



Estimates of Precipitation IDF Curves and Design Discharges for Road-Crossing Drainage Structures: Case Study in Four Small Forested Watersheds in the Southeastern US

D. M. Amatya¹; S. Tian²; D. A. Marion³; P. Caldwell⁴; S. Laseter⁵; M. A. Youssef⁶; J. M. Grace⁷; G. M. Chescheir⁸; S. Panda⁹; Y. Ouyang¹⁰; G. Sun¹¹; and J. M. Vose¹²

Abstract: We compared precipitation intensity-duration-frequency (PIDF) curves developed for four small forested watersheds to spatially interpolated estimates from the National Oceanic and Atmospheric Administration's (NOAA) Atlas-14. We also evaluated the Rational Method (RM) using on-site PIDFs and USGS Regional Regression Equations by comparing their estimated design discharges with a given exceedance probability p (Q_p) to values computed from on-site data fitted to the Log-Pearson (LPIII) distribution. Overall, NOAA's PIDF estimates were not substantially different from the on-site PIDFs. The 25-year and larger Q_p by the RM were in closer alignment with LPIII estimates in the smaller watersheds, whereas Q_p by the USGS were a better fit for the larger ones in most cases. Adapting return period-dependent runoff coefficient improved estimates by the RM in the large lowland watershed, but not in the other smaller high-relief watersheds. We recommend RM with 1-h duration NOAA-PIDF for designing road drainage structures in small and possibly the USGS method for large forested watersheds. However, future studies should focus on validation in watersheds of different sizes and topography. DOI: [10.1061/\(ASCE\)HE.1943-5584.0002052](https://doi.org/10.1061/(ASCE)HE.1943-5584.0002052). © 2021 American Society of Civil Engineers.

Author keywords: Extreme rain events; Flood frequency; Precipitation intensity-duration-frequency (PIDF); NOAA Atlas 14; Rational method (RM); USGS regional regression.

Introduction

The US Department of Agriculture Forest Service (USDAFS) manages a road system consisting of approximately 600,000 km of roads and at least 40,000 stream crossings (Heredia et al. 2016), most of which are located in forested headwater watersheds with small drainage areas. Accurate estimation of peak storm discharge

rates from watersheds is important for the design of drainage works along roadways and related infrastructure (Manning and Kilariski 1998). Many culverts in the US are currently considered undersized for accommodating bankfull flow conditions for 1- to 2-year flooding, one of the important design considerations to be met (USDOT 2018). Improperly sized stream-crossing infrastructure (i.e., road leadoff structures, fords, culverts, and bridges) could

¹Research Hydrologist, USDA Forest Service, Center for Forested Wetlands Research, 3734 Highway 402, Cordesville, SC 29434 (corresponding author). ORCID: <https://orcid.org/0000-0003-2641-9267>. Email: devendra.m.amatya@usda.gov

²Formerly, Research Associate, Dept. of Biological and Agricultural Engineering, North Carolina State Univ., Weavers Lab, P.O. Box 7625, Raleigh, NC 27695. Email: stian.tsy@gmail.com

³Formerly, Research Hydrologist, USDA Forest Service, Center for Bottomland Hardwoods Research, P.O. Box 1270, Hot Springs, AR 71902. Email: SFoxOrder@gmail.com

⁴Research Hydrologist, USDA Forest Service, Coweeta Hydrologic Laboratory, 3160 Coweeta Lab Rd., Otto, NC 28763. Email: peter.v.caldwell@usda.gov

⁵Biological Scientist, USDA Forest Service, Center for Integrated Forest Science, 90 Sloan Rd., Franklin, NC 28734. ORCID: <https://orcid.org/0000-0001-7102-6230>. Email: stephanie.laseter@usda.gov

⁶Professor, Dept. of Biological and Agricultural Engineering, Weavers Lab, P.O. Box 7625, North Carolina State Univ., Raleigh, NC 27695. Email: mayousse@ncsu.edu

Note. This manuscript was submitted on December 23, 2019; approved on October 23, 2020; published online on January 30, 2021. Discussion period open until June 30, 2021; separate discussions must be submitted for individual papers. This paper is part of the *Journal of Hydrologic Engineering*, © ASCE, ISSN 1084-0699.

⁷General Engineer, USDA Forest Service, Center for Forest Watershed Research, Florida A&M Univ., 1740 S. Martin Luther King Jr. Blvd., Perry-Paige Bldg., Suite 303 North, Tallahassee, FL 32307. ORCID: <https://orcid.org/0000-0003-4900-2982>. Email: johnny.m.grace@usda.gov

⁸Research Associate Professor, Dept. of Biological and Agricultural Engineering, Weavers Lab, North Carolina State Univ., P.O. Box 7625, Raleigh, NC 27695. Email: gchescheir@gmail.com

⁹Professor, Dept. of GIS and Geospatial Technology, Univ. of North Georgia, 3820 Mundy Mill Rd., Oakwood, GA 30566. Email: sudhanshu.panda@ung.edu

¹⁰Research Hydrologist, USDA Forest Service, Center for Bottomland Hardwoods Research, 775 Stone Blvd., Thompson Hall, Room 309, Mississippi State, MS 39762. Email: ying.ouyang@usda.gov

¹¹Research Hydrologist, USDA Forest Service Eastern Forest Environmental Threat Assessment Center, 3041 East Cornwallis Rd., Research Triangle Park, NC 27709. ORCID: <https://orcid.org/0000-0002-0159-1370>. Email: ge.sun@usda.gov

¹²Formerly, Senior Research Ecologist, USDA Forest Service, Southern Research Station, Forestry Sciences Laboratory, Center for Integrated Forest Science, P.O. Box 12254, Research Triangle Park, NC 27709. Email: james.m.vose@usda.gov

result in structural failures, and, subsequently, increased flooding, soil erosion, economic losses, and disruption of stream connectivity critical for aquatic organisms (Heredia et al. 2016). Consequently, engineers seek to design hydraulic structures that have a high probability of accommodating extreme flow events that may occur during their useful lives (Novak et al. 2007).

Various standards and methods have been established for hydraulic design of cross drainage structures (Donahue and Howard 1987; Heredia et al. 2016; USDOT 2002). Generally, the hydraulic design of such structures starts by determining the peak discharge (Q_p) exceedance from a specific design flood frequency (FFA) (e.g., 25, 50, or 100 years) (ODOT 2014; USDOT 2012) based on the purpose and life span of the structure. The Q_p can be influenced by several factors including watershed area, antecedent moisture, high flow duration, channel hydraulics, soil, land cover, and management activities. The selection of the design Q_p is based on acceptable risks of failure and cost-benefit analyses because designing for the maximum Q_p of watersheds is seldom economically feasible (USDOT 2018). For example, culverts on rural roads are generally designed for 25–50-year return intervals, depending upon the Federal Highway Administration (FHWA) requirements (USDOT 2002), floodplain management guidelines of Federal Emergency Management Administration, and ordinances of state and local governments (NCDOT 2016; SCDOT 2009; Weaver and Hagans 1994).

Donahue and Howard (1987) states that the greatest source of error in culvert design is in design peak flow analysis. Estimating Q_p when using FFA analysis is particularly challenging in forested headwater watersheds because these watersheds are often ungauged or have limited streamflow records. Challenges exist even when watersheds are gauged (Rogger et al. 2012) because fitting data and extrapolating based on recommended theoretical probability distributions, such as the Log-Pearson Type III (LPIII) (England et al. 2018), is not a straightforward task. Recommendations to combine regional skewness coefficient values from pooled groups of stations with values computed from short on-site records further complicate on-site FFA analyses even when trends are not present in the flood series (England et al. 2018; USGS 1982).

A few empirical methods have been developed to estimate flood magnitudes for various return intervals at sites without sufficient historical discharge data. The USGS publishes Regional Regression Equations (RREs) on a roughly decadal basis, which are fit by performing FFA on a group of stations within an area of similar hydrologic characteristics (England et al. 2018). In addition to the USGS RRE (Feaster et al. 2014; Wagner et al. 2016), the Rational Method (RM) (Kuichling 1889) and the Soil Conservation Service–Curve Number (CN) method (NRCS 2004) have also been widely used to estimate Q_p for design storms (Genereux 2003). These empirical methods use runoff coefficients, CN, and time of concentration parameters to lump the effects of key watershed characteristics (e.g., size, shape, topography, soils, and land use) on runoff generation (USDOT 2002) in response to antecedent moisture (AMC) and climatic conditions [typically represented by precipitation intensity-duration-frequency (PIDF) curves] (US Water Resources Council 1981; Rogger et al. 2012).

The PIDF is fundamental for estimating design Q_p in ungauged watersheds (Eisenbies et al. 2007) but introduces additional uncertainty. NOAA's Atlas-14 based gridded PIDF values (Bonnin et al. 2006; Perica et al. 2013) are commonly used for design applications to determine the PIDF and associated confidence limits. However, the NOAA Atlas-14 sources are based on regional frequency analysis of precipitation recorded at various locations and may not be applicable to small catchments such as those in USDAFS National Forests and long-term Experimental Forests and Ranges (EFRs)

across the conterminous US (Amatya et al. 2016; Laseter et al. 2012; Walega et al. 2020), especially in high altitude mountainous areas with complex terrain and sparse coverage.

Several studies have compared on-site PIDF estimates for urban areas and other physiographic settings to regional PIDF estimates and generally concluded that regional frequency analysis is more efficient and accurate (Hosking and Wallis 2005). However, studies deriving PIDF curves using on-site long-term record from EFRs and comparing results with the Atlas-14 regional estimates are lacking. These comparisons could improve PIDF estimates by identifying magnitudes and sources of differences and increases the likelihood of choosing appropriate PIDFs for culvert design discharge calculations in small, ungauged forested watersheds.

