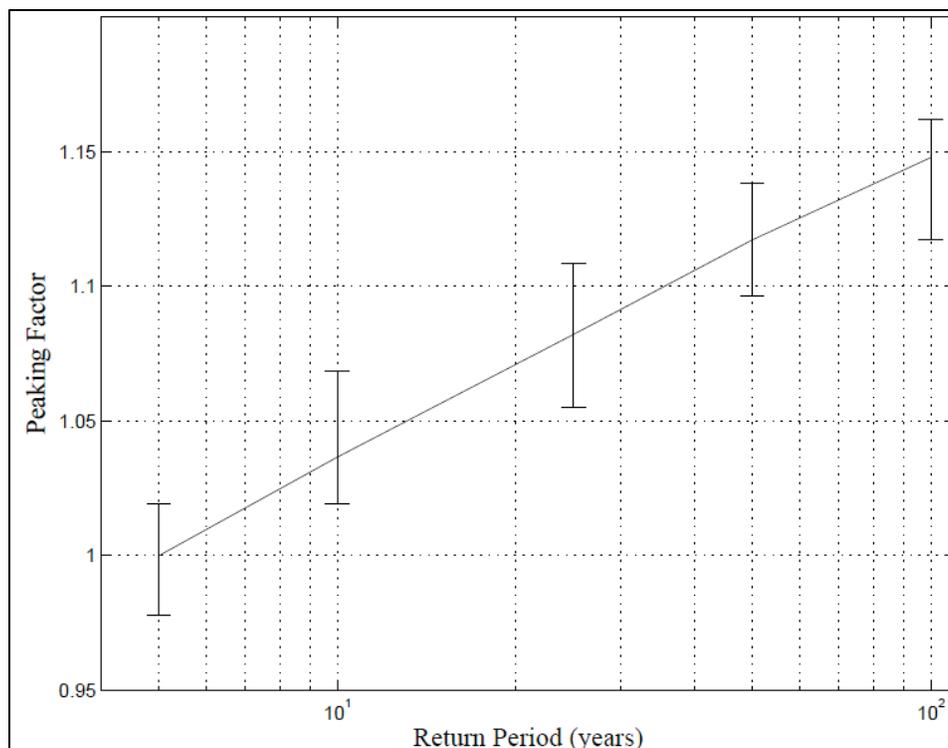
Knowledge of the peaking factor as a function of return period permits inverse calculation for the return period associated with a desired peaking factor magnitude. This knowledge is again useful as a proxy for the amount of risk accepted during a peaking factor analysis. As excess precipitation is linear with increasing precipitation depth, a simple logarithmic transform is typically sufficient to extrapolate the peaking factor beyond modeled return periods. By taking the normalization events at increasing confidence intervals to obtain the peaking factor trend and extrapolating the return period at the desired peaking factor, a range of return periods can be established from lowest (safest) to highest (least safe). The span of the size of the return period range informs one as to how sensitive the analysis is to the normalization value. Equally the lowest range value can determine if a design is sufficiently conservative to meet the return period requirements at the maximum accepted peaking factor.

The procedure above can be applied to determine a return period associated with the PMP as well as the commonly utilized maximum peaking factor value of 1.5. Although by definition the PMP does not have a return period, it is nonetheless a useful exercise. The intersection of the peaking factor trend line with the peaking factor evaluated at the PMP represents the location where the curve in Figure 10 becomes asymptotic. The magnitude of the return period at the intersection characterizes the maximum protection of the system under the assumptions used to formulate the peaking factor analysis. Table 4 gives the expression for the trend line at each confidence percentile in Figure 11 as well as the coefficient of determination. The fit to each trend line, and in particular the 50th percentile, correlates highly to the calculated peaking factors.

Table 4. Equations of fit for the peaking factor analysis in Figure 11 for the considered data analysis confidence intervals.

Percentile	Function	R^2
5th	$PF(y) = 0.0468 \ln(y) + 0.9066$	0.9909
50th	$PF(y) = 0.0495 \ln(y) + 0.9993$	0.9993
95th	$PF(y) = 0.0467 \ln(y) + 0.9532$	0.9837

The return period for the PMP peaking factor and peaking factor of 1.5 are approximated by inverting each function in Table 4 to solve for the return period as a function of peaking factor. The results for each percentile are given in Table 5. A lower uncertainty percentile results in a larger return period for the PMP and peaking factor of 1.5. This follows intuition as a

longer return period is required for smaller magnitude events to reach the same peaking factor. The 50th and 95th percentiles show reversed behavior for the maximum peaking factor. This inversion is a result of the variability in the Monte Carlo procedure used by NOAA to establish the confidence intervals. Their difference, however, is not as great as the difference between either and the 5th percentile. The 5th percentile predicts almost a three times greater return period at the maximum accepted peaking factor than the 50th and 95th percentiles. A similar magnitude difference is also present between the 5th and 95th percentiles of the PMP return period. The range in return period at the PMP peaking factor is approximately 1500 years while that of the 1.5 peaking factor is nearly 200,000 years. The extent of both ranges demonstrates the importance of accounting for uncertainty when conducting a peaking factor analysis to ensure a conservative design.

