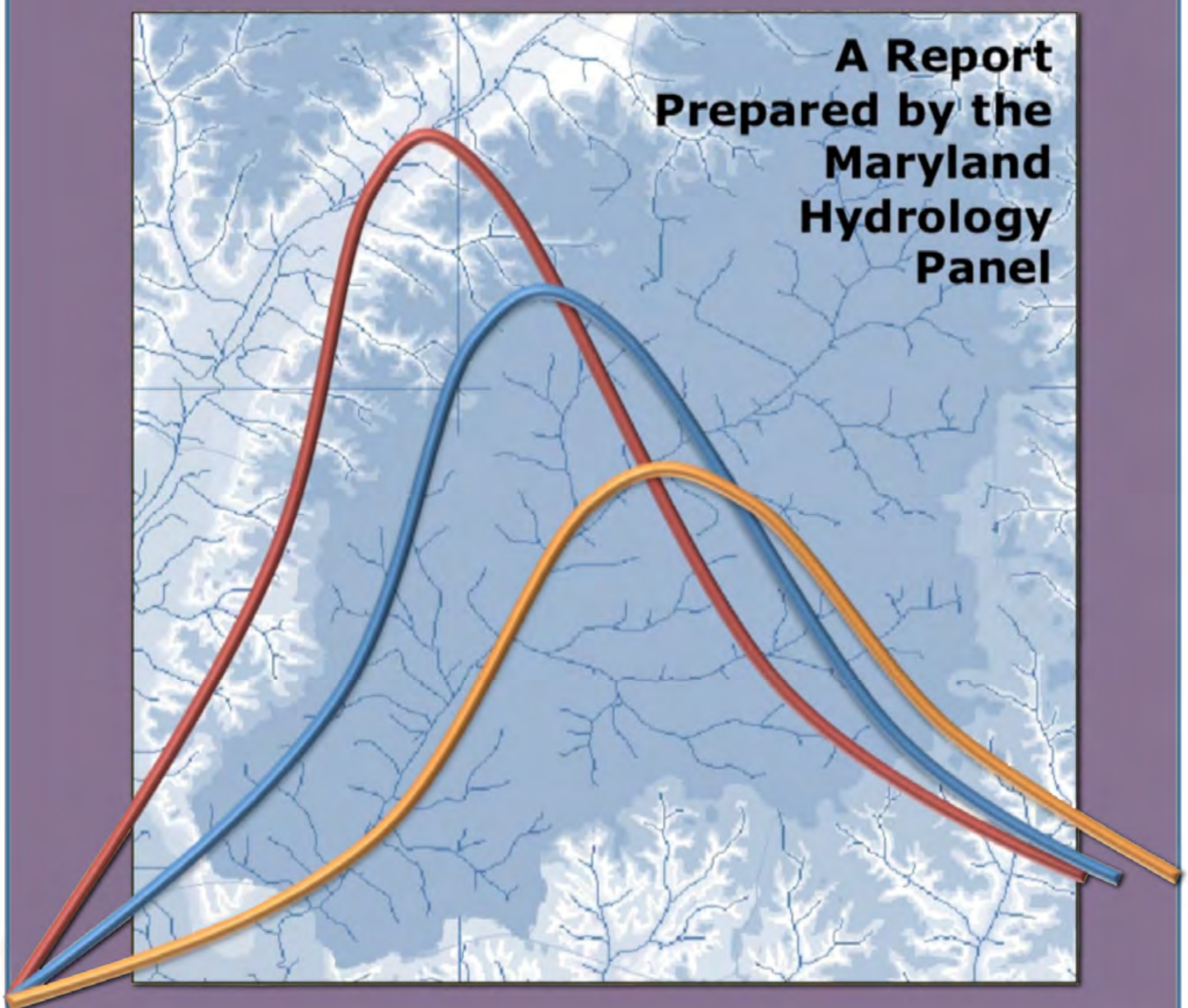
Application of Hydrologic Methods in Maryland

Sixth Edition, July 2023



MARYLAND DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION

MARYLAND

Wes Moore, Governor

Maryland
Department of the Environment

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September 2023

Subject: ***Application of Hydrologic Methods in Maryland***, Sixth Edition, July 2023

Users of this Manual:

On behalf of the State Highway Administration (SHA) and the Maryland Department of the Environment (MDE), we are delighted to endorse and recommend the use of this manual concerning hydrologic practices applied to State of Maryland projects. It is important to note that the manual will be the required criteria for all hydrologic analyses related to SHA bridge and highway design and is recommended for use by other State and local agencies.

This manual is intended to aid the practitioner in the prediction of peak flow rates and flood hydrographs for Maryland streams as well as to offer techniques and tools that will improve the results of rainfall-runoff computer models. The procedures in this manual are consistent with previous editions of this manual with the objective to improve transparency and efficiency in permitting and design of projects in Maryland. The updated procedures are based on the most recent hydrologic data available for decision making and guide the user toward the development of more reliable and consistent watershed models that better reflect the historic stream-gaging data for Maryland. This manual is to be used in conjunction with State and Federal technical manuals, computer user manuals, and technical papers.

We recognize that the technology of hydrology computer programs, geographic information systems software and databases, and remote sensing data collection are continually being advanced. Since the publication of the Fifth Edition of the Hydrology Panel report in July 2020, significant research has been published with respect to documenting the impacts of climate change. The Sixth Edition of the Panel report provides guidance on the types of projects for which future or projected precipitation should be used. These conditions include FEMA hurricane evacuation routes, areas with vulnerable assets that are susceptible to sea level rise, riverine flooding, scour or other disruptive flooding events, and areas with limited adaptive capacity that include roadways that are the only means of emergency vehicle access and would result in significant detour length if closed due to flooding. The use of the updated guidance will lead to more resilient projects and community protections, including to underserved, overburdened and flood-prone communities.

Although parts of this manual will be updated periodically to reflect current technology, data, and methods, it contains many guidelines, recommendations, limitations on procedures, analysis philosophies, and computational tools that will be valuable for the practitioner even under climate change due to global warming. It is important to note that this manual supports the hydrology practices and principles in use for Wetlands and Waterways Permits -- other units at MDE including Dam Safety and Sediment and Stormwater, and local approving authorities, may accept these methods or require other methods to be utilized for hydrology computations.

We appreciate the effort of all who participated in the preparation and review of this manual and pledge our commitment to the continual improvement of the science and applications of hydrology in the State of Maryland.

Very truly yours,

Matt Baker
Deputy Administrator for Project Development
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State Highway Administration

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EXECUTIVE SUMMARY

In June 1996, Maryland Department of Transportation State Highway Administration (MDOT SHA) and the Maryland Department of Environment (MDE) appointed the Maryland Hydrology Panel (the Panel) to explore the development of improved procedures that would ensure an optimal balance between preserving the environmental quality of Maryland streams and the hydraulic performance of highway drainage structures. The Panel: 1) worked closely with the staffs of the two Departments; 2) extensively reviewed Maryland policies and design approaches, as well as those of other States; and 3) conducted extensive research directed toward improving the statistical and deterministic foundations of the two Department's hydrologic modeling procedures.

In February 2001 the Panel issued a report entitled, ***Applications of Hydrologic Methods in Maryland***. Adoption of the recommendations of the February 2001 report led to significant cost and time savings in the design/review processes of the two Departments through better utilization of computer and human resources. Of even greater importance, the implementation of the recommendations increased confidence in the design computations.

As the staffs of the two Departments gained experience and confidence with the procedures recommended by the Panel in 2001, they came forward with numerous suggestions that would lead to even better approaches. The Panel reconvened in the Fall of 2002 and, following the suggestions of the two Departments' staffs, identified sufficient improvements to justify the publication of the second edition of the report in August 2006.

The third edition of the report entitled ***Application of Hydrologic Methods in Maryland*** is dated September 2010. The report was updated to include the Windows version of TR-20 (WinTR-20), revised temporal rainfall distributions based on NOAA Atlas 14, and revised versions of the Fixed Region regression equations for selected hydrologic regions in Maryland. The fourth edition, dated July 2016, was updated to include new regression equations for flood discharges for the Piedmont-Blue Ridge and Appalachian Plateau Regions, regression equations for estimating the 2- and 10-year 90- and 120-day low flows for fish passage and a new chapter on "Estimation of Discharges in Tidal Reaches".

The fifth edition of the report entitled ***Application of Hydrologic Methods in Maryland*** is dated July 2020. The fifth edition was updated to include revised regression equations for the Eastern and Western Coastal Plain Regions and minor revisions were also made to the regression equations for the Appalachian Plateau and Piedmont-Blue Ridge Regions that were published in the July 2016 version of the Panel report. In addition, regression equations were provided in the July 2020 report for estimating the 10-, 50- and 90-percent chance flow duration values for potential use in stream restoration projects and fish passage for culvert design for small streams. Guidance on access to the web server version of GISHydro was provided along with an example of calibrating WinTR-20 to the regression equations using the latest version of GISHydro. The Panel strongly believes that the procedures recommended in the present report, that have already been adopted by both Departments, position Maryland as a national leader in cooperation to ensure that the hydrologic requirements of highway drainage structures and the environmental protection of streams are met.

This sixth edition of the report entitled ***Application of Hydrologic Methods in Maryland*** is dated July 2023. The sixth edition was updated to provide guidance on the use of future or projected precipitation for certain types of MDOT SHA projects, to provide additional guidance on the use of low flows for fish passage in the design of culverts for small streams, and to document revised regression equations for the Eastern and Western Coastal Plain Regions. After the update of the Coastal Plain regression equations documented in the fifth edition (July 2020) of the Panel report, it was discovered that NRCS changed the default approach for aggregating the hydrologic soils groups. Regression analyses were performed that demonstrated the new SSURGO soils data were more accurate for predicting flood discharges. This edition of the report documents the 2022 regression equations for the two coastal plain regions. The only change from the 2020 equations is the use of percent A soils based on the updated SSURGO soils data.

Maryland requires highway drainage structures to pass the floods from watersheds under both existing land use conditions and conditions that can be anticipated when the watershed land use changes to a future "ultimate development" condition. With this edition of the report, the Hydrology Panel recommends that future or projected precipitation also be used for the final design for selected bridge sites as discussed in Chapter 1. The ability to accommodate future-conditions discharges must be met while providing minimal environmental impacts on the stream. The Panel recommends that the deterministic hydrologic model, WinTR-20, developed by the Natural Resource Conservation Service (NRCS) continue to serve as the base method for flood flow predictions. All deterministic hydrologic models, such as WinTR-20, require the estimation of a number of input parameters that are developed through field and map investigations. These parameters are difficult to estimate and research conducted by the Panel shows that these errors can cause significant problems. The Panel concluded that it was mandatory to provide guidance that would minimize the possibility of accepting errors in the WinTR-20 input parameters and, thereby, ensure that the flood flows predicted are within the bounds of floods expected in Maryland. Thus, the Panel presents statistical methods that can be used to calibrate the WinTR-20 model using long term stream gage records collected in Maryland by the U.S. Geological Survey and regression equations documented in this report. The Fixed Region regression equations, for which earlier versions were documented in the August 2006, September 2010, July 2016, and July 2020 versions of this report, were updated in 2022 for the Eastern and Western Coastal Plain Regions. The revised equations in Appendix 3 of this report are the recommended statistical methods for calibrating WinTR-20 for ungaged watersheds.

A key feature that ensures success is the Panel recommendation that requires the use of the software package GISHydro. MDOT SHA funding provided support for the development of GISHydro by the Department of Civil and Environmental Engineering at the University of Maryland. GISHydro provides the required hydrologic information by interfacing the recommended statistical and deterministic modeling procedures with a statewide land-soil-topographic database. Without GISHydro, the procedures recommended by the Panel would be too time and labor consuming to be implemented. With GISHydro, the approaches required by the Panel recommendations can be performed in a fraction of the time and with much more confidence and control than was possible with the traditional procedures of the late 1990's. Both Departments now use GISHydro. The confidence that the procedures are state-of-the-art and are being consistently applied has led to much shorter turn-around time in the design/review/approval process with significant cost savings.

GISHydro is available for production/project work on the Private Virtual Computer Lab at the University of Maryland. A web-based version of GISHydro is under development and evaluation and will be available on the MDOT SHA web servers in the future.

This document presents a set of hydrologic modeling procedures that are updated as technology changes and are designed to ensure an optimal balance between preserving the environmental quality of Maryland streams and the hydraulic performance and safety of highway structures. These procedures are recommended by the Maryland Hydrology Panel for use by the Maryland Department of Environment and the Maryland Department of Transportation State Highway Administration for all watersheds of approximately one square mile and larger. Experience has shown that the procedures are also applicable for watersheds smaller than one square mile.

THE MARYLAND HYDROLOGY PANEL
July 2023

This document presents a set of hydrologic modeling procedures that are designed to ensure an optimal balance between preserving the environmental quality of Maryland streams and the hydraulic performance and safety of highway structures. These procedures are recommended by the Maryland Hydrology Panel for use by the Maryland Department of Environment (MDE) and the Maryland Department of Transportation State Highway Administration (MDOT SHA).

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Dr. Arthur Miller of Pennsylvania State University and Mr. Michael Ports, then of Parsons, Brinkerhoff, served on the Panel through the publication of the February 2001 report. Dr. Robert Ragan, Professor Emeritus, University of Maryland, served on the Panel through the publication of the second edition of the report in August 2006. Donald Woodward, retired Natural Resources Conservation Service, served on the Panel through the publication of the third edition of the report in September 2010. Dr. Michael Casey, George Mason University, served on the Panel for the third edition of the report in September 2010. William Merkel, retired Natural Resources Conservation Service,

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served on the Panel for the publication of the second edition in August 2006 to the fifth edition of the report in July 2020.

Communication between the Panel and the primary user agencies, the MDE and the MDOT SHA, was critical to the successful development of a usable system. David Guignet, of MDE and Andrzej (“Andy”) J. Kosicki, PE, of MDOT SHA (retired as of Jan. 1, 2023) served as liaisons between their agencies and the Panel.