Studies have demonstrated large differences in estimated Q_p for small mixed use, and rural watersheds when using empirical methods (Genereux 2003; Grimaldi and Petroselli 2015; Trommer et al. 1996; Thompson 2006). However, similar comparisons for small forested headwater watersheds are lacking. Comparison of design discharge methods in small forested catchments could provide engineers and land managers with an assessment of the relative performance of the various methods for a given design application, thereby preventing potential costly over or underdesign of road drainage structures (Genereux 2003) and potential ecological impacts to critical forested headwater streams (Richardson and Danehy 2007).

The present study focused on four watersheds in three USDAFS EFR sites in the southeastern US that have longer term (~40 year) records of hourly precipitation and streamflow records (>30-year). Our objectives were to: (1) identify differences between on-site and NOAA regional PIDF estimates and (2) evaluate the performance of the RM (using the on-site PIDF estimates) and USGS RRE by comparing their predicted design discharges to those estimated by the on-site LPIII-based FFA (Fig. 1). Findings from this study are intended to guide forest managers in choosing reliable PIDF data and design discharge estimation methods for cross-drainage infrastructure.

Materials and Methods

Study Sites

We used historic data from three EFRs in the southeastern US, representing high-relief mountains in North Carolina (NC), low-relief mountains in Arkansas (AR), and low-relief coastal plain in South Carolina (SC), respectively (Fig. 2 and Table 1).

Coweeta Watersheds, North Carolina

The study watersheds (35°03' N, 83°25' W) are located at the Coweeta Hydrologic Laboratory (CHL) in the Appalachian Mountain Range within the Blue Ridge Physiographic Province (Fig. 2). The Coweeta Basin (1,626 ha) has a humid temperate climate with a long-term average (LTA) annual precipitation ranging from 1,794 mm at lower elevations to 2,368 mm at high elevations (Laseter et al. 2012). Soils are moderately to well-drained on moderate to steep slopes. The two reference watersheds (low elevation WS14 and high elevation WS27) were selected due to the large elevation-induced climate gradient between the two locations. A detailed site description can be found elsewhere (Caldwell et al. 2016; Laseter et al. 2012).

Alum Creek Watersheds, Arkansas

The study watershed (AC04) is located within the 1,885 ha Alum Creek Experimental Forest (34°48' N, 93°3' W) within the upper Lake Winona basin of the Ouachita National Forest (Fig. 2).

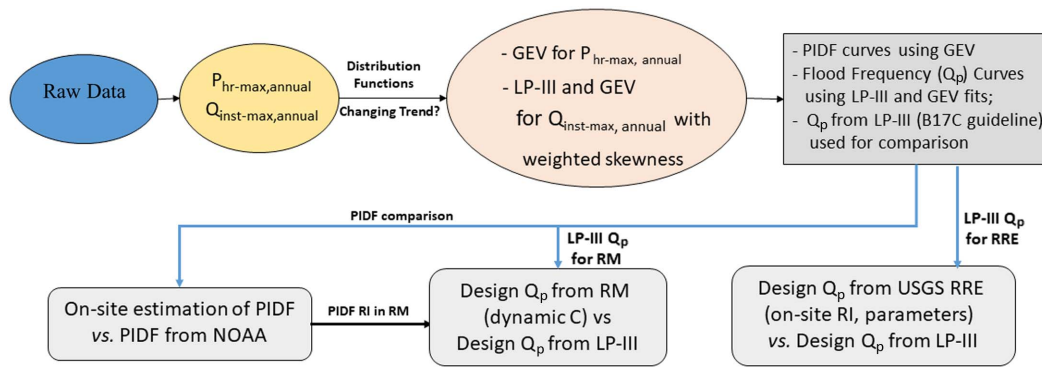


Fig. 1. Model chain diagram for (a) deriving on-site PIDF estimates for comparison with the NOAA PIDFs; and (b) to estimate design discharges using the RM with the on-site PIDF and the USGS RREs for comparison with on-site based design peak discharges using the LP-III distribution for four study watersheds.

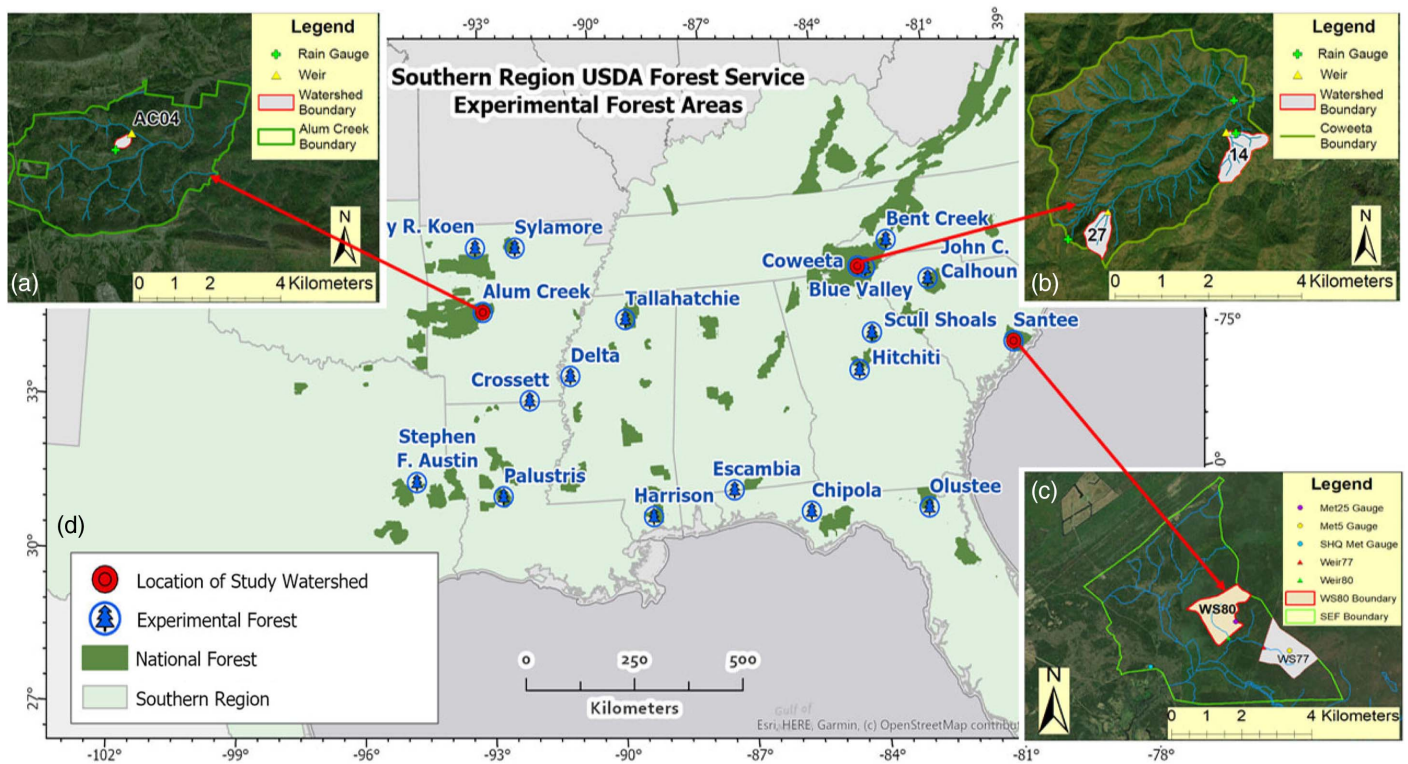


Fig. 2. Locations of the study watersheds: (a) Alum Creek, Arkansas; (b) Coweeta, North Carolina; (c) Santee, South Carolina; and (d) regional map. [Sources: (a) Esri, HERE, Garmin, USDA FSA, Geo Eye, Maxar, © OpenStreetMap contributors and the GIS User Community; (b) Esri, HERE, Garmin, NC CGIA, Earthstar Geographics, © OpenStreetMap contributors and the GIS User Community; (c) Esri, HERE, Garmin, USDA FSA, Geo Eye, Maxar © OpenStreetMap contributors and the GIS User Community; and (d) Esri, HERE, Garmin, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS user community.]

The AC04 watershed is characterized by a humid subtropical climate with LTA annual precipitation of 1,321 mm. Soils are generally less than one meter deep, with high infiltration rates. A detailed description of the watershed is provided by Adams and Loughry (2008).

Santee Watershed, South Carolina

The study watershed is located within the Santee Experimental Forest (SEF) (33.15° N, 79.8° W) and is a part of the Francis Marion National Forest (Fig. 2). The SEF has a humid subtropical climate and a LTA annual precipitation of 1,370 mm. Soils are moderately

to poorly-drained with low permeability and high available water capacity (Harder et al. 2007). The selected watershed, WS80, is 160 ha in size and has been relatively undisturbed as a reference watershed for over 80 years. A detailed site description can be found elsewhere (Amatya et al. 2019; Harder et al. 2007).