Table 5. Return period for the PMP and maximum accepted peaking factor as calculated by inverting the functions fit the peaking factor/return period relationship. The form of each function is given in Table 4.

Percentile	PMP Return Period (years)	Maximum Peaking Factor (1.5) Return Period (years)
5th	2441	321093
50th	1173	118280
95th	915	121635

5 Conclusions

This research utilized the GSSHA model in the Goose Creek watershed to calculate the discharge from design events with various magnitudes of excess rainfall while including the non-linear component of the watershed response. The discharges were then used to develop the UHPF relationship with rainfall excess. The UH results obtained at various return periods demonstrate that the peaking factor from a 5-year return period was well within 1.25 to 1.5 limits. In contrast, the UH derived from events less than 5-year return period exceeded the 1.25 to 1.5 peaking factor limits. Goose Creek unit hydrograph was designed from rainfall event equivalent to 25-year return period rainfall and therefore ensures dam safety given the current UHPF guidance. This study also developed peaking factors' relationship with excess rainfall in the Goose Creek watershed. This peaking factors' relationship with excess rainfall was found to be non-linear.

In the present case study, the land use change effects are not taken into consideration as the urbanization and development within the study watershed is not significant as the impervious area is less than 3% of the catchment area. Yet, there are other watersheds where the land use change is significant (e.g., Ogden et al. 2011), which requires an analysis of the land use change effect in the hydrologic numerical simulations for peaking factor analysis. A further study would be to develop a regionalized peaking factor relationship with excess rainfall based on the climatic regions of continental United States. That way, the transposition of UHPF and peaking factors' relationship with excess rainfall between watersheds with similar climates can be readily investigated. A regionalized watershed characteristics relationship also provides guidance and quick information for prediction, flood hazard assessment, or design purposes on ungauged basins (Ding 2011; Pradhan et al. 2016). In this study, the assumption has been made that the data processing confidence interval can be taken as an initial proxy of the total precipitation depth uncertainty. Imperfect knowledge of the true precipitation depth at each return period must also be accounted for to ensure the reference event has a sufficiently large magnitude to maintain dam safety. Ideally, such an analysis would consider all sources of uncertainty from the watershed such as antecedent conditions, storm spatial/temporal properties, and data processing decisions, which is left as a topic for further research.

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REPORT DOCUMENTATION PAGE

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1. REPORT DATE	2. REPORT TYPE	3. DATES COVERED *(From - To)*
May 2017	Final Report	

4. TITLE AND SUBTITLE	5a. CONTRACT NUMBER
Unit Hydrograph Peaking Analysis for Goose Creek Watershed in Virginia: A Case Study	5b. GRANT NUMBER
	5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S) 7.	5d. PROJECT NUMBER
Nawa Raj Pradhan and Drew Loney	52L3J9
	5e. TASK NUMBER
	5f. WORK UNIT NUMBER

8. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)	8. PERFORMING ORGANIZATION REPORT NUMBER
U.S. Army Engineer Research and Development Center Coastal and Hydraulics Laboratory 3909 Halls Ferry Road Vicksburg, MS 39180-6199	ERDC/CHL TR-17-6

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)	10. SPONSOR/MONITOR'S ACRONYM(S)
U.S. Army Corps of Engineers Washington, DC 20314-1000	HQUSACE
	11. SPONSOR/MONITOR'S REPORT NUMBER(S)

12. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

13. SUPPLEMENTARY NOTES

14. ABSTRACT

Existing U.S. Army Corps of Engineers (USACE) policy suggests unit hydrograph peaking factor (UHPF), the ratio of an observed event unit hydrograph peak to the modeled one for a catchment, ranges between 1.25 and 1.50 to ensure dam safety. It is pertinent to investigate the impact of extreme flood events on the validity of this range through physically based rainfall-runoff models not available during the planning and design of most USACE dams. The UHPF range was analyzed by deploying the Gridded Surface Subsurface Hydrologic Analysis model in the Goose Creek, VA, watershed to develop a peaking factor relationship with excess rainfall for National Oceanic and Atmospheric Administration rainfall depths across various return-period events as well as the National Weather Service probable maximum precipitation event. The effect of statistical uncertainty on UHPF at Goose Creek was determined by applying the 5% and 95% confidence interval precipitation depths to the watershed in addition to the 50% value. This study concluded that a design event with a return period greater than 5-year is required for the unit hydrograph to ensure dam safety. Goose Creek unit hydrograph was designed from 25-year return period rainfall and therefore ensures dam safety given the current UHPF guidance.

15. SUBJECT TERMS
Floods, Flood control, Hydrologic models, Precipitation (Meteorology), Runoff, Unit Hydrograph, Watersheds--Virginia

16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON
a. REPORT	b. ABSTRACT	c. THIS PAGE			Nawa Pradhan
Unclassified	Unclassified	Unclassified	SAR	32	19b. TELEPHONE NUMBER *(Include area code)* 601-634-3837