GLOSSARY OF EQUATION VARIABLES

Symbol	Definition	Units	Page of First Reference
A_G	drainage area of watershed determined using GIS methods	$miles^2$	3-3
A_M	drainage area of watershed determined manually from 1:24,000 scale maps	$miles^2$	3-3
A_{ac}	drainage area of the watershed	acre	3-6
A_{sf}	drainage area of the watershed	$feet^2$	3-10
A_{sm}	drainage area of the watershed	$miles^2$	3-6
Ag	drainage area at the gaging station	$miles^2$	2-10
As	surface area of a tidal basin at mean tide	$feet^2$	6-6
Au	drainage area at the ungaged location	$miles^2$	2-10
ARC	antecedent runoff condition (1 indicates dry, 2 indicates average, 3 indicates wet)	--	4-10
a_i	the ith increment of the watershed area	$miles^2$	3-9
A soils	percent of the drainage area that is classified as NRCS Hydrologic Soil Group A	percent	2-13
c	level of significance defining the prediction interval 100(1−c)% in Student's t distribution (e.g., 0.05)	--	2-11
DA	drainage area	$miles^2$	2-13
ΔD	duration of the unit excess rainfall	minutes	3-7
e	mathematical constant (Euler’s constant) equal to 2.718…	--	3-15
G	average skewness for a given hydrologic region	--	2-7
H	difference in elevation between high and low storm surge levels	feet	6-6
ho	leverage, expresses the distance of the site's explanatory variables from the center of the regressor hull	--	2-12
Ia, I_a	initial abstraction	inches	3-4
IA	percent of the drainage area that is impervious as determined using NRCS imperviousness coefficients and the Maryland Department of Planning land use data (IA > 10% is considered urban)	percent	2-13
i	precipitation intensity	inches/hour	3-12
K_X	the Pearson III frequency factor for recurrence interval, x and skewness, G	--	2-7
L	lag time, the time between the center of mass of the rainfall excess and the hydrograph peak	hours	3-9
L	overland flow length	feet	3-12
L_h	hydraulic length of the watershed	feet	3-10

Symbol	Definition	Units	Page of First Reference
LIME	percent of the drainage area that is underlain by carbonate rock (limestone and dolomite)	percent	2-13
LQw	logarithm of weighted peak discharge at a gaging station	log(feet^3/second)	2-6
LQg	logarithm of peak discharge at a gaging station based on observed data	log(feet^3/second)	2-6
LQr	logarithm of peak discharge computed from the appropriate Fixed Region regression equation	log(feet^3/second)	2-6
LSLOPE LSLOPE	average land slope calculated on a pixel by pixel basis	--	2-13
M	total length of the heavy line contours on a 1:24,000 topographic map	feet	3-10
n	number of gaging stations used in the analysis	--	2-11
N	contour interval between heavy line contours on a 1:24,000 topographic map	feet	3-10
n	Manning's roughness	--	3-12
Ng	years of record at the gaging station	years	2-6
Nr	equivalent years of record for the fixed region regression estimate	years	2-6
p	number of explanatory variables used in the Fixed Region regression equation	--	2-11
P	precipitation depth	inches	3-4
P_2	2-yr, 24-hour rainfall depth	inches	3-12
PD	population density	people/mile^2	4-21
P_{pp}	pp-percent flow duration percentile	feet^3/second	5-25
Q	runoff volume (expressed as depth)	inches	3-4
Qf	final estimate of the peak discharge at the ungaged site	feet^3/second	2-10
Qg	peak discharge at the gaging station based on observed data	feet^3/second	2-7
Q_i	the runoff from watershed area i	inches	3-9
Q_{max}	maximum discharge in a tidal cycle	feet^3/second	6-6
q_{oph}	peak discharge of the unit hydrograph	feet^3/second	3-6
$Q_{p\%}$	seasonal mean flow with a p percent annual chance of exceedance	feet^3/second	5-21
Qr	peak discharge computed from the appropriate Fixed Region equation	feet^3/second	2-9
Q_{T_D}	discharge for return period T [years] and duration D [days]	feet^3/second	5-1
Qu	regression estimate of peak discharge at ungaged location	feet^3/second	2-10
Qw	weighted peak discharge at the gaging station	feet^3/second	2-9
Qx	peak discharge for recurrence interval, x	feet^3/second	2-11

Symbol	Definition	Units	Page of First Reference
R	correlation coefficient	--	2-6
R	ratio of the weighted peak discharge (Qw) to the Fixed Region regression estimate (Qr)	--	2-9
RCN	runoff curve number	--	3-4
RF	areal reduction factor for precipitation	--	3-26
R_h	hydraulic radius	feet	3-12
Rw	scaled ratio for estimating peak discharge at ungaged site	--	2-10
S	potential maximum retention	inches	3-4
S	standard deviation of the logarithms of the annual peak discharges at the ungaged location	log(feet^3/second)	2-6
S	overland flow slope	feet/feet	3-12
SD	standard deviation of estimates of Manning's n for channel flow	--	3-15
SE	standard error of the estimate of flood discharge	log(feet^3/second)	2-11
SE	standard error of the low-flow regression equations	percent	5-1
SEp	standard error of prediction of the Fixed Region regression estimates in logarithmic units	log(feet^3/second)	2-6
T	the critical value of Student's t	--	2-11
T	tidal period (24 hr)	seconds	6-6
T_c	time of concentration of the watershed	hours	3-7
T_p	time to peak of the unit hydrograph	hours	3-6
T_t	travel time	minutes	3-12
T_{ti}	travel time from the center of area ai to the point of reference	hours	3-9
V	overland flow velocity	feet/second	3-12
xo	a row vector of the logarithms of the explanatory variables at a given site	log(various units)	2-12
$(\mathbf{X}^T\mathbf{X})^{-1}$	inverse of the covariance matrix of the regression parameters	log^2(various units)	2-12
Y	average watershed land slope	Percent	3-10

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CHAPTER ONE

1 Introduction

The Maryland Department of Transportation State Highway Administration (MDOT SHA) has been using deterministic models, primarily the WinTR-20 developed by the USDA-Natural Resources Conservation Service, to synthesize hydrographs and to estimate peak discharges for both existing and ultimate development conditions for some time. However, verifying that the WinTR-20 results for a watershed are representative of Maryland conditions is difficult. A report entitled "Analysis of the Role of Storm and Stream Network Parameters on the Performance of the SCS-TR-20 and HEC-1 Under Maryland Conditions," by Ragan and Pfefferkorn (1992), concluded that the WinTR-20 could produce good results, but it was quite sensitive to the values selected for input parameters including the Manning roughness coefficients, representative cross sections, curve numbers, storm structure and storm duration. If the WinTR-20 was to continue to be used, the MDOT SHA wanted guidance that would lead to more dependable performance and confidence that the results would be consistent with Maryland stream flow records.

The Water and Science Administration (WSA) of the Maryland Department of the Environment (MDE) has selected the WinTR-20 model or its equivalent as a standard deterministic method for computing flood flows in Maryland. However, the SHA wanted to make greater use of regional regression equations based on long term USGS stream gaging records. The WSA has been reluctant to accept a general use of regression equations for the following reasons:

- they do not account for ultimate development
- they do not reflect recent land use changes in some hydrologic regions, and
- they do not account for changes in storage and times of concentration.

These are valid concerns in Maryland because of the rapid changes in watershed characteristics being produced by urbanization. However, since regression equations use USGS stream gaging stations in the region for definition, they can provide a reasonable indication of existing runoff conditions and, therefore, can provide a base for calibration of the WinTR-20 or similar deterministic models. In addition, regression equations in the Western Coastal Plain Region and the combined Piedmont-Blue Ridge Region include impervious area as a predictor of land use change. The WSA requires that for a model to be considered for use in estimating flood peaks the model must meet the following conditions:

- Be in the public domain.

- Be generally accepted by the hydrologic community.
- Be verifiable.

Regional regression equations derived from USGS stream gaging stations meet all three of the above criteria. First, the regional regression equations developed for Maryland are in the public domain. Second, the regression methodology is widely used and recognized as acceptable by the hydrologic community. And third, the original data, regression methodologies, and the resulting equations are published and, therefore, readily verifiable.

Standard hydrologic practice strongly recommends that all deterministic models, such as the WinTR-20, be calibrated against local data. Where sufficient actual, measured rainfall and runoff data are available, the WinTR-20 model should be calibrated and, if possible, validated prior to its application. However, on-site rainfall and runoff data are rarely available in actual practice. In these more typical circumstances, regional regression equations developed from stream flow data may be used as a basis to “calibrate” the WinTR-20 model, providing the watershed conditions are consistent with those used to develop the equations.

Because of the need to improve confidence of the WinTR-20 model, the regional regression equation issues outlined above, and an array of other concerns being faced by the two organizations, the MDOT SHA and MDE WSA agreed to appoint a special hydrology panel. The Hydrology Panel (the Panel) was to be composed of professionals with extensive experience in Maryland who, at the same time, were nationally recognized for their substantial contributions to the practice of hydrology. Appointed in the fall of 1996, the Panel was chartered to operate independently of the MDOT SHA or other state agencies. The mission of the Panel was to:

> Review Maryland hydrologic practices and make recommendations concerning peak flood estimating procedures that will best serve to satisfy agency needs, Maryland laws and regulations.

Five versions of a report entitled, ***Application of Hydrologic Models in Maryland*** were published in February 2001, August 2006, September 2010, July 2016 and July 2020. Experiences with the application of recommendations presented in these reports, improvements in GIS technologies, guidance on climate change and estimation of low flows for fish passage and updates to the Maryland regional regression equations led to the publication of this sixth edition of the report. The following section presents the Panel’s recommendations. Subsequent chapters explain the basis for these recommendations and the procedures required for their accomplishment.

1.1 RECOMMENDATIONS

The Panel recommends the use of the software package, GISHydro and future upgrades, for hydrologic analysis in the State of Maryland. The current operational software is GISHydroNXT based on ArcGIS is available on the University of Maryland (UMD) Private Virtual Computer Lab. A web-based version of GISHydro (GISHydroWEB) is under development and evaluation and will be available on MDOT SHA servers in the future.

The terminology GISHydro is used throughout this report to refer to GISHydroNXT, GISHydroWEB or future updates. GISHydro includes delineation of the watershed boundaries, computation of the curve number and time of concentration, and direct interfaces with both the regression equations and the WinTR-20 model. Use of this software ensures reproducibility of watershed characteristics based on the topographic, land cover, and soil databases that are integral to GISHydro. Automated reporting that is built into GISHydro allows reviewers at MDE to independently confirm analyses submitted for their review. Consistency in analysis presentation also helps to streamline the review process.

Information on how to access and use GISHydro is available at the following UMD website:

http://www.gishydro.eng.umd.edu.

1.1.1 Overview of the Modeling Process and the Calibration Requirements

The hydrologic analysis of Maryland State Highway Administration for bridges and culverts must evaluate the behavior of the structure and local stream under both existing and ultimate development watershed conditions. Because two land cover and flow path conditions are involved, the basis for these hydrologic analyses must be a deterministic model that can simulate the runoff processes that occur during and after the storm. The deterministic model is the WinTR-20 model or an approved equivalent. The recommended first step is to calibrate the deterministic model using field and map defined input parameters so that the model adequately describes the runoff processes under existing watershed conditions. After the designer is satisfied that the calibrated deterministic model provides a realistic representation of the existing watershed conditions, the impact of ultimate development will be simulated by adjusting the input parameters to reflect the planned land cover and flow path modifications. With the edition of this sixth edition report, the Hydrology Panel also recommends that future precipitation be used in the final design of certain MDOT SHA projects.

Hydrologic analyses for all watersheds will be supported by field investigations and the design discharges will be determined utilizing two hydrologic models: (1) a probabilistic method based on a local USGS gaging station or approved regression equations that are developed through statistical analyses of USGS stream gage records (Chapter 2); and (2) a flood hydrograph deterministic procedure such as the WinTR-20 or its equivalent. The objective is to use the probabilistic method based on long-term stream gage records to ensure that the WinTR-20 produces peak discharges that are consistent with Maryland conditions. As described in Chapters 3 and 4 of this report, the sensitivity of the WinTR-20 model input parameters and the uncertainties associated with the selection of these parameters are such that calibration against USGS historical data is mandatory. The calibration methodology will be utilized in the following order of priority to determine peak flow:

1. Use a stream gage located at the site with the frequency curve of record being weighted with the regional regression estimates, following the approach presented by Dillow (1996) and described in Chapter 2. Report the discharges as the weighted estimate and an error bound of plus one standard error of prediction. Develop the stream gage frequency curves following the procedures in the Interagency Advisory Committee on Water Information Bulletin 17C "Guidelines for Determining Flood Flow Frequency – Bulletin 17C" (England and others, 2019). Bulletin 17C is the standard reference used by Federal agencies and most state and local agencies for the estimation of flood flow frequency curves for gaged watersheds in the United States.

2. If there is no gage at the site, but there is a gage on the same stream that can be transposed, (the gage's data can be transposed ± half the gaged area upstream or downstream), transpose the gaged record to the site following the approach recommended by Dillow (1996) and described in Chapter 2. Report the discharges as the estimate and an error bound of plus one standard error of prediction.

3. If there is no gage on the stream and the watershed characteristics are within the bounds of those used to derive the approved regional regression equations, apply the regression equations to the watershed. Report the discharges as the regression equation estimate and an error bound of plus one standard error of prediction.

The region between the "best estimate line" of the regional regression equations and the upper bound of plus one standard error of prediction will be defined as the "**calibration window**" for the purposes of these recommendations.

If the peak discharge of the hydrograph synthesized for the design storm is within the calibration window, the analysis will be accepted as a reasonable representation of the runoff for existing watershed conditions, providing that the WinTR-20 input parameters are within the bounds of sound hydrologic practice. The model then forms the basis for simulating the watershed under ultimate development conditions.

If the peak discharge estimated by the deterministic model is outside the calibration window, additional investigations and simulations will be conducted to determine:

1. Are the watershed conditions consistent with those of the USGS stream gages used to develop the approved regional regression equations?

2. Are the regional regression equations appropriate for use on this watershed?

3. Even though the averaged watershed characteristics are consistent with those of the USGS stream gages used to develop the regression equations, are there specific conditions such as extensive stream valley wetlands, a deeply incised channel or other factors that would cause unusually low or high peak discharges?

4. Are the deterministic model parameters defining the curve number, time of concentration and storage attenuation appropriate for the field conditions being simulated? If not, they can be adjusted in accordance with Chapter 4. Some parameter adjustment is allowed because the WinTR-20 is quite sensitive to the assigned values and it is very difficult to select quantities that best represent the watershed conditions. Any adjustments must be justified with supporting documentation and **MUST BE WITHIN THE BOUNDS OF SOUND HYDROLOGIC PRACTICE**.

If there is no term in the regional regression equations that reflects the degree of urbanization and the watershed is greater than 10% impervious, then the WinTR-20 calibration process for existing conditions will be a two-step process. First, the designer will estimate the pre-developed land cover distribution and calibrate to the regression equations for this pre-developed condition. These WinTR-20 discharges will then be adjusted by revising the input parameters to reflect the increased curve numbers and the drainage network of the existing condition. The process is described in section 4.6 of this report. The Panel believes that the uncertainties associated with a "pre-developed calibration" are less than those associated with an approach that requires the designer to select WinTR-20 input parameters without any opportunity for calibration.

If the WinTR-20 peak discharges do not fall within the calibration window of the regression equations, the designer should explain why the existing watershed conditions are significantly different from those defining the regression equations or why the WinTR-20 model is not applicable to this particular watershed. The designer will then select and justify the most appropriate method for the specific watershed.

The focus of the Panel's efforts was the development of procedures for use on watersheds having drainage areas larger than one square mile. Experience on SHA projects has shown that GISHydro and the calibration procedures using the regression equations can be applied on watersheds less than one square mile. When applying the procedures on basins smaller than one square mile, the user must be especially careful to ensure that the watershed boundary generated by GISHydro is consistent with that indicated by the USGS 1:24000 Topographic Maps or more detailed topographic maps. GISHydro develops the watershed boundary from USGS digital elevation data spaced on a 10- or 30-meter grid. As the watershed area becomes smaller, the number of elevation points used by GISHydro to generate the boundary decreases. The consequence is an increasing risk that the boundary generated by the computer delineation may differ from that indicated by topographic maps.