Data Compilation

Long-term on-site precipitation (~40 years) and discharge (>30 years) data were compiled for the four experimental watersheds (Table 1). None of the watersheds has been impacted by

Table 1. General characteristics of the four study watersheds

Characteristic	Watersheds			
	WS27, North Carolina	WS14, North Carolina	AC04, Arkansas	WS80, South Carolina
Area (ha)	39.05	61.03	13	160 ^a
Mean elevation (m)	1,254	878	~300	~6
Mean slope (%)	57	50	~15	<3
Drainage density (m ⁻¹)	0.003447	0.004723	0.006179	0.00085
Time of concentration ^b (T _c , h)	0.34	0.4	0.4	2.1
Runoff coefficient ^c (C)	0.33	0.09	0.59	0.11
Vegetation	Mixed hardwoods	Mixed hardwoods	Pine-hardwood	Pine-hardwood
Mean annual <i>P</i> (mm/year)	2,316	1,842	1,321	1,370
Flow records used in this study	1976–2015	1977–2015	1964–1993/1999–2014 ^a	1969–1981, 1990–1994, 2004–2016
Precipitation records used in this study ^d	1976–2015	1976–2015	1976–2016	1977–2016

^aThe area of WS80, South Carolina, that is considered in the analysis was 2.06 km² (206 ha) before 2001. Thereafter, the installation of a culvert reduced the area to 1.60 km² (160 ha) (Harder et al. 2007).

^bThe time of concentration was calculated using the Kerby-Ramser method, with the Kerby method for overland flow part and the NRCS (2004) method for channel part.

^cRunoff coefficients (C) were obtained as a mean from calibration using the RM with storm event data that produced top 10 peak discharge rates, corresponding peak rainfall intensities, and drainage area for each of the four study watersheds. Obtained C values are comparable to published studies for North Carolina sites (Scaife and Band 2017; Swank and Crossley 1988) and WS80, South Carolina (Amatya and Radecki-Pawlik 2007; Epps et al. 2012) reported in detail in Table SI-1.

^dFlow data for 2003–2014 at AC04 watershed was obtained by regression estimated with adjacent watershed AC11.

anthropogenic flow alterations (e.g., diversions) or urban land uses upstream. Therefore, observed flows did not need adjustment to account for upstream anthropogenic effects. Continuous digital records of precipitation were available starting in 2003 for WS80, 1997 for AC04, and 2004 for the Coweeta watersheds WS14 and WS27. Similarly, digital measurements of stage for estimating streamflow at watershed outlets were available starting from 1990 for WS80, from 1994 at Coweeta WS14 and WS27, and none at AC04. Streamflow rates were estimated using measured stage data with established rating curves based on a standard V-notch weir equation for WS14, a compound V-notch weir equation for WS27 and WS80 (Harder et al. 2007; Reinhart and Pierce 1964), and a 1.35-m H-flume for AC04 (Marion 2004).

Subdaily precipitation and instantaneous Q_p records for the complete period of the analysis were necessary to estimate the PIDF and FFA, respectively. At each location precipitation volume and intensity were recorded by a recording rain gage (Belfort Universal recording rain gage, Belfort Instrument, Baltimore, Maryland) and were verified using a colocated 200-mm (8-in) diameter Standard National Weather Service rain gage. Only three rain gauges (RRG31 adjacent to WS27, RRG41 adjacent to WS14, and RRG06) at Coweeta, North Carolina (Table 1) had complete subdaily precipitation data. The subdaily analog data for WS80 and AC04 were digitized using DataThief version 1.7 software (Tummers 2006). Detailed procedures of using DataThief are provided in the Supplemental Materials (SI-1).

Missing subdaily precipitation data for WS80 were gap-filled using data directly from a nearby Lotti gauge located 1.73 km southeast of and at the same elevation as WS80 from 1977 to 1994 and a nearby NOAA station at Charleston airport from 1995 to 2002. The coefficients of determination (R^2) for precipitation between the WS80 and each of the Lotti gauge and the NOAA station were 0.89 and 0.72, respectively.

For the AC04 watershed, missing discharge measurements during the 1999–2014 period were filled using regression relationships developed with data from AC11, a nearby undisturbed and hydrologically similar watershed (Table 1) (Marion 2004; Walega et al. 2020).

NOAA Atlas-14 estimated PIDFs for each site were downloaded from the Precipitation Frequency Data Server (Perica et al. 2013).

Estimation of the Precipitation Intensity-Duration-Frequency (PIDF)

A time series of annual maximum precipitation was constructed for each station by taking the highest accumulated precipitation for a particular duration (1, 2, 3, 6, 12, and 24 h) in each calendar year (Fig. S1). We used the Generalized Extreme Value (GEV) distribution (Supplemental Materials SI-2) for estimating PIDF because this is commonly accepted in hydrology for modeling extreme events (Bonnin et al. 2006) given a large set of sufficiently independent, identically distributed random variables (e.g., annual peak values of precipitation and/or river discharges) from different periods. The PIDFs for WS80 and AC04 were developed using precipitation data from a single gauge per watershed. For the Coweeta North Carolina site, the PIDF was derived using data from three gauges: one at each of the WS14 and WS27 watersheds and a third gauge at the primary Coweeta weather station included in the NOAA gauge network, as described in the Supplemental Materials. We assumed the derivations of PIDF using 40-years of single gauge data for small headwater watersheds (≤ 160 ha) were adequate for designing forest road culverts, where a 50-year return period design discharge is generally used.

We used the Extreme Value Analysis package in R-software version 3.3.2 for the GEV distribution analysis (Gilleland and Katz 2016). The distribution parameters and quantiles of the posterior probability distribution were estimated using the Bayesian method (Gilleland and Katz 2016). Findings from previous studies (Cheng and AghaKouchak 2014) suggested computing the location parameter of GEV distribution as a function of time under the non-stationary assumption, if a significant temporal change trend exists for the data series (SI-3). Thus, we first conducted a trend analysis based on the Mann-Kendall (M-K) test [Eq. (S2)] for annual peak rainfall intensity (RI). Although the 40-year precipitation record may be somewhat short to consider trends in the scale parameter, this approach is consistent with Wang et al. (2020), who cautioned

that the power of the M-K test increases with sample record and statistical power may be inflated, particularly for short, highly variable samples.

Results of this trend analysis, however, showed no significant ($p > 0.05$) temporal trends for the annual peak RI (Fig. S1). Therefore, we applied the standard GEV distribution in Eq. (S1) (SI-2) without using the time-dependent function.

Design Discharge Estimation

Design peak discharge rates were determined using onsite long-term annual maximum Q_p data, the RM (Kuichling 1889), and the USGS RRE methods (Feaster et al. 2014; Wagner et al. 2016; Weaver et al. 2009) (Fig. 1 and Table 2).

Design Discharge Estimation Using Observed Long-Term Data

Design discharge rates were determined for the four watersheds by fitting historical records of annual maximum Q_p to the LPIII (England et al. 2018) and GEV distributions. The FFA analysis based on the LPIII and GEV distributions was conducted using the Statistical Software Package (HEC-SSP version 2.1) and R-software, respectively. Design discharges (median) and confidence intervals for 2, 5, 10, 25, 50, and 100-year return periods were estimated by incorporating a weighted skew factor obtained from the on-site and regional skews in accordance with Bulletin 17C guidelines (England et al. 2018).

Design Discharge Estimation Using the Rational Method (RM)

The RM (Kuichling 1889) has been widely used for estimating peak runoff rates in engineering designs for small watersheds (<200 acre or 80 ha) in both urban and rural settings (Thompson 2006) (Table 2) but has also been applied in watersheds of up to 260 ha (ODOT 2014; Weaver and Hagans 1994). The principal assumptions of this method are: (1) rainfall intensity is uniform throughout the storm duration, (2) the rainfall distribution is uniform across the drainage area, (3) frequency of the peak discharge occurrence is the same as the frequency of the precipitation producing that event, and (4) the drainage basin has no significant storage. While precipitation is the dominant factor driving events Q_p in small watersheds, AMC is also a key factor affecting rainfall-runoff relationships in less developed watersheds with limited impervious surfaces, especially for events with a return period of 10 years or fewer. In addition, annual Q_p response may not always be strongly correlated with annual peak RI (Wright et al. 2014) due to effects of watershed storage assumed to be negligible in the design storm approach.

The variability of RI in time and space is a major reason for limiting the watershed size when using the RM to estimate Q_p (Hayes and Young 2006). Storage in a watershed and the uncertainty in estimating the runoff coefficient are the main sources of

error in discharge estimation by this method (ODOT 2014; Grimaldi and Petroselli 2015). Therefore, we hypothesized that the RM will perform better on the smaller watersheds in NC and AR than the relatively larger watershed in SC, with potentially larger storage.

The RM can be formulated as

$$Q = C_u C_i A \quad (1)$$

where Q = design discharge; C_u = unit conversion coefficient; C = runoff coefficient or the fraction of the rainfall converted to runoff; i = design rainfall intensity based on watershed time of concentration (t_c); and A = watershed drainage area. A detailed discussion on deriving C and the t_c is provided in the Supplemental Materials SI-4 and SI-5, respectively.

Due to our interest in large design discharges, we computed mean C values by applying the RM using the top 10 Q_p with their corresponding RI (Table 1 and SI-1). These mean C values were further corrected for design of less frequent floods by factors of 1.1, 1.2, and 1.25 for return intervals of 25, 50, and 100 years, respectively, as used in Hayes and Young (2006) and recommended by ODOT (2014). Hayes and Young (2006) evaluated eight small watersheds (1–21.3 ha) consisting of combined road and ditch, pasture, new growth forest, residential, and industrial areas located in central Virginia. We did not use the C values back-calculated from specific storm events from another study (Walega et al. 2020) (Table S1) because those values did not appear to represent watershed response to large events with low frequency, as is the interest of this study.

We computed total t_c as the sum of the overland flow component (Kerby 1959) and the stream channel component (NRCS 2004). While the calculated t_c for Coweeta and Alum Creek sites were less than 1 h (Table 1), we assumed that the maximum RI computed from hourly (the minimum available subdaily) precipitation data would be adequate for these forested watersheds.