An example of when it might not be possible to get the WinTR-20 peak discharges to fall within the calibration window of the regression is in the Piedmont-Blue Ridge physiographic region. In this region, the area of limestone geology is a predictor variable in the Fixed Region regression equations. The area of limestone geology was compiled from geologic mapping from several sources and is not known with precision. A slight shifting of the limestone geology boundary could significantly change the estimated percentage of limestone in a watershed with boundaries intersecting both limestone and non-limestone areas. The uncertainty in estimated limestone geology becomes larger as the overall watershed area gets smaller. Errors and uncertainty in percent limestone geology can have a significant effect on the resultant flood discharges estimated by the Fixed Region regression equations. Because of the uncertainty associated with the determination of limestone geology, the WinTR-20 model estimates should NOT be calibrated to the Fixed Region regression equations for watersheds when there is a significant percentage of limestone (greater than 50 percent) in the watershed. For these watersheds, the Fixed Region regression equations or gaging station data should be used as described below.

Based on comparisons to gaging station data, the WinTR-20 estimates can be very conservative when the percentage of limestone area exceeds 50 percent of the drainage area. If there is a gaging station near the watershed outlet (within 50 percent of the drainage area of the watershed being studied) and the percentage of limestone in the watershed is greater than 50 percent, the analyst should use a weighted average of the gaging station estimates and the Fixed Region regression estimates for existing development conditions following the approach described later in Section 2.3, Estimates for Ungaged Sites near a Gaging Station. If there is no gaging station nearby, then the analyst should use the Fixed Region regression estimates for existing conditions. In each instance, the flood discharges for existing conditions should be adjusted for ultimate development based on the ratio of uncalibrated WinTR-20 flood discharges for the ultimate development and existing development conditions.

1.1.2 Issues Concerning the Selection of WinTR-20 Input Parameters

The first step is to use map and field investigations to select input parameters that are consistent with established hydrologic practice and give a reasonable simulation of existing watershed conditions. If inputs give results that are outside the calibration window, the designer will review the parameters used as inputs to define the WinTR-20 simulation. If the review indicates that a parameter may be incorrect, additional field and map investigations will be used to support any corrections. **In no instance will WinTR-20 inputs be accepted that are outside the bounds of standard hydrologic practice.**

Before attempting to revise input parameters in a WinTR-20 calibration against one of the three approaches listed in Section 1.1.1, the designer should carefully study Chapter 3 of the present report and MD-SHA AWO92-351-046, “Analysis of the role of storm and stream parameters on the performance of SCS-TR-20 and HEC-1 under Maryland Conditions” (Ragan and Pfefferkorn, 1992).

Normally, watersheds having drainage areas larger than one square mile will be delineated using the digital terrain modeling capabilities of GISHydro or manually on 1:24,000 USGS quad sheets or more detailed maps. Special care must be taken in locating the ridgeline on the eastern shore or in other areas of low relief. The designer should always perform a map check of the automatic boundary delineation of GISHydro that uses 10- or 30-meter resolution USGS digital terrain data.

The WinTR-20 model will be run using the latest precipitation-frequency information from NOAA Atlas 14, Volume 2 (Bonnin and others, 2006) and center-peaking NRCS hyetographs based on NOAA Atlas 14 as design storms. The precipitation depths of these design storms will be defined from the NOAA Atlas 14 web site. The Panel recognizes that changes in the duration and/or structure of the design storm used as an input to the WinTR-20 produces significant changes in the magnitude of the peak discharge and shape of the runoff hydrograph. More research is needed to finalize a synthetic storm structure and duration to be used for specific frequencies and locations in Maryland. Until new research on storm structure is complete, the designer should use design storms developed in WinTR-20 or GISHydro from NOAA Atlas 14 data. Twelve- and 6-hour durations may be developed from data contained in the 24-hour storm distribution. Table 1-1 shows the acceptable storm durations that may be used to calibrate the WinTR-20 model and develop final design peak discharges. The storm duration selection is based on the total time-of-concentration (T_c) to the point of study. In general, the duration of the design storm should in no case be less than the total T_c of the watershed.

If the WinTR-20 discharges are not within the calibration window after adjustment for the time of concentration, runoff curve number, storm durations and other input data, then the analyst should evaluate the use the upper 90-percent confidence limit of the precipitation depth as provided in Atlas 14. The upper 90-percent confidence limits from Atlas 14 are now available for use in GISHydro and more guidance is provided in Chapter 4.

Table 1-1: Acceptable Storm Durations (hrs) for Total Watershed T_C

Time of Concentration	Return Period						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	>100-yr
< 6 hrs	6/12/24	6/12/24	6/12/24	12/24	12/24	12/24	24
6-12 hrs	12/24	12/24	12/24	12/24	12/24	12/24	24
12-24 hrs	24	24	24	24	24	24	24
> 24 hrs	24*/48	24*/48	24*/48	24*/48	24*/48	24*/48	24*/48

*If T_C is less than 36 hours, the engineer may choose the 24-hour duration

An example of development of 6-hour and 12-hour duration design storms for Howard County, Maryland is presented in Appendix 7. A spreadsheet was developed to calculate the 6, 12, and 24-hour storm distributions for locations within Maryland. In all instances, the hyetograph time increment, Δt, shall not exceed 6 minutes (0.1 hour).

Intensity-Duration-Frequency (IDF) curves are developed from point measurements. The spatial distribution of rainfall within a storm generally produces an average depth over an area that is a function of watershed area and storm duration. Figure 3.10, reproduced from USWB-TP-40, illustrates this phenomenon. The Panel recommends that the hydrologist adjust the design storm rainfall to reflect spatial distribution. If the hydrologist is using GISHydro, the adjustment is automatic. If the hydrologist is conducting a study outside the GISHydro environment, the adjustment for spatial distribution should be made using the graph in Figure 3-10.

The NRCS presents runoff curve numbers for many hydrologic soil-cover complexes as a range covering “good”, “fair” and “poor” — conditions that may be difficult to determine. Also, as discussed in Chapter 3, the assumption that $I_a = 0.2S$ is fundamental in the calculation of runoff volume in terms of a Runoff Curve Number (RCN). Figure 10-1 of USDA-NRCS-NEH, Part 630, Chapter 10, (2004) presented in this report as Figure 3-2, shows that there is significant scatter in the data used to support the assumption that $I_a = 0.2S$. Thus, the Panel recommends that the designer be granted a reasonable degree of latitude in the selection of RCN values for individual land parcels during the calibration process providing the values remain within the range recommended by NRCS and the decision is justified in writing. Adjustments must be made on a parcel-by-parcel basis and cannot be made by simply changing the overall watershed RCN.

The commonly used peak rate factor of 484 in NRCS dimensionless unit hydrograph (DUH) is known to vary for different terrain. For streams in the Eastern and Western

Coastal Plain Regions, a peak rate factor of 284 is recommended. The peak rate factor of 284 was determined to be applicable to the flatter watersheds in the Eastern Coastal Plain Region. However, some watersheds in the Western Coastal Plain Region have watershed characteristics that deviate significantly from the Eastern Coastal Plain streams. For those watersheds, a peak rate factor of 484 is more appropriate. The designer will use the peak rate factors as shown in Table 3-1.

The NRCS lag equation to estimate the time of concentration should not be used on watersheds having drainage areas in excess of five square miles. The hydraulic length in the equation should be longer than 800 feet because shorter lengths result in artificially short lag times.

The lag equation is not included as a recommended procedure in USDA-NRCS, WinTR-55, “Small Watershed Hydrology” (2009). Thus, the Panel recommends that the lag equation not be used in urban ($\geq$ 10 percent impervious) watersheds until additional research becomes available. It should be noted that the lag equation was developed using data from agricultural watersheds.

The Panel recommends that the velocity approach of NRCS be used to estimate the time of concentration in urban and suburban watersheds. A regression equation described in Appendix 6 can be used to evaluate the reasonableness of the time of concentration estimate by the velocity method. The NRCS velocity approach is based on estimating the travel times of the three segments of flow – overland, shallow concentrated, and open channel – through the watershed. The NRCS kinematic wave equation should be used to estimate time of overland flow travel with a maximum flow length of 100 feet. Because the quantity of flow and, therefore, the hydraulics are different for each storm frequency it is logical to expect that the time of concentration will be different for a 2-year storm than for a 100-year storm. The Panel recommends that bankfull conditions that many consider to approximate the 2-year storm conditions be used to estimate the time of travel through the main channel.

Use GISHydro or 1:24,000 scale USGS 7.5-minute quadrangle sheets or more detailed topographic maps to estimate channel length. It is recognized that this scale cannot adequately represent meanders and, therefore, estimated length may be too short and slope too steep. When field investigations or more detailed maps indicate that such is the case, the designer may increase the estimated length, providing the increase is justified in writing.

It is difficult to estimate the correct Manning roughness coefficient. Variations in the estimate of the Manning roughness can produce significant changes in the time of concentration and, therefore, the estimated peak discharge. The designer should exercise extreme care in estimating the main channel roughness and use discharge comparisons with the regression equations to improve the estimates.

As stated earlier, velocities at “bankfull” conditions are to be used in estimating the time of travel through the main channel. Selection of the representative bankfull hydraulic radius is difficult because the bankfull cross-section varies along the length of the channel. A “best estimate” can be made using field and map investigations and then brought into agreement with the calibration window through corrections justified by additional field and/or map investigations.

When the watershed is divided into sub-basins, the routing cross sections and the channel and overbank roughness coefficients are difficult to estimate and can have a significant impact on the attenuation simulated by the routing procedure. The hydrologist must select a routing cross section that is representative of the overall channel length. The digital terrain modeling capabilities of GISHydro provide a rapid way to explore the variations of cross sections along the channel.

In situations where errors can result in loss of life or major economic damage, routing cross sections should be developed through detailed mapping along the stream.

When the economics of a project do not justify detailed surveys along the length of a stream, reasonable modeling results can be produced with:

- Bankfull cross sections developed from regional regression equations that relate channel depth and width to the drainage area above the cross section; (Equations for use in Maryland are presented in Appendix 4.)
- Routing sections developed by drawing perpendicular transects to the channel across the contours, as is the approach followed by GISHydro;

Regression equation and map transect estimates of cross sections should be supported by field investigations to ensure that the sections are realistic for the watershed involved.

If there are culverts or other storage producing structures along the stream, the attenuation should be reflected in the inputs to the WinTR-20.

Where available, comprehensive planning maps, as opposed to zoning maps, should be used to predict future land cover. The planning maps incorporate key elements of time and spatial distribution that are not apparent on zoning maps.

1.2 RATIONALE

1. Each watershed will be analyzed by two widely accepted approaches, one statistical (local gage or regional regression equations) and one deterministic (WinTR-20 or equivalent). In the past, the effort associated with such an approach would have been prohibitive. With the current capabilities of GISHydro that includes delineation of the watershed boundaries, curve number computation and direct interfaces with both the regression equations and WinTR-20, the tasks can

be performed in considerably less time than was required by conventional techniques.

2. Studies have shown that uncalibrated WinTR-20 models often predict peak discharges that are not consistent with the peak discharges that have been observed at Maryland stream gages. A major contributor to this problem is the fact that it is very difficult to select the curve number, the Manning roughness coefficients and the “typical” cross sections that represent the watershed conditions. Small errors in the selection of these parameters can lead to incorrect estimates of the volume of runoff, time of concentration, storage attenuation and, therefore, lead to peak flow predictions that are too high or too low. Calibration against a USGS gage, or regression equations that are based on statistical analyses of USGS stream gages, can aid the designer in the selection of appropriate hydrograph input parameters that will usually produce estimated peaks that are consistent with Maryland conditions. The calibration will also provide confidence that the WinTR-20 is not over predicting to cause unnecessary construction costs and not under predicting to cause unnecessary flooding risks.

3. The recommended procedures are consistent with accepted practice, especially with AASHTO (1991) that states, “What needs to be emphasized is the need to calibrate to local conditions. This calibration process can result in much more accurate and consistent estimates of peak flows and hydrographs… Should it be necessary to use unreasonable values for variables in order for the model to produce reasonable results, the model should be considered suspect and its use carefully considered.” An example of an inappropriate use of the WinTR-20 would be to use an NRCS dimensionless hydrograph peak factor of 484 on the Eastern Shore of Maryland where the recommended peak factor is 284.

4. The recommended procedure is to make use of the USGS stream flow records or regional regression equations as the cornerstone for calibrating the hydrograph model. Using the statistics-based methods ensures that the deterministic model provides a realistic representation of existing watershed conditions. Once confident that the WinTR-20 model represents the existing conditions, the designer can vary the input parameters to simulate changes in the land cover and drainage network associated with ultimate development and be fairly confident in the final results.

5. It is not the intent of this report to recommend that the calibration of the deterministic model be accomplished at the upper bound of the calibration window. Rather, the prediction limits can be used to provide an indication of the level of uncertainty associated with the discharge selected. Assuming that the regional regression equation estimates are unbiased, 50% percent of the peaks measured on watersheds having these characteristics will be higher and 50% will be lower than the expected value. Approximately 68% of the peak discharges will

fall between plus and minus one standard error of the expected value. Thus, there is an approximately 84% chance that the peak discharge for this type of watershed will not exceed that indicated by the upper bound. Similarly, there is an 84% chance that a measured peak flow for this type of watershed will be greater than that indicated by the lower bound. **For purposes of "calibrating" the WinTR-20 model, the model parameters can be adjusted, within the bounds of sound hydrologic practice, so the estimated flood discharge falls within a calibration window defined by the regression estimate (expected value) and the upper bound of plus one standard error of prediction.**

1.3 NEED FOR CONTINUING RESEARCH

As described in Chapter 7 of this report, many areas of hydrology require additional research if we are to improve our confidence in the modeling process. It is imperative that a continuing, well-conceived and adequately funded research program be implemented to address a number of problems, especially,

1. Improving the structure and duration of the design storms;

2. Using the time-area curve available from the digital terrain data to generate geomorphic unit hydrographs that are unique for the watershed being modeled;

3. Until procedures for the future use of geomorphic unit hydrographs can be implemented, research must continue on the regionalized peak factors to be used with the NRCS dimensionless unit hydrograph;

4. Improving methods for estimating times of concentration in rural and urban watersheds;

5. Improving procedures for peak discharge transposition of gaging station data to ungaged locations near the gaging station;

6. Developing improved regression equations for incorporating land use change for estimating the 2- to 500-year peak discharges for rural and urban streams;

7. Developing guidelines for estimating NRCS runoff curve number and impervious area from information on planning and zoning maps;

8. Developing improved guidelines for selecting concurrent return periods for storm surge and riverine peak discharge;

9. Planning for climate change in terms of both tidal-influenced systems affected by sea level rise and riverine systems where precipitation intensity-duration-frequency (IDF) is anticipated to change.