Design Discharge Estimation Using USGS Regional Regression Equations (RREs)

The USGS RREs are developed periodically using generalized least squares regression based on regionalized flood-frequency information determined for a group of gauged stations within a hydrologically homogeneous region. These predictive equations represent the statistical relationship between FFA estimates and basin characteristics (e.g., basin area, impervious percentage, slope, maximum rainfall intensity, etc.) and are widely used to estimate design discharge at different return intervals (Table 3). While Feaster et al. (2014) provided the most recent USGS RRE for North Carolina, South Carolina, and Georgia, we used the equations reported by Weaver et al. (2009) for the Coweeta because Feaster et al. (2014) did not include equations for Region 2 (Blue Ridge ecoregion) in North Carolina where the Coweeta site is located. A similar report for Arkansas was provided by Wagner et al. (2016). These USGS publications used annual peak flow rate data from gauging stations with

Table 2. Applicability of the Rational Method (developed by Kuichling 1889) and USGS Regional Regression Equations Methods (published by Feaster et al. 2014; Wagner et al. 2016; Weaver et al. 2009)

Reference	Site	Drainage area [km ² (ha)]	Minimum USGS area limit [km ² (mi ²)]	Applicable for USGS?	Applicable for Rational method?
Weaver et al. (2009)	WS27NC	0.39 (39)	2.59 (1.0)	No	Yes
Weaver et al. (2009)	WS14NC	0.61 (61)	2.59 (1.0)	No	Yes
Wagner et al. (2016)	AC04AR	0.13 (13)	0.259 (0.1)	No	Yes
Feaster et al. (2014)	WS80SC	1.60 (160)	0.259 (0.1)	Yes	No

Table 3. USGS Regional flood-frequency equations for estimating peak discharge (in cfs) at various return intervals. The equations for WS80, South Carolina, are for the Coastal Plain region and the WS14 and WS27, North Carolina, sites are for the Piedmont Blue Ridge region

Return interval	WS14 and WS27, North Carolina	AC04, Arkansas	WS80, South Carolina
2-year	$110 \times A^{0.779}$	$10^{2.850} \times A^{0.761} \times S^{-0.462}$	$26.3 \times A^{0.5908} \times 10^{(I_{24,50Y} \times 0.0515)}$
5-year	$209 \times A^{0.749}$	$10^{3.171} \times A^{0.737} \times S^{-0.510}$	$40.6 \times A^{0.5958} \times 10^{(I_{24,50Y} \times 0.0623)}$
10-year	$288 \times A^{0.736}$	$10^{3.329} \times A^{0.723} \times S^{-0.526}$	$51.8 \times A^{0.6004} \times 10^{(I_{24,50Y} \times 0.0666)}$
25-year	$398 \times A^{0.724}$	$10^{3.490} \times A^{0.707} \times S^{-0.535}$	$67.1 \times A^{0.6067} \times 10^{(I_{24,50Y} \times 0.0708)}$
50-year	$479 \times A^{0.718}$	$10^{3.589} \times A^{0.696} \times S^{-0.537}$	$78.4 \times A^{0.6111} \times 10^{(I_{24,50Y} \times 0.0738)}$
100-year	$575 \times A^{0.713}$	$10^{3.674} \times A^{0.686} \times S^{-0.536}$	$90.5 \times A^{0.6154} \times 10^{(I_{24,50Y} \times 0.0762)}$

Note: Regional flood-frequency equations were obtained from USGS publications for rural basins in North Carolina (Weaver et al. 2009), Arkansas (Wagner et al. 2016), and South Carolina (Feaster et al. 2014). A = drainage area in square miles; S = basin shape factor (the square of the length of the longest flow path in a stream basin divided by the drainage area of the stream basin) and was set as 3.17 in AC04 based on calculation using USGS method (Wagner et al. 2016); and $I_{24,50Y}$ refers to the 50-year maximum precipitation (in) for a duration of 24 h (used 12.3 in/day for WS80 based on historical records in Fig. S4 instead of published data in NOAA Atlas-14, Volume 2 (Bonnin et al. 2006). From Table 9 (Feaster et al. 2014), average standard errors of predictions for the preceding regression equations for Region 1 (Blue Ridge-Piedmont), the region where the North Carolina sites are located, vary from 15% for the 10-year return period to 32.1% for the 100-year return period. However, the errors for the WS80 South Carolina site in Region 4 (Coastal) are somewhat larger varying from 36.7% for the 10-year return period to 42.7% for the 100-year return period. Similarly, Wagner et al. (2016) reported the standard error prediction in the range of 32.76%–59.53% for all four regions for the Arkansas sites.

10 or more years of data only through 2011 for the NC and SC regions (Feaster et al. 2014), only through 2006 for the NC region (Weaver et al. 2009), and through 2013 for the AR site (Wagner et al. 2016). The USGS RREs for the three sites and their parameters are summarized in Table 3. The equations used for the SC site (Region 4, coastal) use drainage area (minimum 26 ha), percentage of impervious land cover (minimum 0.02%), and the 24-h 50-year maximum rainfall intensity [range of 165–277 mm based on NOAA Atlas-14 (2) data only through 2000 as explanatory variables (Bonnin et al. 2006)]. The equations, used for the NC watersheds in the Piedmont-Blue Ridge Region, use only drainage area and percentage of impervious land cover as explanatory variables (Weaver et al. 2009). Equations for AC04 site in AR in Flood Region B, Sub-region 1 (Wagner et al. 2016), also use basin shape factor, defined below in Table 3 footnotes. Although only WS80 was within the range of drainage area recommended for these equations [Feaster et al. (2014), updated from Weaver et al. (2009)], we also compared them with the RM for the other three watersheds (Table 2).

Model Evaluation Criteria

The performance of the RM and USGS RRE in predicting the Q_p of various frequencies was evaluated by comparing predictions to the observed flood frequencies obtained using the LPIII and GEV distributions and the computed relative percent error as

$$RE = \frac{\text{Predicted } Q_p - \text{Observed } Q_p}{\text{Observed } Q_p} \times 100 \quad (2)$$

as a performance evaluation criterion. Details of statistical evaluations for performance of the LPIII and GEV distributions compared to the observed flood frequency data are given in the Supplemental Materials.

Results and Discussion

Comparison between PIDF Estimates

Dissimilarities were evident between the PIDF estimates from on-site data and NOAA regional estimates for duration ≤ 3 h (Fig. 3) and Fig. S3 for durations ≥ 6 h. Specifically, the NOAA regional analysis-based PIDF underestimated RI by 10%–60% when compared to estimates using data from Santee, SC, and the percent errors increased for longer return intervals. On-site PIDF estimates for the two Coweeta locations (RRG06 and RRG41) at low

elevations were comparable (percent error $<10\%$) to the PIDF from the NOAA regional estimates for short durations of ≤ 3 h. In contrast, the NOAA-based PIDF overpredicted RI by more than 22% for long duration (≥ 6 h) with a return interval of 25 years or longer (Fig. S3). Similar differences in PIDF values for RRG06 and nearby RRG41 on lower elevation were observed, likely due to the inclusion of only the RRG06 and no other on-site gauges in the NOAA network (Figs. S4–S6). In addition, compared to the PIDF from on-site data for station RRG31 at the higher elevation, NOAA-based PIDF overestimated RI by as much as 30% for duration ≤ 12 h and return intervals >25 year, although it provided comparable estimates for the 24-h duration (Fig. S3).

The PIDF curves derived from on-site data for Alum Creek site in AR were relatively comparable to the NOAA estimates, while the PIDF values derived from the NOAA method clearly overestimated RI for the 1-h duration at other sites (Fig. 3). The better agreement between the PIDF values derived from on-site data and NOAA regional interpolation for the Alum Creek site, in comparison to the other sites, is attributed to the NOAA values used for this site, which were based on data through 2013, compared to data through 2004 for the SC and NC regions. For example, the relationship between the WS80 (Santee) on-site based 24-h maximum rainfall data and the Charleston airport-based NOAA data for a longer period (1977–2016, excluding filled data) showed a higher R^2 (0.61) compared to 0.37 when only 1977–1993 data were used.

The on-site PIDF estimates tended to have a larger uncertainty band (Fig. 3) than the NOAA estimates because of their relatively brief data record period and we did not use data from some other on-site gauges within its vicinity (~ 8 km), as shown in the Supplemental Materials (SI-6). NOAA's regional frequency analysis uses data from several sites for interstation interpolation accounting for both the distance from the weather stations and the elevations, thereby increasing the number of stations to perform the inference, and resulting in more accurate estimates (Hosking and Wallis 2005; Kysely et al. 2011). This approach, however, may introduce uncertainty due to spatial heterogeneity across stations, e.g., elevation, land cover, etc. For example, there were no stations within 25-km distance and only 2 stations within 32 km of WS80 (Santee) (Fig. S4). The inclusion of data from stations that are far away from the target site may result in a loss of accuracy particularly during summer-fall tropical storm events with high spatial variability. However, as England et al. (2018) argued, our estimates based on on-site PIDF can also be expected to vary widely from one site to another even within a small region due to sampling variability

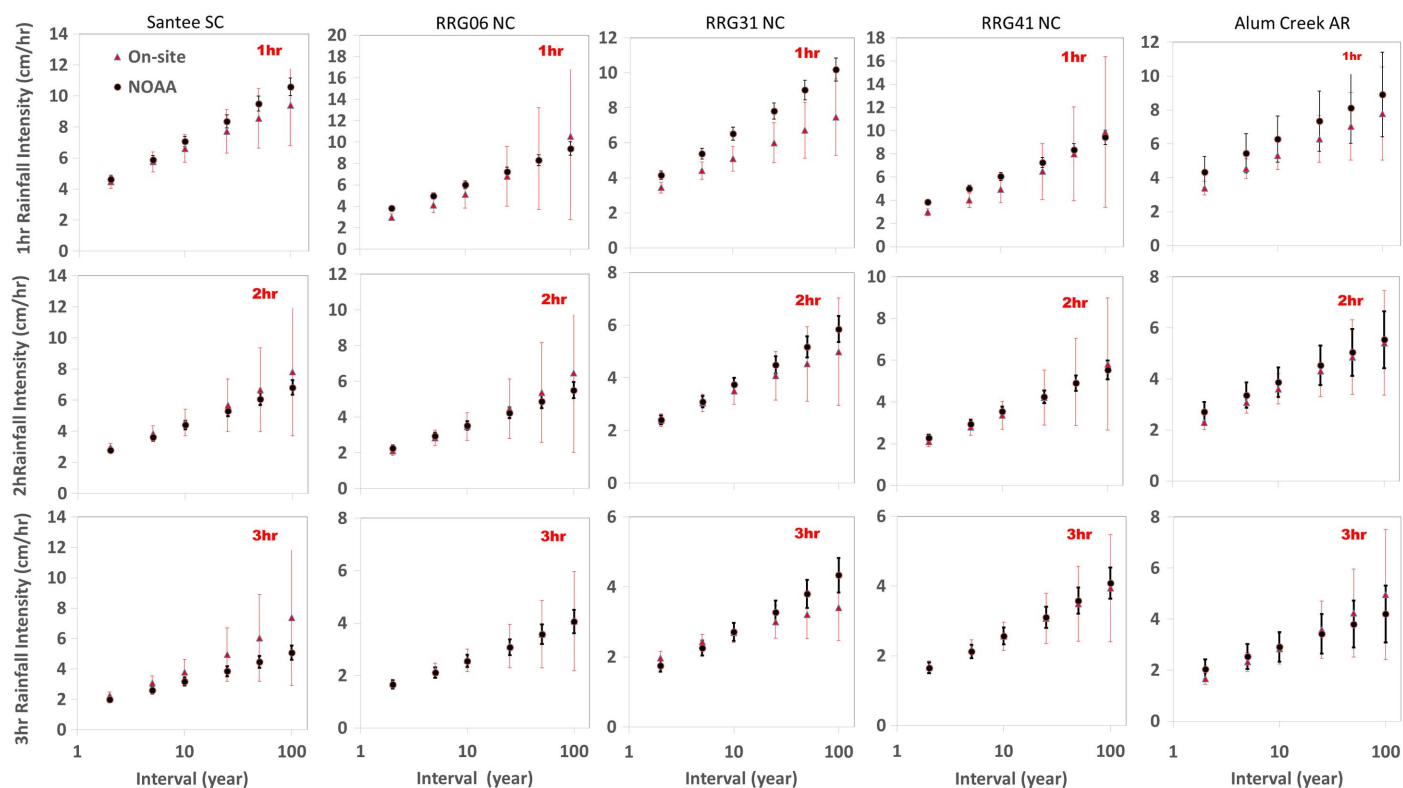


Fig. 3. Comparison of the GEV distribution-based PIDF curves (for durations of 1, 2, and 3 h) derived from on-site data (triangles) and estimated by the NOAA regional method (black circles) for the rain gauges at Santee, South Carolina (first column), Coweeta, North Carolina (2nd, 3rd, and 4th columns), and Alum Creek, Arkansas (5th column). The solid points are estimated magnitudes of rainfall intensities, while the vertical bars represent 5th and 95th percentiles.

and its effect on the uncertainty in the skewness coefficient, rather than actual climatic differences between sites.

Thus, our analysis shows the NOAA regional PIDF estimates yielded more conservative (higher) RI for smaller watersheds with less than an hour estimated t_c where the RM is most applicable. One likely reason is because the NOAA results capture storm data from a number of stations over a larger area that would have otherwise been missed by a single gauge. The opposite seems to be true for the larger watershed (WS80), which implies a larger storm cell size yielding a longer t_c . The on-site PIDF data estimated higher RI (more conservative) than the NOAA data for the longer durations (2- and 3-h) rain (Fig. 3) on the largest WS80 site. Thus, it is plausible to consider the on-site based RI estimates together with cost-benefit analyses in infrastructure designs at this site.

Flood Frequency Analysis Using Discharge Records

Design FFA curves derived by fitting the LPIII and GEV distributions with the historical discharge records from the four watersheds were in close agreement for return intervals of <25-year (Fig. S7). Meanwhile, the GEV-based flood discharges at return intervals >50 years are evidently larger (>34% for the 100-year) than the LPIII-based predictions for WS80, SC and AC04, and AR (Fig. S7 and Table 4). This may be attributed to the GEV distribution, which is more sensitive to extreme values and yields larger uncertainty bands (England et al. 2018; Hosking and Wallis 2005). Nevertheless, this simple comparison does not imply any preference between the two distribution types because of the large uncertainties for estimating >50-year flooding using only 40-year measurements. A comparative evaluation between these two distributions using various statistics and probability plots, including the Akaike

Information Coefficient (AIC), Bayesian Information Coefficient (BIC), and Anderson-Darlington Criterion (ADC) (SI-7), presents LPIII as a slightly better choice than the GEV (Table S5). Hence the LPIII was used in rest of the analyses.

Comparison of Flood Frequencies with Previous Studies

Amatya and Radecki-Pawlik (2007) reported lower design discharge for WS80 using the Pearson III-type FFA distribution because they used daily average peak discharge Q_{peak} instead of instantaneous maximum rate from a limited 13-year (1969–1981) record. Particularly, the inclusion of large storm events (e.g., on October 3–4, 2015, due to an indirect effect of Hurricane Joaquin in SC) could increase the estimated Q_p (St. George and Mudelsee 2018). The difference between the present study and the study by Amatya and Radecki-Pawlik (2007) demonstrates that use of data record spanning different periods could capture different flood regimes yielding different outcomes of the flood frequency analysis. In this study, the estimated design discharges based on LPIII model for AC04 (Fig. S7) are comparable to estimates using a different empirical method at this same site (Marion 2004). Eisenbies et al. (2007) concluded that even where data are available for relatively long-term periods (20–50 years) and the Q_p distribution is fairly well known, there is little confidence in estimates of Q_p for return periods >50 years.

Comparison between RM and USGS RRE Design Peak Discharge Estimates

Large differences were found between the design Q_p estimated by the RM utilizing on-site PIDFs and the USGS RRE (Table 4).

Table 4. Design discharges (cms km^{-2} or $\text{m}^3 \cdot \text{s}^{-1} \text{km}^{-2}$) derived from historical records (LPIII and GEV based median design discharges) and estimated by the Rational method with on-site PIDF curves and USGS regional regression equations (RREs)

Watershed, state	Return period	On-site LPIII	On-site GEV	USGS RRE to LPIII	Rational method to LPIII
WS14, North Carolina	2-year	0.5	0.5	1.7 (230%)	0.7 (48%)
	5-year	0.7	0.7	3.3 (370%)	1.0 (43%)
	25-year	1.0	1.0	6.5 (550%)	1.8 (78%)
	50-year	1.1	1.1	7.9 (620%)	2.4 (120%)
	100-year	1.2	1.3	8.9 (640%)	3.1 (150%)
WS27, North Carolina	2-year	1.8	2.1	1.8 (0%)	3.2 (75%)
	5-year	2.6	2.8	3.7 (42%)	4.0 (56%)
	25-year	4.1	4.4	6.5 (58%)	6.4 (55%)
	50-year	4.6	5.1	8.9 (93%)	7.4 (61%)
	100-year	5.4	5.9	10.8 (100%)	8.6 (58%)
WS80, South Carolina	2-year	0.6	0.6	1.5 (150%)	0.9 (50%)
	5-year	1.7	1.4	3.1 (82%)	1.2 (−29%)
	25-year	4.6	4.8	6.6 (43%)	1.9 (−58%)
	50-year	6.5	7.6	8.4 (29%)	2.4 (−63%)
	100-year	9.1	12.0	10.3 (13%)	3.0 (−67%)
AC04, Arkansas	2-year	4.6	4.6	9.3 (102%)	5.6 (21%)
	5-year	7.7	6.9	19.8 (157%)	7.4 (−3%)
	25-year	11.5	12.3	43.8 (281%)	11.3 (−1%)
	50-year	13.1	14.6	56.7 (333%)	13.8 (6%)
	100-year	14.6	17.7	71.2 (388%)	16.0 (9%)

Note: Values in parentheses represent the relative percent errors (RE) compared to design discharges from the LPIII distribution (Positive RE is overprediction and negative is underprediction).

Compared to the RM, the USGS RREs–based design Q_p estimates were consistently higher for return intervals >25 years for all sites. For example, the USGS estimate of $8.9 \text{ m}^3 \cdot \text{s}^{-1} \text{km}^2$ for the 100-year return period at WS14 site was almost three times greater than the $3.1 \text{ m}^3 \cdot \text{s}^{-1} \text{km}^2$ estimate by the RM for the same site and return period (Table 4). One likely reason for this result is the RM assumes that flood estimates respond linearly to increases in RI [Eq. (1)] as opposed to exhibiting a linear relationship to the drainage area, which the USGS method assumes. Interestingly, discrepancies between the two methods were smallest (0.77%, on average) for smaller high-gradient WS27 watershed (39 ha) in NC in contrast with 210% error, on average, for WS14 at Coweeta, North Carolina. The smaller average error for the WS27 was due to both underestimates of lower return intervals and overestimates of return intervals greater than 25 years by the USGS RRE compared to the RM.