1.4 CLIMATE CHANGE GUIDANCE

The hydrologic procedures described in the Hydrology Panel report incorporate the use of ultimate land development for estimation of the final design discharges. The historical precipitation from NOAA Atlas 14, Volume 2, based on data through 2000, is used in the TR-20 model along with the ultimate land development for estimation of the final design discharges. Areas of Maryland have experienced some extreme events since 2000 and it is possible that NOAA Atlas 14, Volume 2, may be underestimating current precipitation in Maryland. For that reason, the National Weather Service is currently working on an update of Atlas 14, Volume 2, that includes the states of Maryland, Delaware, Pennsylvania, Virginia, North Carolina and South Carolina that will become NOAA Atlas 14, Volume 13. Development of Volume 13 will take a few years but will likely provide increased estimates of precipitation depth for a given frequency.

With likely increases in future precipitation, **the Hydrology Panel recommends that MDOT-SHA use future precipitation estimates from Global Climate Models for design of selected projects as described below.** Considerable research on climate change is available since the publication of the Fifth Edition of the Hydrology Panel report in 2020. For example, Brubaker and others (2022) have developed procedures for estimating future IDF curves for Maryland by adjusting estimates from Atlas 14, Volume 2 but these procedures have not been peer reviewed and implemented in any operational guidance.

In 2021 a consortium including Rand Corporation, Carnegie Mellon University and the Northeastern Regional Climate Center at Cornell University developed a web tool for estimating future precipitation for the Chesapeake Bay Watershed (https://midatlantic-idf.rcc-acis.org/). This work was funded by NOAA through their Mid-Atlantic Regional Integrated Sciences and Assessments (MARISA) program. The development of this web site and the data used in the analysis are described in a Rand Corporation report that is available at (https://www.rand.org/content/dam/rand/pubs/tools/TLA1300/TLA1365-1/RAND_TLA1365-1.pdf).

The MARISA web tool is readily available and provides ratios for all counties in the Chesapeake Bay watershed (including all counties in Maryland) that are used to increase precipitation depths from NOAA Atlas 14, Volume 2. Specifically, the MARISA web site provides ratios for:

- The 2-, 5-, 10-, 25-, 50- and 100-year precipitation depths (100-year ratio used for 500-year event),
- Emission scenarios for RCP 4.5 (low) and RCP 8.5 (high),
- Two time periods: 2020-2070 and 2050-2100, and
- Ratios for the 10th, 25th, median, 75th, and 90th percentiles (exhibiting the uncertainty in the ratios).

The Hydrology Panel recommends that MDOT SHA use the median ratio for the high emission scenario RCP 8.5 for the time period 2050-2100. The median ratio for the RCP 8.5 scenario will be incorporated in GISHydro as a future precipitation layer for easy access. Future precipitation should NOT be used in calibrating the WIN TR-20 model. Future precipitation is recommended for the final design discharges for the following types of projects in Maryland:

- FEMA hurricane evacuation routes,
- Areas with vulnerable assets that are susceptible to sea level rise, riverine flooding, scour or other disruptive flooding events, and
- Areas with limited adaptive capacity that include roadways that are the only means of emergency vehicle assess and would result in significant detour length if closed due to flooding.

The purpose and need for a given drainage structure is defined in the project's objective statement that is established early in the planning phase. The vulnerabilities at the site and quantitative performance measures that require the drainage infrastructure to provide a certain service life or target a specific flooding problem is defined before the hydrologic analyses begin. The decision to require future precipitation is determined during the early planning process. Meeting the performance measures may require replacing and redesigning an existing drainage infrastructure using future precipitation data. However, a recently built structure would not be replaced or redesigned as this would not be cost effective.

The median ratios for the RCP 8.5 scenario vary from 1.13 to 1.21 across recurrence intervals (2-100 years) and counties in Maryland in the MARISA dataset. These ratios are applied for all durations of precipitation implying the NOAA Atlas 14 historical precipitation will be increased on average from 13 to 21 percent. The RCP 8.5 scenario was chosen because the ratios are more consistent across recurrence interval events and counties in Maryland than the RCP 4.5 scenario. The use of future precipitation for critical sites will provide more resilient infrastructure design for the remainder of this century.

Another aspect of climate change is sea level rise. There are several sources of future projections of sea level rise for Maryland. The most recent report for Maryland is entitled "Sea Level Rise: Projections for Maryland 2018" (Boesch and others, 2018). The most likely range (66 percent chance) of sea level rise as defined by Boesch and others (2018) is:

- 0.8 to 1.6 feet by 2050, and
- 2.0 to 4.2 feet by 2100 if gas emissions continue to increase unabated.

Boesch and others (2018) also indicate there is about a 5-percent chance that sea level rise will exceed 2.0 feet by 2050 and exceed 5.2 feet by 2100. However, elevation of coastal roads and structures to accommodate sea level rise is not currently part of MDOT SHA operational plans because of prohibitive costs.

CHAPTER TWO

2 Statistical Methods for Estimating Flood Discharges

2.1 INTRODUCTION

The Maryland Department of Transportation State Highway Administration (MDOT SHA) has a long history of using statistical methods for estimating flood discharges for the design of culverts and bridges in Maryland. MDOT SHA has funded several regional regression studies over the last 40 years: Carpenter (1980), Dillow (1996), Moglen and others (2006), the revised regression equations documented in Appendix 3 of the September 2010 version of the Hydrology Panel report, revised regression equations for the Piedmont-Blue Ridge and Appalachian Plateau Regions as documented in Thomas and Moglen (2016), and revised regression equations for the Eastern Coastal Plain Region (Thomas and Sanchez-Claros, 2019a) and for the Western Coastal Plain Region (Thomas and Sanchez-Claros, 2019b). In this Sixth Edition of the Hydrology Panel report, MDOT SHA updated the regression equations for the Eastern Coastal Plain Region and for the Western Coastal Plain Region using an updated version of the SSURGO soils data.

Carpenter (1980) developed regression equations for three hydrologic regions (North, South and Eastern) in Maryland by relating flood discharges based on Bulletin 17A (U.S. Water Resources Council, 1977) at 225 rural gaging stations (114 in nearby states) to watershed and climatic characteristics. Carpenter (1980) also used short-term rainfall-runoff data collected at eight small stream sites to calibrate a watershed model and simulate annual peak discharges at these stations using long-term rainfall data. The simulated annual peak discharges were analyzed using Bulletin 17A guidelines to estimate the design flood discharges at each station. For 17 other small stream stations in the Appalachian Plateau and Piedmont Regions with only observed data for the period 1965-76, Carpenter adjusted the flood discharges based on comparisons to nearby long-term stations to be more representative of a longer period of record.

Dillow (1996) developed regression equations for five hydrologic regions in Maryland (Appalachian Plateau, Blue Ridge, Piedmont, Western and Eastern Coastal Plains, see Figure 2-1). Dillow's study superseded the study by Carpenter (1980). Dillow (1996) used flood discharges based on Bulletin 17B estimates (Interagency Advisory Committee on Water Data [IACWD], 1982) at 219 rural gaging stations (112 in nearby states) in developing his regression equations. Dillow (1996) also utilized the rainfall-runoff estimates for the small watersheds that were developed by Carpenter (1980). He chose not to use Carpenter's (1980) adjusted design discharges for the small watersheds with observed data for the period 1965-76 but used design discharges based on the observed short-term record.

Moglen and others (2006) evaluated three approaches for regional flood frequency analysis using data for rural and urban (≥10% impervious) gaging stations: the Fixed Region approach, the Region of Influence method (Burn, 1990) and regional equations based on L-Moments (Hosking and Wallis, 1997). The Fixed Region approach is analogous to the approach taken by Carpenter (1980) and Dillow (1996) where regression equations are developed for a fixed geographic region and are based on Bulletin 17B estimates at the gaged sites. For the Region of Influence approach, regression equations are based on gaging stations that have the most similar watershed characteristics to the ungaged site of interest. There are no geographic flood regions and the regression equations are different for each ungaged site. For the gaged sites, flood discharges based on Bulletin 17B guidelines were used in the Region of Influence analysis. The L-Moment approach (Hosking and Wallis, 1997) uses linear moments, a linear combination of the untransformed annual peak discharges (not the logarithms), to estimate the parameters of the frequency distribution. Several frequency distributions can be used in the L-Moment approach, but the Generalized Extreme Value distribution was shown to be most appropriate for Maryland streams. For estimation at an ungaged site, the L-Moment approach is analogous to an index flood approach where the mean annual flood is estimated from a regression equation based on watershed characteristics and design discharges such as the 100-year discharge, are estimated as a ratio to the mean annual flood.

Carpenter (1980) and Dillow (1996) used the generalized skew maps in Bulletins 17A and 17B (same map) in developing the weighted skew estimates to define the design discharges at the gaging stations. Moglen and others (2006) developed new estimates of generalized skew as described later and illustrated that these estimates of generalized skew were more accurate than those from the Bulletin 17B map.

Moglen and others (2006) compared estimates of flood discharges from the Fixed Region, Region of Influence, and L-Moment methods to Bulletin 17B estimates at the gaged sites and determined that the Fixed Region approach was most accurate. The Fixed Region approach uses the five hydrologic regions shown in Figure 2-1; separate rural and urban equations were developed for the Piedmont Region (a total of six sets of equations). The Fixed Region Regression Equations (FRRE) developed by Moglen and others (2006) were included in Appendix 3 of the August 2006 version of the Hydrology Panel report.

For the September 2010 version of the Hydrology Panel report (Third Edition), the FRRE were revised for the Eastern and Western Coastal Plain regions using recently released SSURGO soils data. In addition, the rural gaging stations in the Piedmont and Blue Ridge Regions (see Figure 2-1) were combined to better define the region influenced by karst geology. The regression equations for the urban watersheds in the Piedmont Region and the regression equations for the Appalachian Plateau were not revised. The FRRE are described in Appendix 3 of the September 2010 version of the Hydrology Panel report.

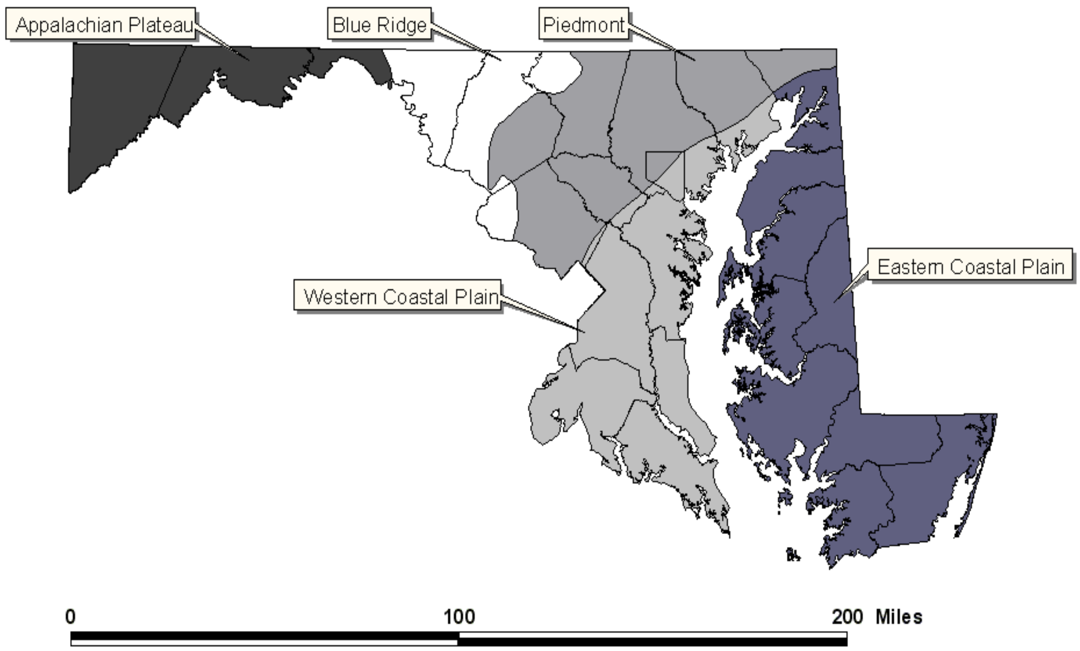

Figure 2-1: Hydrologic Regions Defined by Dillow (1996) and Used by Moglen and Others (2006)

Thomas and Moglen (2016) developed revised regression equations for the Piedmont, Blue Ridge and Appalachian Plateau Regions based on annual peak flow data through the 2012 water year and Bulletin 17B guidelines (IACWD, 1982). The Piedmont-Blue Ridge Regions were combined into one region with one set of regression equations based on data for 96 gaging stations for both rural and urban watersheds. The regression equations were also updated for the Appalachian Plateau Region using data for 24 rural gaging stations. The hydrologic regions used by Thomas and Moglen (2016) are shown in Figure 2-2. The revised regression equations from Thomas and Moglen (2016) are described in detail in Appendix 3 of the July 2016 Hydrology Panel report. For the Fifth Edition of the Hydrology Panel report, the regression equations developed by Thomas and Moglen (2016) were revised as described in Appendix 3. For the Appalachian Plateau Region, land slope based on DEM data dated May 2018 were used in lieu of legacy DEM data used in the original development of the equations. For the Piedmont-Blue Ridge Region, the flood frequency estimates for small rural watersheds were adjusted to account for time sampling error following procedures described by Carpenter (1980) prior to development of the revised regression equations.

Thomas and Sanchez-Claros (2019a) developed revised regression equations for the Eastern Coastal Plain Region based on annual peak flow data through the 2017 water year and Bulletin 17C guidelines (England and others, 2019). The regression equations were based on 36 rural gaging stations in Maryland and Delaware. Thomas and Sanchez-Claros (2019b) developed revised regression equations for the Western Coastal Plain Region based on annual peak data through the 2017 water year and Bulletin 17C guidelines (England and others, 2019). The regression equations were based on 23 rural and urban gaging stations in Maryland.

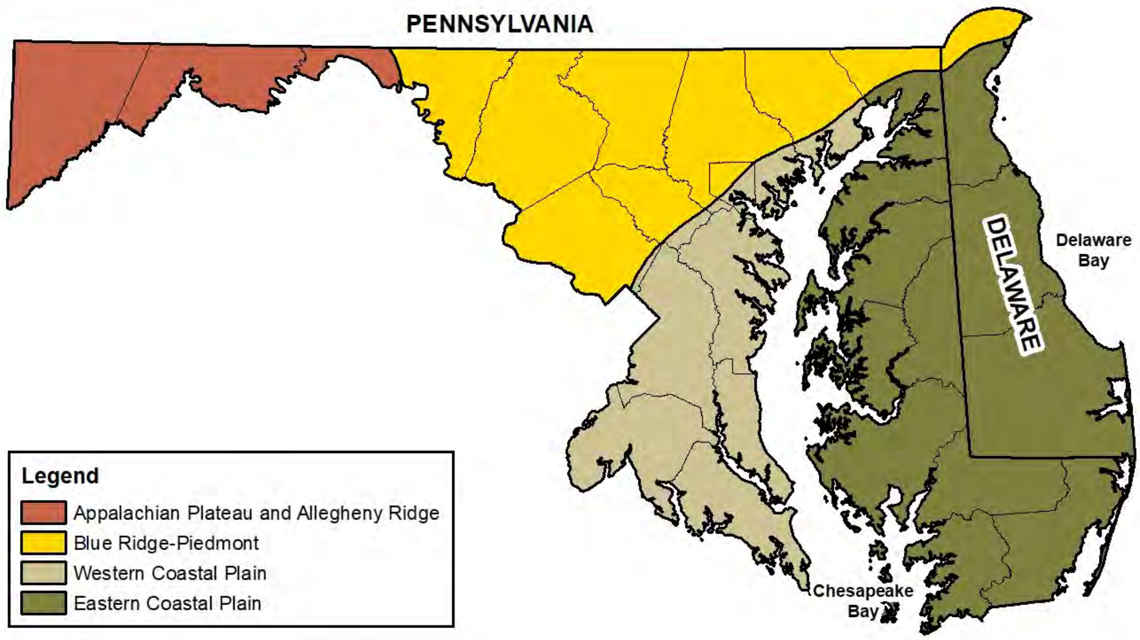

Figure 2-2: Hydrologic Regions Used by Thomas and Moglen (2016), Thomas and Sanchez-Claros (2019a) and Thomas and Sanchez-Claros (2019b)

For this Sixth Edition of the Hydrology Panel report, the regression equations for the Eastern and Coastal Plain Regions were revised using updated SSURGO soils data. The only change in the regression equations was the percent A soils. The annual peak data were still based on data through the 2017 water year. Details on the revised regression equations are given in Appendix 3.