Comparison between Empirical and Observed Design Discharge Estimates

A cursory examination of computed design discharges for various return periods showed the RM was superior to the USGS RRE for estimating the design of Q_p when compared against the LPIII model of the observed data (Table 4). The relative errors showed that the USGS RRE overestimated design discharges, on average for all frequencies, by 482%, 59%, 63%, and 252% for WS14, WS27, WS80, and AR04, respectively, when compared to LPIII based estimations. In contrast, the RM provided design discharge estimates with lower mean percent errors for small watersheds yielding only 88% and 61% overestimate for the WS14 and WS27, respectively, and only 6% overestimate for the AC04 watershed. In the larger WS80 watershed, the RM underestimated the design discharge for a return interval of ≥ 25 years by as much as 63% in contrast to only 28% overestimate by the USGS RRE. Larger error by the RM is likely due to its application to a gently sloped watershed with substantial storage that has a large drainage area (160 ha), beyond the

120-ha limit recommended for the method (Thompson 2006; ODOT 2014). These results suggest that the RM is more reliable for the smaller watersheds (WS14 and WS27 in NC and AC04 in AR) while the USGS RRE are more reliable for the large WS80 watershed for return intervals typically of interest for the design of road cross-drainage structures (≥ 25 years).

One possible reason for the poor performance of the USGS RRE for three of the four forested watersheds is the fact that the USGS equations were derived using data from watersheds larger than our study watersheds (Table 2). For instance, the RRE for the Blue Ridge region in North Carolina, where the WS14 and WS27 watersheds are located, were based on measurements from rural watersheds larger than 260 km^2 (Weaver et al. 2009). The tendency of the USGS RRE to overestimate design discharge for small undisturbed watersheds has been reported in other studies (Marion 2004; Thompson 2006; Trommer et al. 1996). Genereux (2003) cautioned that extrapolation to smaller watersheds outside the bounds of their regression data may result in a potentially significant and unknown uncertainty.

In general, the RM performed better for estimating design Q_p for three high-gradient watersheds (AC04 in Alum Creek, followed by WS27 and WS14 in Coweeta) when compared to the coastal lowland watershed (WS80) in Santee, South Carolina (Tables 2, 4, and S6). The relatively good performance of the RM for AC04 watershed compared to WS27 and WS14 watersheds is attributed to its smaller size ($AC04 < WS27 < WS14$), which complies with the RM assumptions of uniform rainfall and negligible storage better than the other two watersheds. Genereux (2003) suggested that the RM, based on a 50-year design Q_p estimate, was superior to the estimates obtained by other methods including the USGS RRE, the Natural Resource Conservation Service TR-55 method, and the North Carolina Department of Transportation methods for a 19.6-ha 66% forested watershed in the Piedmont region of North Carolina. However, multiple studies have shown that the empirical runoff coefficient in the RM can be a critical source of error (Grimaldi and Petroselli 2015).

Additional studies have reported that RM estimates can be significantly improved by using runoff coefficients and t_c derived from field measurements, where available, (Hayes and Young 2006; Thompson 2006; Trommer et al. 1996) and different return intervals (McEnroe et al. 2013). We hypothesize that the use of one-hour duration instead of subhourly duration based on the estimated t_c for the smaller watersheds (Table 1) might have led to lower RI and yielded lower design discharge values by the RM and closer agreement with the LPIII model (Table 4) than otherwise would have been obtained with the RI for subhourly duration. In other words, the bias causing RI underestimation offsets the tendency of RM to overestimate design discharges for these watersheds. Thus, it is important to carefully examine both RI and Q_p estimation methods to identify the method that better represents the hydrologic behavior of forested headwater watersheds. For example, RI is the dominant factor for small areas, the storage characteristics for larger drainage basins, and the AMCs for less developed areas, especially for rainfall events with a return period of 10 years or less (Virginia DOT 1999).

Also, the use of the NOAA RI data for 1-h duration that were greater than the on-site data for the WS27 and AC04 sites (Fig. 3) would have yielded even larger overestimates by the RM. Nevertheless, the relatively better estimation of design discharge, compared to the USGS RRE, for the high relief watersheds suggests that using RI and frequency at 1-h duration is a better approximation than USGS RRE for some small forested watersheds with t_c less than 1 h. However, our results and other studies have shown that the design estimates of t_c generally were shorter than the

estimates derived from the Rational Hydrograph Method (Hayes and Young 2006), which is another source of uncertainty warranting additional studies.

The RM clearly yielded a lower bias compared to the USGS method, with slope close to unity (R^2 and NSE = 0.95) for all three small watersheds (WS14 and WS27 in North Carolina and AC04 in AR) (Table S6). The opposite was found to be true for the largest watershed WS80 in SC, with the USGS method yielding the least biased relationship with a slope close to unity and $R^2 = 0.99$ and NSE = 0.75 (Table S6). This confirms that the RM performed better on the smaller watersheds but not on the large WS80 watershed, while the USGS was applicable on the large watershed in this study (Table 2). In addition to the overall larger watershed area, greater storage provided by wetlands and floodplain (30% of the area) (Amatya et al. 2019) and a 1,365 m long stream channel might have also influenced the runoff coefficient in the RM. The literature suggests that any storage features, including detention ponds, channel, and floodplains that do not completely fill and reach a steady inflow-outflow condition during the duration of a design storm are not properly represented with the RM (ODOT 2014).

The relationships between the design of Q_p derived from on-site data and RI of various return intervals (2, 5, 25, 50, and 100-year) were highly linear ($R^2 > 0.98$) for the three watersheds with high relief (WS14, WS27, and AC04) (Fig. 4). This is consistent with a recent study that showed that the WS14 and WS27 watersheds exhibited relatively linear responses of total storm flow to the sum of event rainfall and lateral flow controlling the Q_p (Scaife and Band 2017). This may further indicate that there is no need to use

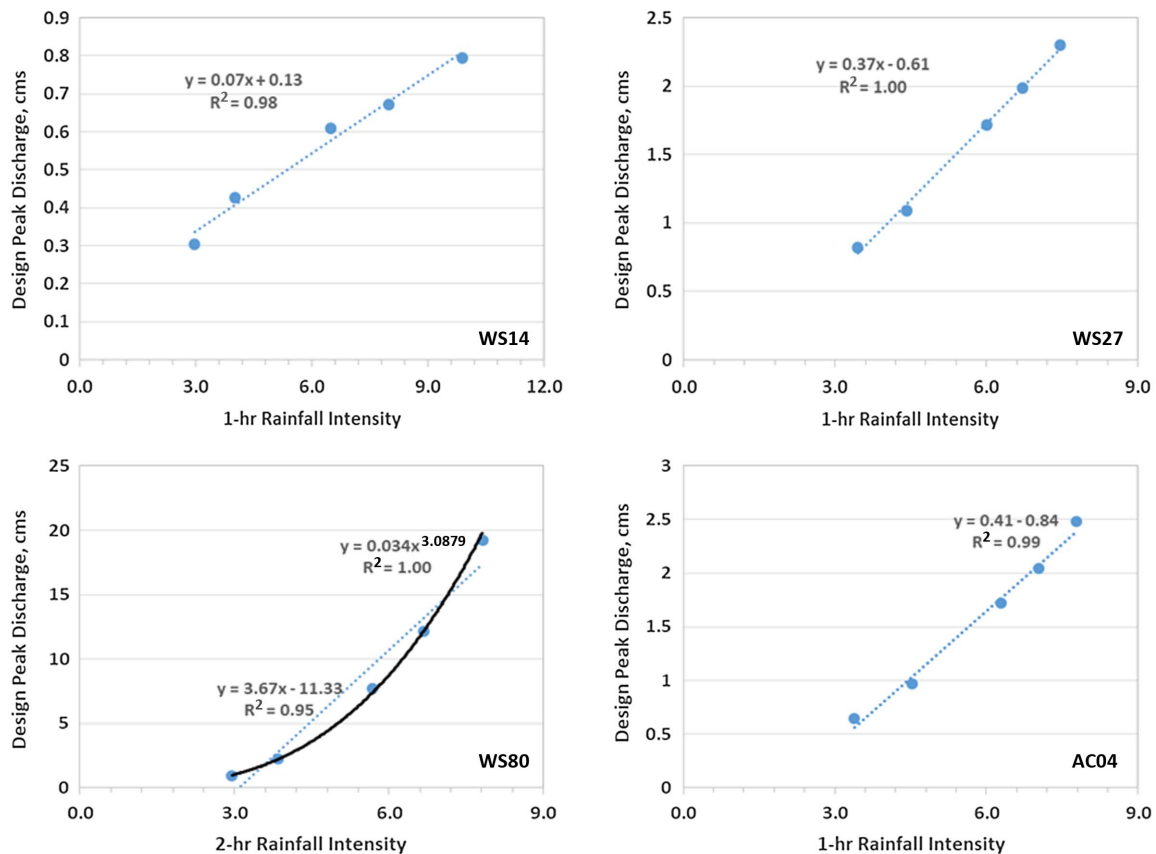


Fig. 4. The relationship between rainfall intensities for 1-h duration for WS14, WS27, and AC04 and 2-h duration for WS80 and corresponding design peak discharges [$\text{cms} (\text{m}^3 \cdot \text{s}^{-1})$] derived from on-site data for various return intervals (2-, 5-, 25-, 50-, and 100-year). Both the on-site data based flood frequency analysis and PIDF curves were derived using the GEV distribution function.

recurrence-interval correction factors when using the RM in small forest watersheds with high relief.