The physiographic regions shown in Figure 2-1 and Figure 2-2 appear as crisp lines separating one region from another, and thus one set of regression equations from another. Engineers should exercise caution when analyzing watersheds near these physiographic boundaries. For instance, the Fall Line, which separates the Piedmont from the Western Coastal Plain region, is more appropriately considered a region of some width, rather than a crisp line. Within this area close to physiographic region boundaries it is possible for a watershed that is strictly located within one region to exhibit flood behavior more consistent with the neighboring physiographic region. In GISHydro, the software automatically detects if the watershed comes within 5 km of the physiographic boundary and prints a warning if this is the case.

2.2 FLOOD DISCHARGES AT GAGING STATIONS

Estimates of design discharges, such as the 100-year flood discharge, are made at gaging stations where there are at least 10 years of annual peak discharges by using Bulletin 17C (England and others, 2019). These guidelines are used by all Federal agencies and several state and local agencies for flood frequency analysis for gaged streams. Bulletin 17C guidelines include fitting the Pearson Type III distribution to the logarithms of the annual peak discharges using the sample moments and the Expected Moments Algorithm (EMA) to estimate the distribution parameters and provide for (1) use of interval data to indicate uncertainty in annual peak discharges, (2) outlier detection and adjustment, (3) adjustment for historical data using multiple perception threshold values, (4) development of generalized skew, and (5) weighting of station and generalized (regional) skew.

Computer programs for implementing Bulletin 17C guidelines and the new Bulletin 17C guidelines are available from the U.S. Army Corps of Engineers (USACE) HEC-SSP Program (HEC-SSP Statistical Software Package, User's Manual, Version 2.2, 2019) and the U.S. Geological Survey (USGS) PeakFQ Version 7.4 (Program PEAKFQ User's Manual, Flynn and others, 2006: Veilleux and others, 2014). Annual peak discharges for approximately 200 gaging stations in Maryland are available from the USGS over the World Wide Web at http://water.usgs.gov/md/nwis/sw . The annual peak data and the available computer programs can be used to estimate design discharges for Maryland gaged streams.

If the gaged watershed has undergone significant change during the period of record, the annual peak data may not be homogeneous. The user should ensure that the data are homogeneous and exhibit no significant trends due to land-use change before performing the frequency analysis. A simple way to check on this is to plot the annual peak discharges versus time and determine if there are any noticeable trends in the data. These plots are available on the USGS NWIS web site. Statistical procedures for performing a more quantitative evaluation of trends and non-homogeneity in flood data are discussed by Pilon and Harvey (1992), McCuen and Thomas (1991) and McCuen (1993). The Mann-Kendall test for trend is available in the USGS PeakFQ program.

Regional skew analyses were performed for all rural gaging stations for all hydrologic regions. The current values are for Eastern and Western Coastal Plain Regions are 0.38 with a standard error of 0.38 as described in Appendix 3 of this report. For the 2016 update of the regression equations for the Piedmont, Blue Ridge and Appalachian Plateau Regions, a regional skew of 0.43 and a standard error of 0.42 were used as described in Appendix 3 of this report.

Watershed characteristics for 188 gaging stations are given in Appendix 1. Flood discharges for the 1.25-, 1.50-, 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year peak discharges at 188 gaging stations in Maryland and Delaware are given in Appendix 2. The flood discharges for the Piedmont, Blue Ridge and Appalachian Plateau are based on annual peak data through the 2012 water year. For the Eastern Coastal Plain and Western Coastal Plain Regions, the flood discharges are based on annual peak data though the 2017 water year. Estimates of design discharges are available in Appendix 2 to those users who choose not to perform their own Bulletin 17C analysis. The watershed characteristics in Appendix 1 and the flood discharges in Appendix 2 were used to develop the FRRE provided in Appendix 3. **The FRRE given in Appendix 3 of this report are recommended for use in Maryland and supersede previous regression equations.**

If the watershed characteristics of the gaging station are similar to those used in deriving the regression equations, then the best estimate of design discharges at the gaging station is considered to be weighted estimates based on gaging station data and the Fixed Region regression estimates. The procedures for weighting the gaging station and regression estimates are described below.

In accordance with Appendix 9 of Bulletin 17C guidelines (England and others, 2019), it is assumed that an estimate at a single gaging station is independent of the regional regression estimate. Assuming independence of estimates, Hardison (1976) has shown that a weighted estimate, obtained by weighting each estimate inversely proportional to its variance, and has a variance less than either of the individual estimates. Hardison (1976) further demonstrated that weighting two estimates inversely proportional to their variances was comparable to weighting by the equivalent years of record. The following weighting equation described by Dillow (1996) should be used:

$$LQw = (LQg * Ng + LQr * Nr) / (Ng + Nr) \qquad (2.1)$$

where LQw is the logarithm of the weighted peak discharge at the gaging station, LQg is the logarithm of the peak discharge at the gaging station based on observed data, LQr is the logarithm of the peak discharge computed from the appropriate Fixed Region regression equation, Ng is the years of record at the gaging station, and Nr is the equivalent years of record for the Fixed Region regression estimate.

The equivalent years of record of the regression estimate is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of prediction of the regional regression equation. The equivalent years of record (Nr) is computed as follows (Hardison, 1971):

$$Nr = (S/SEp)^2\ R^2 \qquad (2.2)$$

where S is an estimate of the standard deviation of the logarithms of the annual peak discharges at the ungaged site, SEp is the standard error of prediction of the Fixed Region regression estimates in logarithmic units, and R^2 is a function of recurrence interval and skewness and is computed as (Stedinger and others, 1993):

$$R^2 = 1 + G * Kx + 0.5 * (1 + 0.75 * G^2) * Kx^2 \tag{2.3}$$

where G is an estimate of the average skewness for a given hydrologic region, and Kx is the Pearson Type III frequency factor for recurrence interval x and skewness G. Average skewness values G were defined using design discharges for each region as follows: 0.39 for the Appalachian Region, 0.48 for the rural and urban watersheds in the Blue Ridge and Piedmont Regions, 0.541 for the Western Coastal Plain Region and 0.330 for the Eastern Coastal Plain Region.

In order to estimate the equivalent years of record at an ungaged site, the standard deviation of the logarithms of the annual peak discharges (S in Equation 2.2) must be estimated. Average values of S were computed for each region and are as follows:0.3070 for the Piedmont-Blue Ridge Region, 0.2353 log units for the Appalachian Plateau Region, 0.3196 log units for the Western Coastal Plain Region, and 0.3104 log units for the Eastern Coastal Plain Region.

A computer program, originally developed by Gary Tasker, USGS, for use with the Dillow (1996) equations, computes the weighted estimate given in equation 2.1 and determines the equivalent years of record and standard errors of prediction for these estimates. The equivalent years of record for the weighted estimate is assumed to be Ng + Nr (see Equation 2.1), the sum of the years of gaged record and equivalent years of record for the regression estimate. The program originally developed by Tasker was updated over the years as new regression equations were developed for Maryland. The program was a standalone Fortran program but was also incorporated into GISHydro. With the revision of the Eastern and Western Coastal Plain Region equations in this edition of the report, the standalone program is no longer applicable. The computation of the weighted estimates, equivalent years of record and standard error of prediction are only available within GISHydro.

An example of computing a weighted estimate at a gaging station, Youghiogheny River near Oakland, Maryland (station 03075500), a 134.2-square-mile rural watershed Appalachian Plateau Region is illustrated below. The flood discharges for station 03075500 (Qg in cfs) based on 72 years of record are taken from Appendix 2 and are given in Table 2-1. Also provided in Table 2-1 are the Fixed Region (Appalachian Plateau Region) regression estimates (Qr in cfs) at station 03075500.

Table 2-1: Flood Frequency Estimates for Youghiogheny River near Oakland, Maryland (station 03075500) based on Gaging Station data, Regression Equations and a weighted estimate

Return period (years)	Station (Qg) (cfs)	Regression (Qr) (cfs)	Weighted (Qw) (cfs)
2	4,280	3,290	4,180
5	6,660	5,230	6,450
10	8,580	6,920	8,320
25	11,400	9,480	11,100
50	13,900	11,800	13,600
100	16,700	14,300	16,300
500	24,600	21,800	24,200

The Fixed Region regression estimates in log units (LQr) are weighted with the station estimates in log units (LQg) using Equation 2.1. The weighting factors are the years of record at station 03075500 (Ng = 72) and the equivalent years of record (Nr) for the regression equations that are computed from the Tasker Program. The weighted estimates are shown in Table 2-1. For example, the 100-yr weighted estimate is computed from Equation 2.1 as follows using the logarithms of the flood discharges:

$$
\begin{aligned}
LQw &= (LQg * Ng + LQr * Nr) / (Ng + Nr) \\
&= (4.222716 * 72 + 4.155336 * 12) / (72 + 12) \\
&= 4.213091 \text{ log units, where } Qw = 16{,}300 \text{ cfs.}
\end{aligned}
$$

The equivalent years of record for the weighted estimate is assumed equal to the sum of the observed record length (72 years) and the equivalent years of record from the regression equation (12 years). Therefore, for the 100-yr weighted estimate, the equivalent years of record are 84 years.

Figure 2-3 illustrates the process of weighting station data with the regional regression estimates.

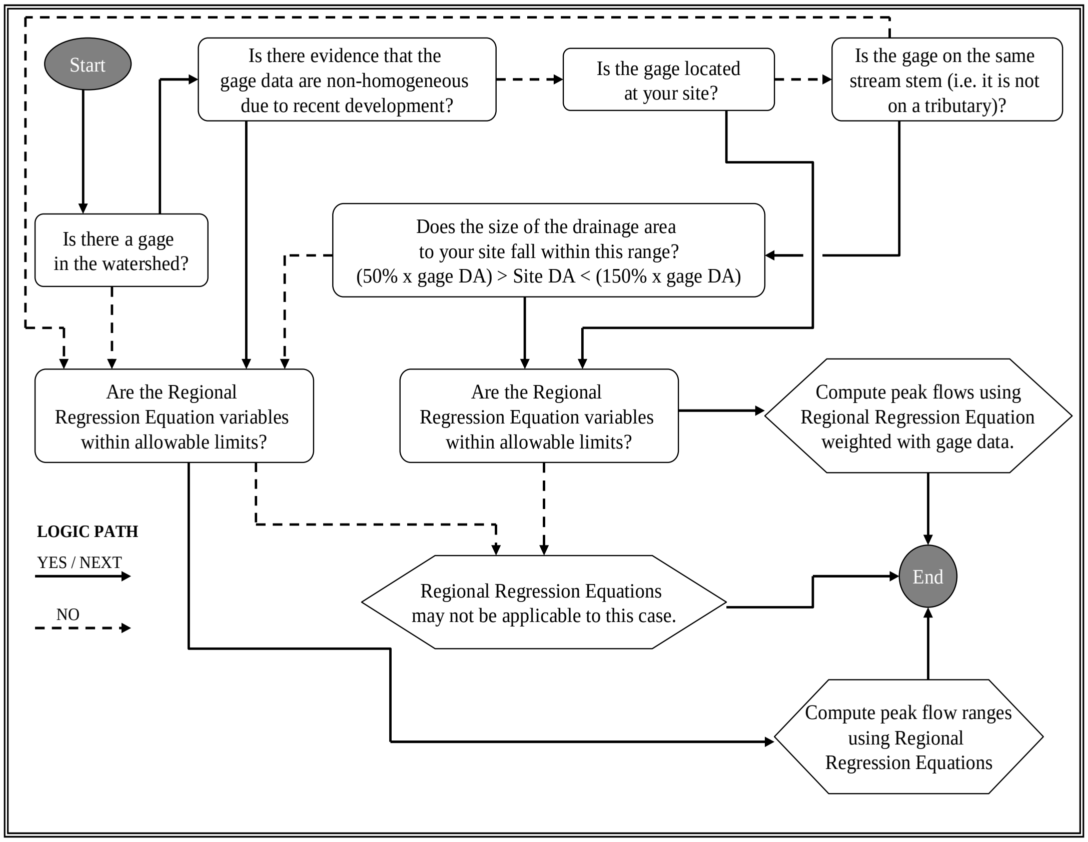

Figure 2-3: Fixed Region Regression Equation Flow Chart

2.3 ESTIMATES FOR UNGAGED SITES NEAR A GAGING STATION

Procedures described by Dillow (1996) are recommended for obtaining estimates of design discharges for ungaged sites that are on the same stream as the gaging station, have similar watershed characteristics as the gaging station and are within 50 percent of the drainage area of the gaging station. Data shown in Appendix 1 can be used to determine if the gaged stream has watershed characteristics similar to those used in developing the regression equations. The procedure involves three steps:

1. Compute the ratio (R) of the weighted estimate to the Fixed Region regression estimate at the gaging station

$$R = Qw/Qr \qquad (2.4)$$

where Qw and Qr are the weighted and regression estimates in cfs.

2. Scale the ratio R based on the difference in drainage area between the ungaged site and the gaging station using the following equation (Sauer, 1974):

$$Rw = R - [(2\,|Ag - Au|)/Ag] * (R - 1) \tag{2.5}$$

where Rw is the scaled ratio, Ag is the drainage area in square miles at the gaging station and Au is the drainage area in square miles at the ungaged location.

3. Compute the final estimate (Qf) at the ungaged site as

$$Qf = Rw * Qu \tag{2.6}$$

where Qu is the Fixed Region regression estimate in cfs at the ungaged site.

Equation 2.5 was developed with the limiting assumption that estimates would only be extrapolated upstream and downstream on the same stream to 0.50 or 1.50 times the drainage area of the gaging station. If Equation 2.5 is used beyond these limits, then irrational results may be obtained. If the gaged watershed has undergone significant change during the period of record, then the annual peak data may not be homogeneous and the extrapolation procedure may not be appropriate.