If we do not use the correction factors (1.1, 1.2, and 1.25, for return intervals of 25, 50, and 100 years, respectively) while estimating design discharge for the three high relief watersheds, the mean absolute errors (Table 4) will be reduced to 67%, 45%, and 3% for WS14, WS27, and AC04, respectively. However, a nonlinear relationship was found for the lowland coastal watershed (WS 80), with an R^2 nearly equal to 1.0 for a power relationship as opposed to $R^2 = 0.95$ for the linear one (Fig. 4), which also had the largest drainage area and higher storage capacity (Amatya et al. 2019; Harder et al. 2007). The nonlinear relationship between the design Q_p and RI on WS80, in contrast with the other three sites in this study, could suggest the need for using runoff coefficients that vary among extreme rainfall events of different probabilities, potentially due to dynamic antecedent storage as affected by rainfall and ET. This is expected, as water table position and microtopography influencing storage, are critical factors that affect streamflow patterns, stormflow peaks, and volume on shallow coastal forests (Sun et al. 2002; Walega et al. 2020). Shallow, saturated overland flow is the dominant runoff generation mechanism for WS80 and the adjacent landscapes (Amatya et al. 2019), where runoff rates were found to increase exponentially at one well location once the soil was saturated and water was ponded on the land surface (Harder et al. 2007). Further, storm event-based runoff coefficients for WS80 ranged from 0.01 to 0.74 under different AMC and storm intensities (Epps et al. 2012) (Table S1). Rainfall has been shown to be sensitive to the AMCs in runoff generation particularly during the growing season with high ET demand on this landscape (Epps et al. 2012; La Torre Torres et al. 2011). If feasible, the use of dynamic runoff coefficients derived from on-site measurements could improve the applicability of the RM for such lowland forested watersheds.

Future studies may consider using data also from other streamflow gages surrounding the study gauge within each of the three EFR sites (Table 1) to provide a locally regionalized basis for estimating the LPIII shape parameter, as suggested by Hosking and Wallis (2005).

Summary and Conclusions

The primary goal of this study was to evaluate design discharge estimation by RM using PIDF curves derived from on-site data and the USGS RRE compared to design discharge estimates from observed data for various return periods of interest for culvert design. Long-term (~40 year) precipitation and (>30-year) discharge records from four small, forested watersheds located in the Southeastern US were used for the PIDF and the flood frequency analysis, respectively. Compared to the on-site PIDF estimates, published NOAA Atlas-14 predictions substantially either under or overestimated rainfall intensities for longer return periods for the Coweeta, North Carolina and Santee, South Carolina sites, and provided estimates close to the on-site values for the Alum Creek, Arkansas site for durations >1 h. NOAA Atlas-14 rainfall intensities of 1-h duration were generally higher than estimates from on-site data. This suggests that NOAA data could be used for conservative design discharge estimates for small forested watersheds, as consequences of a structural failure due to less conservative estimates are often much higher than any overdesign costs.

While performance was generally poor, the RM performed relatively better than the USGS RRE for estimating design peak discharges in the three small, high-relief watersheds. On the contrary, the USGS RRE performed better at the large WS80 site in SC

(Table 2). The larger drainage area and the greater water storage capacity of WS80 most likely explain the low performance of the RM on that watershed. Thus, our results on these four headwater forest watersheds clearly corroborate drainage-area thresholds recommended for the application of these methods in watersheds with other land uses (Table 2).

Given the findings of this study, the authors recommend using the RM with on-site PIDF-based RI for estimating design Q_p on the gauged study sites and NOAA PIDF for small, ungauged sites (Table 4). However, the results of these case studies should be cautiously interpreted before extrapolating them to other catchments because: (1) on-site based PIDF may have relatively large uncertainty in the sample skewness coefficient due to its sensitivity to extreme events in modest length records, as argued by England et al. (2018); (2) high variability in runoff coefficients used in the RM; and (3) its limited application on a single large lowland watershed.

While the performance of the RM may be improved by using both C and t_c based on observed data for watersheds, additional comparisons are recommended between watersheds of different sizes with varying topographic gradients, particularly on larger, high-gradient forest watersheds. On the other hand, USGS RRE is likely a better choice for larger forested watersheds within the method drainage area limits (Table 2). However, further investigations on the various climatic and catchment parameters used in the USGS equations (e.g., 50-year 24-h rainfall intensity estimates) for design discharges of all return periods considered may be warranted.

Data Availability Statement

Some or all data, models, or code generated or used during the study are available from the corresponding author by request.

Acknowledgments

The authors would like to acknowledge the USDA Forest Service Southern Research Station (SRS) and Region 8 for funding this study. The authors also wish to thank Shawna Reid at the SRS and former graduate student at North Carolina State University, Jessica Allen, for help with a map figure and data digitization, respectively. The authors also thank Maureen Stuart and Greta Langhenry at the SRS for editing help in the earlier version of the manuscript, Melissa Griffin at SC Department of Natural Resources for providing data from the Charleston Airport, SC, and Paul Moore, P.E. at Ward Edwards Engineering, SC, and Dr. Debabrata Sahoo, P.E. formerly at Woolpert, SC, for their constructive suggestions on parts of the revised manuscript. Last but not the least, the authors express their sincere thanks to the editor and all the anonymous reviewers for their patience and great constructive comments and suggestions that, we believe, helped to enhance the manuscript quality in this revision. Furthermore, the authors also thank the reviewers of earlier version of this manuscript. *“The opinions presented in this article are those of the authors and should not be construed to represent any official USDA or US Government determination or policy.”*

Supplemental Materials

Figs. S1–S8 and Tables S1–S5 are available online in the ASCE Library (www.ascelibrary.org).