In the case where the ungaged site is between two gaging stations, estimates of Qg should be obtained by interpolating between the two gaging stations on the basis of a logarithmic plot of peak discharge versus drainage area. An estimate of Ng is obtained as an arithmetic average of the record length at the two gaging stations using the differences in drainage area between the ungaged site and the gaging stations as the weighting factor. The values of LQg and Ng so obtained should be used in Equation 2.1 to get a final weighted estimate for the ungaged site.

The weighted estimates at the Youghiogheny River near Oakland, Maryland (shown in Table 2-1), where the drainage area is 134.16 square miles, are extrapolated upstream to an ungaged location where the drainage area is 89.7 square miles. For this procedure to be applicable, the watershed characteristics at the ungaged site should be similar to those at the gaged site. For this example, the weighted (Qw) and regression (Qr) 100-yr flood discharge at station 03075500 are 16,300 and 14,300 cfs, respectively, and the regression estimate (Qu) at the ungaged location is 10,300 cfs. The adjusted 100-yr flood discharge at the ungaged location on the Youghiogheny River is computed to be 10,800 cfs using Equations 2.4 to 2.6 as follows:

$$R = Qw/Qr = 16{,}300/14{,}300 = 1.13986$$

$$\begin{aligned} Rw &= R - \{[(2\,|Ag - Au|)/Ag] * (R - 1)\} \\ &= 1.13986 - \{[(2\,|134.16-89.7|)/134.16] * (0.13986)\} = 1.047162 \end{aligned}$$

$$Qf = Rw * Qu = 1.047162 * 10{,}300 = 10{,}800 \text{ cfs.}$$

The equivalent years of record are 36.1 years for the 100-yr flood discharge at the ungaged location. This value is interpolated between 84 years for the weighted station data at 134.16 square miles and 12 years for the Fixed Region regression equation estimate at 0.5 times the gaged drainage area at 0.5 * 134.16 = 67.08 square miles. The computation is

$$84 - [(84 - 12) * 44.46/67.08] = 36.1 \text{ years.}$$

2.4 ESTIMATES AT UNGAGED SITES

The FRRE given in Appendix 3 can be used for estimating the 1.25-, 1.50-, 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year peak discharges for rural and urban watersheds in Maryland which are not significantly affected by detention storage, urbanization, tidal marshes or changing land-use conditions such as mining, excavation or landfill activities. Equations applicable to urban watersheds are available for just the Western Coastal Plain and Piedmont and Blue Ridge Region.

In addition, the watershed characteristics for the site of interest should be within the range of the watershed characteristics of the gaging stations used in the regional analysis. Watershed characteristics used in the development of the FRRE are given in Appendix 1. These data can be used to determine if the ungaged site has similar watershed characteristics as those used in developing the regression equations.

The standard error of prediction for the ungaged site is computed as the sum of the model and sampling error as described by Hodge and Tasker (1995). Given the standard error of prediction for an ungaged site, the equivalent years of record are computed by Equation 2.2. Prediction intervals are then computed as:

$$\log Qx + T(c/2, n - p) \times [SE^2(1 + ho)]^{0.5} \qquad \text{upper value} \qquad (2.7a)$$

$$\log Qx - T(c/2, n - p) \times [SE^2(1 + ho)]^{0.5} \qquad \text{lower value} \qquad (2.7b)$$

where Qx is the flood discharge for recurrence interval x, T is the critical value of Student’s t for a 100 (1-c) percent prediction interval with n−p degrees of freedom, n is the number of gaging stations used in the regression analysis, p is the number of explanatory variables in the Fixed Region regression equation, SE is the standard error of estimate in log units, and ho is the leverage of the site.

The standard error of prediction (SEp) is more accurate than using the standard error of estimate given in Appendix 3. The standard error of estimate given in Appendix 3 is a measure of the variability of the station data about the regression equation and is less than the standard error of prediction which is a measure of how well the equations predict flood discharges at an ungaged site. The standard error of prediction includes both the variability about the regression equation and the errors in the regression coefficients.

The leverage expresses the distance of the site's explanatory variables from the center of the convex data set (called the Regressor Variable Hull) defined by the explanatory variables in the regression analysis (Montgomery and Peck, 1982). The prediction intervals are directly related to the magnitude of the leverage for a given site. The leverage is computed as (**bold letters** denote a matrix):

$$ho = xo\,(\mathbf{X}^T\mathbf{X})^{-1}\,xo^T \qquad (2.8)$$

where xo is a row vector of the logarithms of the explanatory variables at a given site, $(\mathbf{X}^T\mathbf{X})^{-1}$ is the covariance matrix of the regression parameters (T means transpose), xo^T is a column vector of the logarithms of the explanatory variables at a given site.

Equations 2.7 and 2.8 and the data in Appendix 1 are used to compute the prediction limits. For plus and minus one standard error of prediction, there is a 68 percent chance that the true discharge is between the upper and lower prediction limits.

The range of watershed characteristics for each hydrologic region is given in Table 2-2. The watershed characteristics were estimated using GIS data from several sources as described in Appendix 1. The FRRE for each hydrologic region are given in Appendix 3 along with the standard error of estimate and the equivalent years of record. The FRRE are based on 35 stations in the Eastern Coastal Plain, 23 rural and urban stations in the Western Coastal Plain, 64 rural and 32 urban stations in the Piedmont and Blue Ridge, and 24 stations in the Appalachian Plateau. A total of 178 stations were used to derive the FRRE in Appendix 3.

In developing the FRRE, forest cover and impervious area near the midpoint of the period of systematic data collection were used for the urban watersheds. For gaging stations discontinued before 1999, forest cover and impervious area for the 1985 land use conditions were generally used. For the rural watersheds this is not an issue since forest cover and impervious area are not significantly changing with time. In applying the regression equations, the analyst should use the current land use conditions to obtain estimates of the flood discharges for existing conditions.

For streams that cross regional boundaries, the regression equations for each region should be applied as if the total drainage area was in each region. These estimates should then be weighted by the percentage of drainage area in each region. The weighted flood frequency estimates can be obtained using GISHydro.

Table 2-2: Range of Watershed Characteristics for Each Hydrologic Region in Maryland

Variable	Eastern Coastal Plain	Western Coastal Plain	Piedmont and Blue Ridge (Rural and urban)	Appalachian Plateau
DA [mi^2]	0.91 to 113.8	0.96 to 350.21	0.11 to 816.4	0.52 to 294.1
IA [%]	---	0 to 36.8	0.0 to 53.5	---
A soils [%]	0.5 to 82.7	1.7 to 85.2	---	---
LIME [%]	---	---	0.0 to 81.7	---
LSLOPE [ft/ft]	0.00463 to 0.0220	---	---	0.0640 to 0.25265

DA	Drainage area in square miles measured on horizontal surface.
A soils	Percent of DA that is classified as NRCS Hydrologic Soil Group A based on SSURGO soils data.
IA	Percent of DA that is impervious as defined by the Maryland Department of Planning land use data.
LIME	Percent of DA that is underlain by carbonate rock (limestone and dolomite), from map given in Appendix 3.
LSLOPE	Average land slope of the watershed in feet per feet.

2.5 FUTURE RESEARCH TO IMPROVE REGRESSION EQUATIONS

The FRRE are applicable to both rural and urban watersheds in the Western Coastal Plain and Piedmont and Blue Ridge Regions. For the urban watersheds, a “relatively constant period of urbanization” was defined as a change in impervious area of less than 50 percent during the period of record. If a watershed had 20 percent impervious area at the beginning of record, it could have no more than 30 percent impervious area at the end of the time period (Sauer and others, 1983). The periods of record for a few urban stations were reduced to obtain a more homogeneous period of record with respect to land use. Several urban gaging stations were discontinued in the late 1980s and land use data for 1985 were considered most appropriate. Also, data collection began for several urban watersheds around 2000 and the time period of data collection was not long enough to show significant change in land use characteristics. For the recently established urban gaging stations, land use conditions in 2002 or 2010 were considered representative for the annual peak data. For some urban gaging stations, a time-varying mean approach (Kilgore and others, 2016; 2019) was used to account for changing land use conditions. For future analyses, a more detailed approach should be developed for determining a homogeneous period for frequency analysis or for adjusting the annual peak data to existing conditions.

The Maryland Department of Planning (MDP) data were used to estimate land use conditions such as impervious area. The MDP approach is to assign a percentage of impervious area to various land use categories. For example, Institutional Lands are assigned an impervious area of 50 percent but there is considerable variation in impervious area for this land use category. Impervious area as estimated from the MDP data was statistically significant in estimating flood discharges for urban watersheds in the Western Coastal Plain and Piedmont-Blue Ridge Regions but this variable did not explain as much variability as anticipated. For future regression analyses, more accurate or detailed measures of urbanization (impervious area, percentage of storm sewers, length of improved channels, etc.) should be used for characterizing urbanization and its effect on flood discharges. Improved measures of urbanization would likely provide more accurate regression equations in the future.

Many of the gaging stations on small watersheds (less than about 10 square miles) were discontinued in the late 1970s resulting in generally short periods of record for the small watersheds in Maryland. As described earlier, Carpenter (1980) and Dillow (1996) utilized estimates of flood discharges from a calibrated rainfall-runoff model for 10 gaging stations in Maryland. Carpenter (1980) also adjusted flood discharges at 17 other small watersheds based on comparisons to nearby long-term gaging station data. Moglen and others (2006) utilized both of these adjustments in developing the FRRE that were documented in the August 2006 version of the Hydrology Panel report. Thomas and Moglen (2016) extended the record at four short-term stations using data at nearby long-term stations. In addition, Thomas and Moglen (2016) also defined frequency curves at eight stations using a graphical approach where the log-Pearson Type III distribution did not reasonably fit the annual peak data. For the Eastern Coastal Plain analyses, Thomas and Sanchez-Claros (2019a) used a graphical frequency analysis at three gaging stations and extended the record for two short-term stations using data at nearby long-term stations. For the Western Coastal Plain analyses, Thomas and Sanchez-Claros (2019b) used a time-varying mean approach (Kilgore and others, 2016; 2019) or a more homogenous period of record to account for changing land use conditions for six stations.

There are many other short-record stations in Maryland for which no adjustment was made. For future regression analyses, a more systematic approach for adjusting the short-record stations should be developed. In addition, stream-gaging activities should be resumed on several of the small watersheds where there are less than 15 years of record. Improving the data base of small watershed data would provide more accurate regression equations in the future.

Finally, only gaging stations located in Maryland were used in developing the FRRE in Appendix 3 because the required land use data were not available in neighboring states. The exception was the inclusion of gaging stations in Delaware for the Eastern Coastal Plain analysis where consistent land use data are available for 2002. More detailed land use data should be developed using procedures consistent with Maryland for the neighboring states like Pennsylvania, Virginia and West Virginia so that additional gaging stations could be included in future regional regression analyses for Maryland.

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CHAPTER THREE

3 BEHAVIOR OF THE WinTR-20 MODEL IN RESPONSE TO UNCERTAINTIES IN THE INPUT PARAMETERS

3.1 OVERVIEW

The WinTR-20 model is a deterministic hydrologic model that synthesizes a single event runoff hydrograph as a function of a rainfall input and watershed characteristics. The model is designed to operate on a time varying rainfall to produce a hydrograph that simulates the role of the watershed area; land cover; hydrologic soil types; antecedent runoff conditions; topography; characteristics of the overland, shallow confined, and channel flow paths; and storage attenuation such as that created by flood plains, wetlands, structures, and ponds. A single watershed can be modeled by inputting the drainage area, time of concentration, curve number and a time-intensity rainfall distribution. If the watershed is large or heterogeneous, it can be divided into a number of subwatersheds with their hydrographs attenuated by routing through the stream network that the user defines in terms of length, slope, roughness, cross-section and any storage elements or structures that may be distributed along its length.

Because the WinTR-20 model can simulate watershed conditions and changes in these conditions in terms of relatively simple input parameters, it continues to be the baseline for hydrologic analyses that require hydrographs for both existing and ultimate development conditions. The first step is to select model parameters that are consistent with established hydrologic practice and give a reasonable simulation of existing hydrologic conditions. After the user is satisfied that the model is satisfactory for existing watershed conditions, the curve number and flow network parameters can be changed to simulate the hydrologic response of the watershed under a future, or ultimate development, land cover distribution and drainage hydraulics.

The WinTR-20 model, like most deterministic hydrologic models, is quite sensitive to the values chosen for the input parameters. These sensitivities and the uncertainties surrounding their selection make it difficult to ensure that the WinTR-20 results are representative of all Maryland conditions. The tendency among designers has been to select parameters that lead to over-prediction in many cases. This is supported by U.S. Water Resources Council (1981) tests on ten procedures for estimating peak discharges for ungaged watersheds. Each procedure was applied by five persons at gaging stations with at least 20 years of observed peak-flow records. Based on 105 applications at 21 gaging stations in the Midwest and Northwest Regions of the country, it was found that the TR-20 model with no calibration overestimated the 100-yr flood discharge by about 55%, the 10-yr discharge by about 60% and the 2-yr discharge by about 55%.

The Panel recognizes the parameter sensitivities of the WinTR-20 model and its tendency to over predict. However, the Panel has concluded that these problems can be overcome and that the WinTR-20 model can be a sound, dependable model for simulating existing and ultimate conditions for most watersheds provided that it is calibrated for local conditions. Calibration of all deterministic models is strongly recommended by AASHTO (1991, pgs. 7-17, 7-18). The Panel recommends that it become standard practice to require that the WinTR-20 be calibrated for existing watershed conditions against one of the statistics-based procedures of Chapter 2, provided that the watershed conditions are consistent with those above the USGS gage or the sample used to derive the approved regional regression equations. The approved regional regression equations are based on statistical analyses of stream gages in Maryland and adjacent states having record lengths between 10 and over 70 years. Thus, a successful calibration following the procedures outlined in Chapter 4 and Appendix 5 can produce reliable WinTR-20 peak discharges that are consistent with Maryland conditions.

In order to gain insight into the sensitivities associated with the TR-20 input parameters under Maryland conditions, the SHA sponsored a study by Ragan and Pfefferkorn (1992). This study entitled, "Analysis of the Role of Storm and Stream Parameters on the performance of SCS-TR-20 and HEC-1 under Maryland Conditions," was conducted on the 21.3 square mile Northwest Branch watershed in Montgomery County. The Northwest Branch was selected because it had been the subject of many studies by various organizations and, therefore, had an excellent data base along with an established GIS that managed the land and stream elements of the watershed. There were 76 surveyed stream cross-sections along 71,000 feet of channel, detailed soil data, high resolution color IR defined land cover and long-term stream flow records. All these data were in digital formats and interfaced with a GIS. Most of the examples of hydrograph responses to variations in TR-20 input parameters that follow in Chapter 3 are from the Ragan and Pfefferkorn (1992) experiments.

The remaining sections of Chapter 3 discuss the issues that the Panel examined with respect to defining the input parameters to the WinTR-20 model. Chapter 4 and the appendices discuss procedures that will assist the WinTR-20 user in the selection of input parameters during the calibration process.