References

- Adams, M. B., and L. Loughry. 2008. *Experimental forests and ranges of the USDA Forest Service*. General Technical Rep. No. NE-321. Washington, DC: USDA, Forest Service, Northeastern Research Station.
- Amatya, D., J. Campbell, P. Wohlgenuth, K. Elder, S. Sebestyen, S. Johnson, E. Keppeler, M. Adams, P. Caldwell, and D. Misra. 2016. "Hydrological processes of reference watersheds in experimental forests." Chap. 14 in *Forest hydrology: Processes, management and assessment*, edited by D. M. Amatya, T. M. Williams, L. Bren, and C. deJong, 141–161. Wallingford, UK: Centre for Agriculture and Bioscience International Publishers.
- Amatya, D. M., and A. Radecki-Pawlik. 2007. "Flow dynamics of three experimental forested watersheds in coastal South Carolina." *ACTA Sci. Polonorum Formatio Circumiectus* 6 (2): 3–17. <https://doi.org/10.13031/trans.13387>.
- Amatya, D. M., T. M. Williams, J. E. Nettles, R. W. Skaggs, and C. C. Trettin. 2019. "Comparison of hydrology of two Atlantic coastal plain forests." *Trans. ASABE* 62 (5): 1509–1529. <https://doi.org/10.13031/trans.13387>.
- Bonnin, G. M., D. Martin, B. Lin, T. Parzybok, M. Yekta, and D. Riley. 2006. "Precipitation frequency atlas of the United States." *NOAA Atlas* 14 (2): 1–65.
- Caldwell, P. V., C. F. Miniati, K. J. Elliott, W. T. Swank, S. T. Brantley, and S. H. Laseter. 2016. "Declining water yield from forested mountain watersheds in response to climate change and forest mesophication." *Global Change Biol.* 22 (9): 2997–3012. <https://doi.org/10.1111/gcb.13309>.
- Cheng, L., and A. AghaKouchak. 2014. "Nonstationary precipitation intensity-duration frequency curves for infrastructure design in a changing climate." *Sci. Rep.* 4 (2014): 7093. <https://doi.org/10.1038/srep07093>.
- Donahue, J. P., and A. F. Howard. 1987. "Hydraulic design of culverts on forest roads." *Can. J. For. Res.* 17 (12): 1545–1551. <https://doi.org/10.1139/x87-237>.
- Eisenbies, M., W. Aust, J. Burger, and M. B. Adams. 2007. "Forest operations, extreme flooding events, and considerations for hydrologic modeling in the Appalachians—A review." *For. Ecol. Manage.* 242 (2–3): 77–98. <https://doi.org/10.1016/j.foreco.2007.01.051>.
- England, J. F., Jr., T. A. Cohn, B. A. Faber, J. R. Stedinger, W. O. Thomas Jr., A. G. Veilleux, J. E. Kiang, and R. R. Mason Jr. 2018. *Guidelines for determining flood flow frequency—Bulletin 17C*. Techniques and Methods 4-B5. Reston, VA: US Department of the Interior, USGS.
- Epps, T. H., D. R. Hitchcock, A. D. Jayakaran, D. R. Loflin, T. M. Williams, and D. M. Amatya. 2012. "Characterization of storm flow dynamics of headwater streams in the South Carolina lower coastal plain." *J. Am. Water Resour. Assoc.* 49 (1): 76–89. <https://doi.org/10.1111/jawr.12000>.
- Feaster, T. D., A. J. Gotvald, and J. C. Weaver. 2014. *Methods for estimating the magnitude and frequency of floods for urban and small, rural streams in Georgia, South Carolina, and North Carolina, 2011*. Columbia, SC: USGS.
- Genereux, D. P. 2003. "Comparison of methods for estimation of 50-year peak discharge from a small, rural watershed in North Carolina." *Environ. Geol.* 44 (1): 53–58. <https://doi.org/10.1007/s00254-002-0734-5>.
- Gilleland, E., and R. W. Katz. 2016. "extRemes 2.0: An extreme value analysis package in R." *J. Stat. Software* 72 (8): 1–39. <https://doi.org/10.18637/jss.v072.i08>.
- Grimaldi, S., and A. Petroselli. 2015. "Do we still need the rational formula? An alternative empirical procedure for peak discharge estimation in small and ungauged basins." *Hydrol. Sci. J.* 60 (1): 67–77. <https://doi.org/10.1080/02626667.2014.880546>.
- Harder, S. V., D. M. Amatya, T. J. Callahan, C. C. Trettin, and J. Hakkila. 2007. "Hydrology and water budget for a forested Atlantic coastal plain watershed, South Carolina 1." *J. Am. Water Resour. Assoc.* 43 (3): 563–575. <https://doi.org/10.1111/j.1752-1688.2007.00035.x>.
- Hayes, D. C., and R. L. Young. 2006. *Comparison of peak discharge and runoff characteristic estimates from the rational method to field observations for small basins in central Virginia*. Scientific Investigations Rep. No. 2005-5254. Reston, VA: Department of the Interior, USGS.
- Heredia, N., B. Roper, N. Gillespie, and C. Roghair. 2016. *Technical guide for field practitioners: Understanding and monitoring aquatic organism passage at road stream crossings*. Technical Rep. No. TR-101. Washington, DC: USDA, Forest Service, National Stream and Aquatic Ecology Center.
- Hosking, J. R. M., and J. R. Wallis. 2005. *Regional frequency analysis: An approach based on L-moments*. Cambridge, UK: Cambridge University Press.
- Kerby, W. 1959. "Time of concentration for overland flow." *Civ. Eng.* 29 (3): 174.
- Kuichling, E. 1889. "The relation between the rainfall and the discharge of sewers in populous districts." *Trans. Am. Soc. Civ. Eng.* 20 (1): 1–56.
- Kyselý, J., L. Gaál, and J. Picek. 2011. "Comparison of regional and at-site approaches to modelling probabilities of heavy precipitation." *Int. J. Climatol.* 31 (10): 1457–1472. <https://doi.org/10.1002/joc.2182>.
- Laseter, S. H., C. R. Ford, J. M. Vose, and L. W. Swift. 2012. "Long-term temperature and precipitation trends at the Coweeta Hydrologic Laboratory, Otto, North Carolina." *Hydrol. Res.* 43 (6): 890–901. <https://doi.org/10.2166/nh.2012.067>.
- La Torre Torres, I. B., D. M. Amatya, G. Sun, and T. J. Callahan. 2011. "Seasonal rainfall-runoff relationships in a lowland forested watershed in the Southeastern." *Hydrol. Process.* 25 (2011): 2032–2045. <https://doi.org/10.1002/hyp.7955>.
- Mannering, F. L., and W. P. Kilareski. 1998. *Principles of highway engineering and traffic analysis*. 2nd ed. 340. New York: Wiley.
- Marion, D. A. 2004. *An improved flood-frequency model for small watersheds in the upper Ouachita Mountains*. General Technical Rep. No. SRS-74. Washington, DC: USDA, Forest Service, Southern Research Station.
- McEnroe, B. M., C. B. Young, A. R. Williams, and M. Hinshaw. 2013. *Estimating design discharges for drainage structures in Western Kansas*. Rep. No. K-TRAN: KU-124. Topeka, KS: Kansas DOT.
- NCDOT (North Carolina DOT). 2016. *Guidelines for drainage studies and hydraulic design*. Raleigh, NC: NCDOR.
- Novak, P., A. Moffat, C. Nalluri, and R. Narayanan. 2007. *Hydraulic structures*. 4th ed. New York: Taylor & Francis.
- NRCS (Natural Resources Conservation Service). 2004. *National engineering handbook: Part 630—Hydrology*. Washington, DC: USDA Soil Conservation Service.
- ODOT (Oklahoma DOT). 2014. "Hydrology." Chap. 7 in *Roadway drainage design manual*. Oklahoma City, OK: ODOT.
- Perica, S., D. Martin, S. Pavlovic, I. Roy, M. St Laurent, C. Trypaluk, D. Unruh, M. Yekta, and G. Bonnin. 2013. *NOAA Atlas 14 precipitation-frequency Atlas of the United States*. Silver Spring, MD: National Oceanic and Atmospheric Administration.
- Reinhart, K. G., and R. S. Pierce. 1964. *Stream gauging stations for research on small watersheds*. Agriculture Handbook No. 268. Washington, DC: USDA, Forest Service.
- Richardson, J. S., and R. J. Danehy. 2007. "A synthesis of the ecology of headwater streams and their riparian zones in temperate forests." *For. Sci.* 53 (2): 131–147. <https://doi.org/10.1093/forestscience/53.2.131>.
- Rogger, M., B. Kohl, H. Pirkl, A. Viglione, J. Komma, R. Kirnbauer, R. Merz, and G. Blöschl. 2012. "Runoff models and flood frequency statistics for design flood estimation in Austria—Do they tell a consistent story?" *J. Hydrol.* 456 (Aug): 30–43. <https://doi.org/10.1016/j.jhydrol.2012.05.068>.
- Scaife, C. I., and L. E. Band. 2017. "Nonstationarity in threshold response of stormflow in southern Appalachian headwater catchments." *Water Resour. Res.* 53 (8): 6579–6596. <https://doi.org/10.1002/2017WR020376>.
- SCDOT (South Carolina DOT). 2009. *Requirements for hydraulic design studies*. Columbia, SC: SCDOT.
- St. George, S., and M. Mudelsee. 2018. "The weight of the flood-of-record in flood frequency analysis." Supplement, *J. Flood Risk Manage.* 12 (S1): e12512. <https://doi.org/10.1111/jfr3.12512>.
- Sun, G., S. G. McNulty, D. M. Amatya, R. W. Skaggs, L. W. Swift, J. P. Shepard, and H. Riekerk. 2002. "A comparison of the watershed hydrology of coastal forested wetlands and the mountainous uplands in the

- Southern US." *J. Hydrol.* 263 (1–4): 92–104. [https://doi.org/10.1016/S0022-1694\(02\)00064-1](https://doi.org/10.1016/S0022-1694(02)00064-1).
- Swank, W. T., and D. J. Crossley. 1988. *Forest hydrology and ecology at Coweeta*. New York: Springer.
- Thompson, D. B. 2006. *The rational method, regional regression equations, and site specific flood-frequency relations*. Austin, TX: Texas DOT.
- Trommer, J., J. Loper, and K. Hammett. 1996. *Evaluation and modification of five techniques for estimating stormwater runoff for watersheds in west-central Florida*. Reston, VA: US Department of the Interior, USGS.
- Tummers, B. 2006. "DataThief III." Accessed October 14, 2017. <https://datathief.org/>.
- USDOT. 2002. *Highway hydrology: Hydraulic design series (HDS) no. 2*. Publication No. FHWA-NHI-02-001. Washington, DC: Federal Highway Administration, USDOT.
- USDOT. 2012. *Evaluating scour at bridges*. Hydraulic Engineering Circular No. 18, Publication No. FHWA-HIF-12-003. Washington, DC: Federal Highway Administration, USDOT.
- USDOT. 2018. *US forest service transportation resiliency guidebook: Addressing climate change impacts on US forest service transportation assets*. Cambridge, MA: USDOT John A. Volpe National Transportation Systems Center in Cambridge.
- USGS. 1982. *Guidelines for determining flood frequency*. Bulletin #17B. Reston, VA: USGS.
- US Water Resources Council. 1981. "Estimating peak flow frequencies for natural ungaged watersheds: A proposed nationwide test." In *Hydrology committee*. Washington, DC: US Water Resources Council.
- Virginia DOT. 1999. "Drainage manual." Chap 6 in Vol. 2 of *Hydrology: Virginia stormwater management handbook*. 1st ed. Richmond, VA: Virginia DOT.
- Wagner, D. M., J. D. Krieger, and A. G. Veilleux. 2016. *Methods for estimating annual exceedance probability discharges for streams in Arkansas, based on data through water year 2013*. Scientific Investigations Rep. Reston, VA: USGS.
- Walega, A., D. M. Amatya, P. Caldwell, D. Marion, and S. Panda. 2020. "Assessment of storm direct runoff and peak flow rates using improved SCS-CN models for selected forested watersheds in the Southeastern United States." *J. Hydrol. Reg. Stud.* 27 (Feb): 100645. <https://doi.org/10.1016/j.ejrh.2019.100645>.
- Wang, F., W. Shao, H. Yu, G. Kan, X. He, D. Zhang, M. Ren, and G. Wang. 2020. "Re-evaluation of the power of the mann-kendall test for detecting monotonic trends in hydrometeorological time series." *Front. Earth Sci.* 8 (Feb): 14. <https://doi.org/10.3389/feart.2020.00014>.
- Weaver, J., T. D. Feaster, and A. J. Gotvald. 2009. Vol. 2 of *Magnitude and frequency of rural floods in the Southeastern United States, through 2006*. Reston, VA: USGS.
- Weaver, W. E., and D. K. Hagans. 1994. *Handbook for forest and ranch roads: A guide for planning, designing, constructing, reconstructing, maintaining and closing wildland roads*. Washington, DC: Pacific Watershed Associates for Mendocino County Resource Conservation District in Cooperation with California Department of Forestry and Fire Protection and the USDA Soil Conservation Service.
- Wright, D. B., J. A. Smith, and M. L. Baeck. 2014. "Flood frequency analysis using radar rainfall fields and stochastic storm transposition." *Water Resour. Res.* 50 (2): 1592–1615. <https://doi.org/10.1002/2013WR014224>.