3.2 DRAINAGE AREA

The scale of the map can create an error in the estimate of the drainage area. Delineating on a small-scale map, such as 1:100,000, probably will not give the same drainage area as one would obtain from a 1:24,000 or 1:4,800 scale map. Normally, watersheds having drainage areas larger than one square mile will be delineated on a 1:24,000 scale USGS 7.5-minute quadrangle sheet or using digital terrain data. Special care must be exercised in flat terrain such as the Eastern Coastal Plain because of the wide spacing of contours and lack of definitive of ridge lines.

Hydrologists and designers for Maryland projects frequently use GISHydro. GISHydro is a geographic information system that generates watershed boundaries and stream networks using USGS digital terrain data to automatically deleneate drainage area boundaries. Two issues must be recognized with any automated drainage area delineation method. The first issue is training. The person using automated techniques must be thoroughly trained in the GIS software and familiar with the digital terrain data. The procedure can give excellent results, but if the user does not know what he or she is doing, significant errors can result. For example, if one tries to delineate a watershed that is too small — one containing only a few elevation points — the results will be very questionable. Figure 3-1, developed from a study by Fellows (1983), shows the percent difference between watershed areas manually delineated on paper 1:24,000 scale maps and those grown from digital terrain data as a function of the number of elevation points inside the boundary. A_M is the area determined "manually" by visually tracing the ridge lines on 1:24,000 scale maps. A_G is the area "grown" using the digital terrain data.

A second issue that must be recognized is resolution — the spacing of the elevation points in the data base. GISHydro provides 30-meter resolution digital terrain data for all of Maryland. There may be instances where the watershed boundary extends across a state boundary. In such an instance, the user might have to use data from another source that has a 90-meter resolution. The 90-meter data may not give the same level of accuracy as the 30-meter data. If the area of the watershed is incorrect, the peak discharge will be incorrect as well.

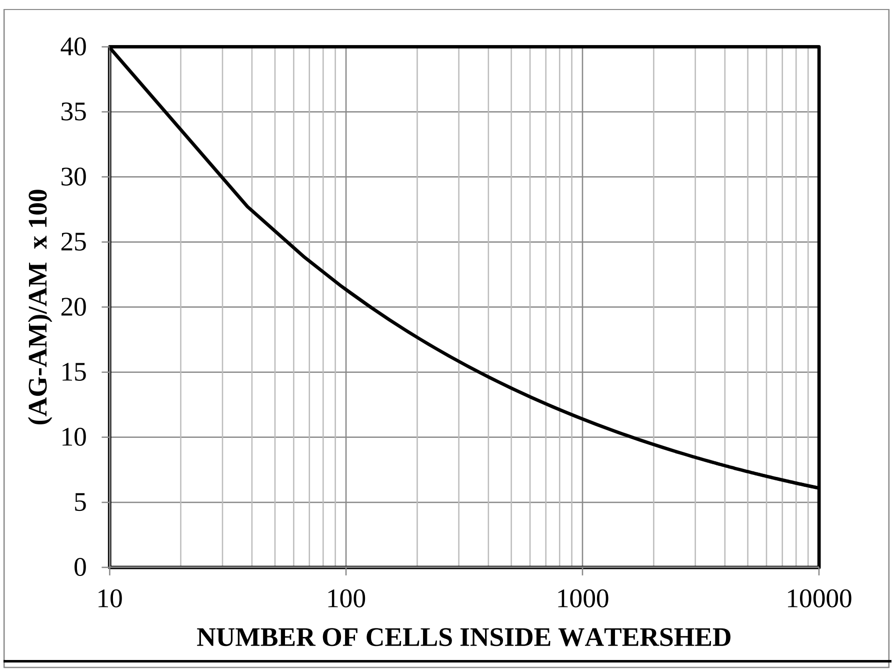

Figure 3-1: 99% Confidence Error Envelope for Difference Between Manually and Automatically Defined Areas

It is emphasized that all watershed and subwatershed boundaries developed with GISHydro must be checked to ensure that there is good agreement with the areas obtained from paper format 1:24,000 USGS quad sheets.

3.3 VOLUME OF RUNOFF

A deterministic model must have a component that estimates the rainfall excess that becomes the volume of the runoff hydrograph. Thus, there must be a means to account for the interception, infiltration and depression storage processes that occur in the watershed. In the NRCS family of models, the rainfall excess is estimated by a Runoff Curve Number (RCN) that is a function of the land cover, the underlying soil type, and antecedent runoff conditions (ARC). Tables 2-2a thru 2-2d from U.S. Department of Agriculture (1986) are recommended for use in hydrologic analyses using the WinTR-20.

The rainfall excess, or volume of runoff under the hydrograph, is given by Equation 3.1

$$Q = (P - 0.2S)^2 / (P + 0.8S) \qquad (3.1)$$

where: $$S = (1000/RCN) - 10 \qquad (3.2)$$

Tables 2a through 2d in TR-55 assign curve numbers in terms of “good,” “fair,” or “poor” condition in some of the land cover categories. First, it may be difficult for the designer to determine which of the conditions is appropriate for each land parcel in the watershed. Further, the curve numbers were derived using watershed data collected from across the United States. Thus, the specific curve number for a given soil-cover complex may or may not be appropriate for the particular Maryland watershed under investigation. Finally, Equation 3.1 is a simplification of

$$Q = (P - I_a)^2 / [(P - I_a) + S] \qquad (3.3)$$

where it is assumed that:

$$I_a = 0.2\ S \qquad (3.4)$$

The data on which the assumption of Equation 3.4 is based, presented as Figure 10-1 in USDA- NRCS-NEH, 630, Chapter 10, (2004), are shown here as Figure 3-2.

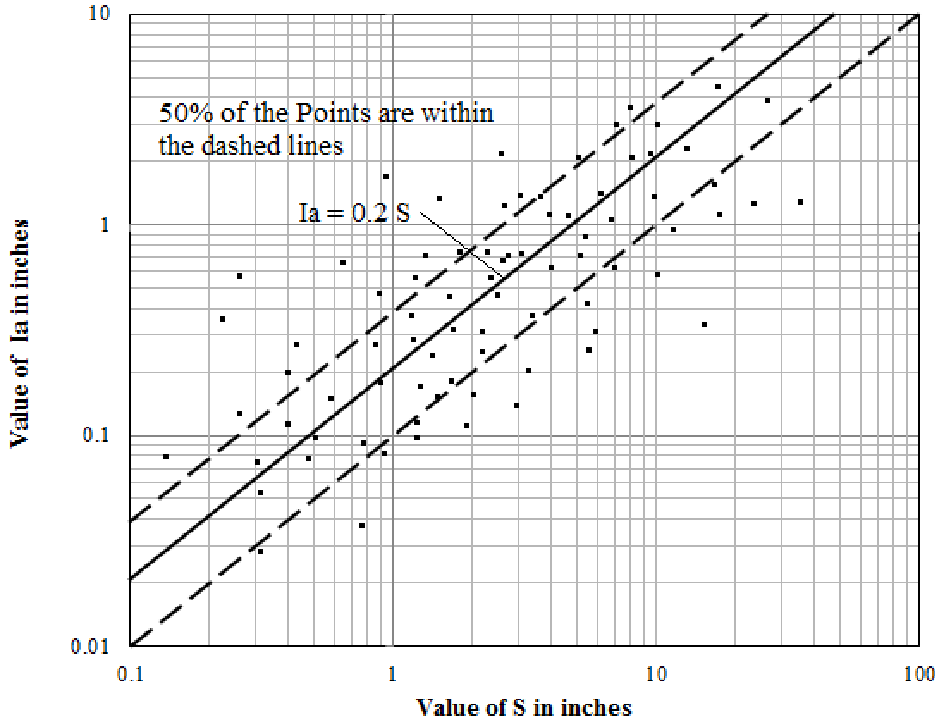

Figure 3-2: Relationship Between I_a and S

(Plotted points are derived from experimental watershed data) Source: Figure 10-1 of USDA-NRCS-NEH Part 630 Hydrology, Chapter 10)

The consequences of making an error in the determination of the weighted curve number for a natural watershed are illustrated by Figure 3-3 from Ragan and Pfefferkorn (1992).

The purpose of this Section 3.3, Volume of Runoff, is to encourage users of the WinTR-20 to recognize that estimating the volume of surface runoff using the curve number approach is an imperfect process. Thus, as described in Chapter 4, the Panel recommends that the user exercise a degree of flexibility in the selection of curve numbers to represent specific land/soil complexes provided that the basis for the decision is explained.

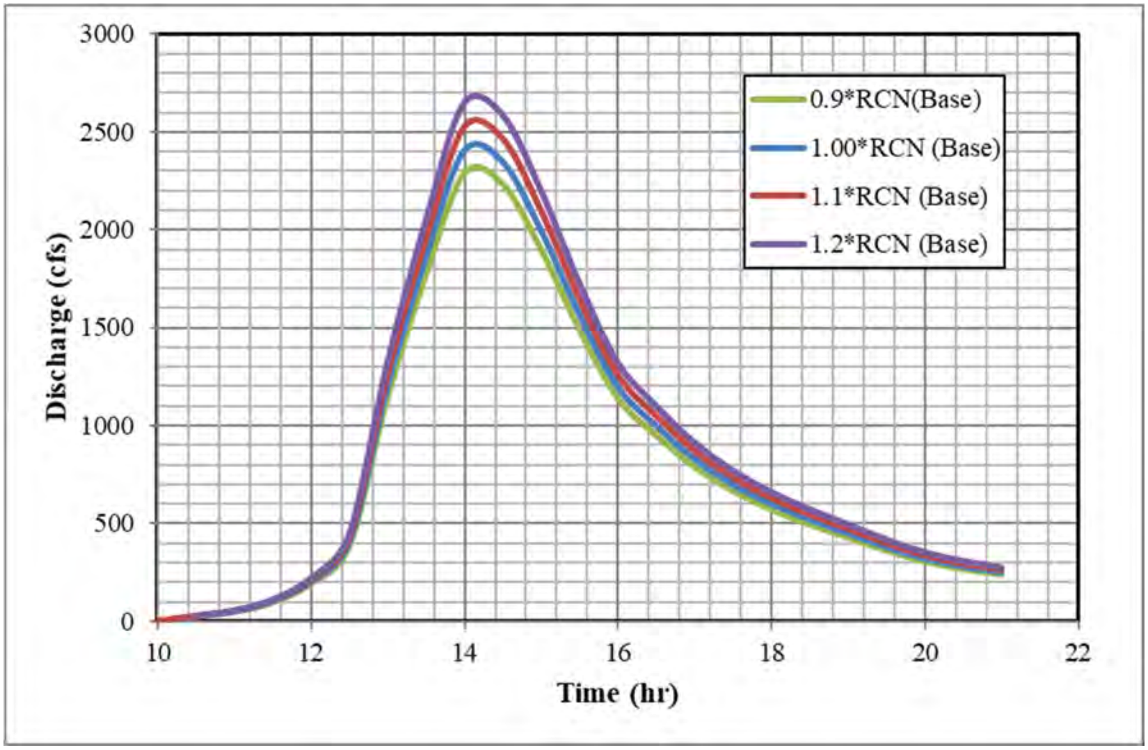

Figure 3-3: Hydrograph Response to Changing RCN

3.4 PEAK DISCHARGE AND SHAPE OF THE RUNOFF HYDROGRAPH

3.4.1 The Dimensionless Unit Hydrograph

A storm occurring on a low-relief watershed with wide, flat streams will produce a long duration hydrograph with a low peak discharge in comparison with that generated by a high-relief mountain basin having steep narrow channels. Many deterministic models, including the WinTR-20, simulate the interrelationships among the runoff processes through a unit hydrograph (UHG). If stream flow records are available for the subject watershed, the WinTR-20 allows a site specific UHG to be input. If possible, the derived site specific UHG should be used. However, the usual circumstance is to use the default dimensionless UHG built into the WinTR-20. While the NRCS dimensionless UHG is thoroughly discussed in Chapter 16 of NRCS-NEH Part 630, Hydrology, several issues are presented here for completeness.

The dimensionless UHG controls the shape and peak discharge of the runoff hydrograph using the drainage area, the volume of runoff, and the time of concentration as input parameters. NRCS-NEH Part 630, Hydrology gives the peak discharge of the unit hydrograph that the WinTR-20 convolves with the time-distribution of rainfall excess as

$$q_{oph} = 484A_{sm}Q / T_p \qquad (3.5)$$

$$T_p = \Delta D/2 + 0.6T_c \tag{3.5a}$$

where A_{sm} is watershed area in square miles and T_p is the time to peak in hours. In Equation 3.5, Q is 1.0 inches because it is a unit hydrograph. Time to peak is a function of the duration of the unit excess rainfall, ΔD, and the time of concentration T_c as shown in equation 3.5a.

The constant value of 484 is the "peak rate factor." NRCS-NEH Part 630, Hydrology points out that "this factor has been known to vary from about 600 in steep terrain to 100 in very flat swampy country." A UHG with a peak rate factor of 284 has been used for some time on the flat watersheds of the Maryland Eastern Coastal Plain.

In the case of the Maryland Eastern Coastal Plain UHG, the lower peaking factor accounts for the greater storage and longer travel times of the flat wetlands often found on streams in that area. However, one must be aware that a peak flow rate can sometimes be effectively changed by subdividing the watershed into sub-basins and then routing the sub-basin hydrographs through the storage provided by the network of connecting streams. In general, models that have larger (more than one square mile) sub-basins should use the regional dimensionless unit hydrograph. In Maryland, Brubaker and others (2021) completed a research study on peak rate factors but were not able to define reasonable or consistent associations between peak rate factor and watershed properties. Until other values are published, the designer may use the peaking factor values for the Maryland Dimensionless Unit Hydrographs shown in Table 3-1. The dimensionless unit hydrograph to be used when the peak factor is 284 is presented in Table 3-2.

Table 3-1: Unit Hydrograph Peak Factors

REGION	PEAK FACTOR
Eastern Coastal Plain	284
Western Coastal Plain	284 or 484
Piedmont	484
Blue Ridge	484
Appalachian	484

Table 3-2: Dimensionless Unit Hydrograph for Use When Peak Factor is 284

```
TITLE     DELMARVA UNIT HYD PRF NEAR 284
 4 DIMHYD              .02
 8          0.0        .111        .356        .655        .896
 8          1.0        .929        .828        .737        .656
 8          .584       .521        .465        .415        .371
 8          .331       .296        .265        .237        .212
 8          .190       .170        .153        .138        .123
 8          .109       .097        .086        .076        .066
 8          .057       .049        .041        .033        .027
 8          .024       .021        .018        .015        .013
 8          .012       .011        .009        .008        .008
 8          .006       .006        .005        .005       0.0
 9 ENDTBL
```

If a watershed lies within more than one region, the WinTR-20 model can be split into appropriate parts with corresponding regional dimensionless unit hydrographs (DUH). In Maryland, this occurs at the boundary between the Western Coastal Plain and the Piedmont, known as the Fall Line. If the WinTR-20 flood discharges agree with the regional estimates without use of two DUH, then no additional action is needed. If the WinTR-20 flood discharges are not within the calibration window, the user should subdivide the watershed at the Fall Line and use the two DUH as appropriate. If a significant portion (75% or more) of the watershed falls within one region, then use that region's dimensionless unit hydrograph.

In addition to the probable variation of the peak rate factor as a function of the watershed topography, it can also be seen from Equation 3.5 that the peak discharge of the UHG is a function of the time of concentration, T_c. As described later in this chapter, the time of concentration is difficult to define. Thus, the NRCS dimensionless or any other "nationally-derived" synthetic UHG defined in terms of a few parameters can create errors in the runoff estimate. In the future there may be approaches that allow the use of more site specific UHG's, even when no stream flow records are available. Because of the availability of the USGS digital terrain data, the "geomorphic" UHG using a time-area-curve concept that tracks the flow path of each grid cell in the watershed should be a practical approach in the future.

3.4.2 Time of Concentration and Lag

Definitions

Travel time is the time it takes for runoff to travel from one location in a watershed to another location downstream. Estimating travel time is complicated by the fact that it may occur on the surface of the ground or below it or a combination of the two. The *Time of Concentration* is the time required for runoff to travel from the hydraulically most distant part of the watershed to the outlet of the watershed. Recall that it is the time of concentration that is input to the WinTR-20 to define the peak discharge of the unit hydrograph from the dimensionless UHG. The *Lag* can be thought of as a weighted time of travel. If the watershed is divided into increments, and the travel times from the centers of the increments to the watershed outlet are determined, then the lag is calculated as:

$$L = \frac{\Sigma\,(a_i\,Q_i\,T_{ti})}{\Sigma\,(a_i\,Q_i)} \qquad (3.6)$$

where:

L	is the lag time, in hours;
a_i	is the i^{th} increment of the watershed area, in square miles;
Q_i	is the runoff from area a_i, in inches;
T_{ti}	is the travel time from the center of a_i to the point of reference, in hours.

NRCS-NEH Part 630, Hydrology provides the empirical relation

$$L = 0.6\,T_c \qquad (3.7)$$

where T_c is the time of concentration.

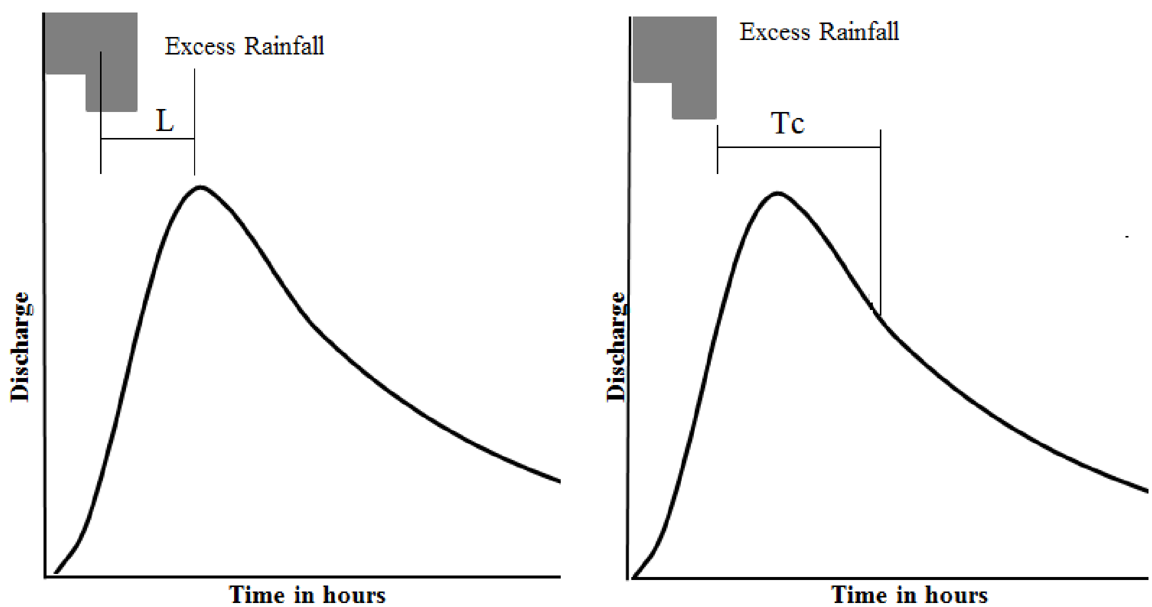

Figure 3-4: Graphical definitions of lag time and time of concentration

Lag, as defined by NRCS, is the time from the center of mass of the rainfall excess to the peak rate of runoff as shown by Figure 3-4 (left). Similarly, the time of concentration is the time from the end of the rainfall excess to the point on the falling end of the hydrograph where the recession curve begins, as shown in Figure 3-4 (right). It is quite difficult to determine the time that the rainfall excess begins and ends. Where sufficient rainfall and runoff data are not available, the usual procedures for determining L and T_c are outlined in the following sections.

NRCS-NEH Part 630, Hydrology discussed two methods for estimating time of concentration and lag when hydrograph data are not available. These methods, the curve number method and the flow path hydraulics method (also known as the velocity method), are discussed in the following sections.

3.4.3 Watershed Lag Method to Estimate Time of Concentration

One parameter that is needed for input to the WinTR-20 is the time of concentration. The designer may use Watershed Lag Equations or graphs. The time-of-concentration is calculated as:

$$T_c = 1.67\ L \tag{3.8}$$

where both T_c and L are in either hours or minutes.

The NRCS Watershed Lag Equation is:

$$L = \frac{L_h^{\,0.8}\,(S+1)^{\,0.7}}{1900\ Y^{\,0.5}} \tag{3.9}$$

where: L is the Lag, in hours

L_h is the hydraulic length of watershed, in feet

S is $1000/\mathrm{RCN} - 10$ (3.10)

Y is the average watershed land slope (perpendicular to flow), in Percent

<u>The NRCS Watershed Lag Equation may not be used when the drainage area is greater than five square miles. The minimum length used in the Lag Equation shall be 800 ft.</u> Shorter lengths will result in artificially low lag times.

There are several ways to estimate the watershed slope, Y, and they may not agree with each other. The original version of the GISHydro used the average slope categories assigned to the soil types. This is probably the weakest approach. The optimal approach is to use the 10- or 30-meter resolution digital terrain data that are available for Maryland in GISHydro. Slopes estimated with 90-meter data will not agree with the 30-meter data. Another approach is to digitize the areas between “heavy line” contours, assign average elevations to these enclosed areas and then weight them for the watershed. The “heavy line” contours are those such as 100 feet, 200 feet, etc. Finally, the lengths of the heavy line contours can be measured and the watershed slope estimated as:

$$\text{Watershed Slope} = M\ N / A_{sf} \tag{3.11}$$

where: M	is the total length of heavy line contours, in feet
N	is the contour interval, in feet
A_{sf}	is the drainage area in, square feet

The hydraulic L_h length in feet can be estimated from a map or the following relation can be used:

$$L_h = 209(A_{ac})^{0.6} \tag{3.12}$$

where A_{ac} is area in acres.

In summary, there are several issues in the use of the empirical lag equation approach that impact the time of concentration and, thereby, the peak discharge of the storm hydrograph. The uncertainties in the value of the curve number discussed in Section 3.3 represent one problem. Estimating the hydraulic length is another. And the value assigned to the slope depends on the estimation approach adopted.

The reader will note that the lag equation is not included as a procedure in WinTR-55, <u>Hydrology for Small Watersheds</u>. Thus, the Panel cautions against the use of the lag equation in urban (≥ 10% impervious) watersheds until additional research becomes available.

3.4.4 Estimating the Time of Concentration from Flow Path Hydraulics

The time of concentration is the cumulative flow time required for a particle of water to travel overland from the hydraulically most remote point overland, through the shallow concentrated flow channels, and through the mainstream network to the watershed outlet. The time may increase as a consequence of flow through natural storage such as lakes or wetlands or ponding behind culverts or other man-made structures. Estimating the time of concentration by simulating the hydraulics of each flow path component is treated in this section. Because the quantity of flow and, therefore, the hydraulics are different for each storm frequency, it is logical to expect that the time of concentration will be different for a 2-year storm than for a 100-year storm. Recognizing this, the Panel recommends that bankfull conditions that many consider to approximate the 2-year storm conditions be used to estimate the time of concentration.

3.4.5 Overland Flow

At the upper reaches of a watershed, runoff does not concentrate into well-defined flow paths, such as rills, gullies, or swales. Instead it probably flows over the surface at reasonably uniform, shallow depths as sheet flow. Sheet flow is evident on long, sloping streets during rainstorms. After some distance, sheet flow begins to converge into concentrated flow paths that have depths noticeably greater than that of the shallow sheet flow. The distance from the upper end of the watershed or flow surface to the point where significant concentrated flow begins is termed the overland flow length. For impervious surfaces the overland flow length can be several hundred feet. For pervious erodible surfaces and surfaces with vegetation, concentrated flow will begin after relatively short overland flow lengths.

In the upper reaches of a watershed, overland flow runoff during the intense part of the storm flows as a shallow layer with a reasonably constant depth. An equation, referred to as the kinematic wave equation for the equilibrium state, can be developed using Manning's equation with the assumption that the hydraulic radius equals the product of the rainfall intensity and the travel time, i.e., $R_h = i\,T_o$; hydraulic radius is equal to the uniform flow depth for a wide open channel. Using the velocity equation with the travel time (minutes) equal to the time of concentration, Manning's equation becomes:

$$V = \frac{L}{T_t(60)} = \frac{1.49}{n} R_h^{2/3} S^{1/2} = \frac{1.49}{n} \left(\frac{iT_t}{60(12)} \right)^{2/3} S^{1/2} \tag{3.13}$$

in which i is rainfall intensity [in./hr], T_t is travel time [min], S is the average slope [ft/ft], and L is the flow length [ft]. Solving for the travel time yields:

$$T_t = \frac{0.938}{i^{-0.4}} \left(\frac{nL}{\sqrt{S}} \right)^{0.6} \tag{3.14}$$

Equation 3.14 requires the rainfall intensity i for the time of concentration. Since T_t is not initially known, it is necessary to assume a value of T_t to obtain i from a rainfall IDF curve and then compute T_t. If the initial assumption for T_t is incorrect, then a new estimate of i is obtained from the IDF curve using the computed value of T_t. The iterative process should be repeated until the value of T_t does not change. Generally, only one or two iterations are required.

To bypass the need to solve Equation 3.14 iteratively, Welle and Woodward (1986) assumed a power-model relationship between rainfall intensity and rainfall duration. Using a return period of two years, they substituted the 2-year, 24-hour rainfall depth for the rainfall intensity i and derived the following alternative model for Equation 3.14:

$$T_t = \frac{0.42}{P_2^{0.5}} \left(\frac{nL}{S^{0.5}} \right)^{0.8} \tag{3.15}$$

in which L, S, and T are as defined in Eq. 3.13, P_2 is the 2-year 24-hr rainfall depth [in]. Equation 3.15, which is presented in USDA-NRCS- NEH Part 630 Chapter 15 (USDA, 2010), has the important advantage that an iterative solution is not required.

In addition to the previously mentioned assumptions, these two kinematic wave equations assume the following: (1) constant rainfall intensity, i; (2) no backwater effects; (3) no storage effects; (4) the discharge is only a function of depth, for example $q = ay^b$, and (5) planar, non-converging flow. These assumptions become less realistic as the slope decreases, the surface roughness increases, or the length of the flow path increases.

The overland or “sheet flow” Manning n values for use with Equations 3.14 and 3.15 are given in Table 3-3 and are for very shallow flow depths. These values reflect the effects of rain drop impact; drag over plane surfaces; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. The 24-hour rainfall depth P_2 for Equation 3.15 can be computed as the product of 24 hr and a 24-hour intensity obtained from an IDF curve for the 2-year return period.

Table 3-3: Manning’s Roughness Coefficients “n” for Sheet Flow

Surface Description	**N**
Concrete, Asphalt, bare smooth ground	0.011
Gravel, rough ground	0.02
Fallow (no residue)	0.05
Cultivated Soils:	
Residue cover > 20%	0.06
Residue cover < 20%	0.17
No-till Cultivated (corn–mature growth)	0.30
Cultivated (corn-mature growth)	0.50
Cultivated – fallow (no residue)	0.60
Soybeans (full growth)	0.15
Grass:	
Short and sparse	0.15
Dense turf (residential lots & lawns)	0.24
Very dense, tall, rough surface, uncut	0.40
Short Pasture grasses	0.10
Woods:	
Light undergrowth	
Dense undergrowth	0.40
	0.80

(The values in this table are a composite of information compiled by Engman [1986] and USDA [2010])

3.4.6 Shallow Concentrated Flow

The shallow concentrated flow segment of the time of concentration is generally derived using Figure 15-4 of the NEH Part 630 chapter 15 (USDA, 2010) or similar graphs. The flow velocities of Figure 15-4 are computed using the Manning’s equation; and the information in Table 15-3 of NEH 630 Chapter 15. The selected values of the Manning n are those normally expected for channel flow.

Use of the NEH Part 630 Chapter 15 graph (and the values of n and R listed above) may underestimate the travel time by overestimating the flow velocity for upper reaches of the shallow concentrated flow path. In shallow depths the hydraulic radius approaches the depth of flow. In this shallow flow range the n value should represent a higher resistance than that which would be used for channel flow. For example, a wide grass swale with flow depths of less than 0.5 feet and grass 6-inches high or more, the n value may fall between the 0.24 value for sheet flow and the 0.05 value for channel flow. In this case the designer might select an n value of 0.10 which better represents this shallow concentrated flow.

For more insight on the behavior of the Manning n in grassed channels, the reader should examine pages 179-188 in Chow (1959) which discuss the extensive experimental work of W.O. Ree (1949). Ree's experiments showed that Manning roughness coefficients varied with the type, density and height of grass and the product of the velocity and hydraulic radius. Shallow depths with low velocities produced roughness coefficients as high as 0.5.

3.4.7 Open Channel Flow

Estimating the travel time through the main stream requires the user to model the length, slope, roughness and the typical bankfull cross section. While a good map is assumed to provide a reasonable estimate of the length and slope of the stream, it is very difficult to select the Manning roughness coefficient and the "typical" cross section. Even if one uses stream gaging to determine a roughness coefficient at a point, the coefficient is likely to be different at another discharge or at another point along the stream. The cross section varies significantly along the stream, so it is difficult to determine which is the "typical" section. Errors in the cross selections can lead to incorrect estimates of the time of concentration and storage conditions and, therefore, lead to peak predictions that are too high or too low.

3.4.8 Length and Slope of Streams

The Panel recommends that the USGS 1:24,000 quadrangle sheets or comparable digital terrain data be the standard for determining the length and slope of streams used to estimate part of the time of concentration. It is recognized that the 1:24,000 scale cannot adequately represent the meanders of many streams. Thus, the estimated length may be too short and the slope too steep. When field investigations indicate that this may be a problem, the user should seek a larger scale map or support changes through additional field investigations or aerial photography.

3.4.9 Open Channel Manning Roughness Coefficient

There are two major uses of Manning roughness coefficient in WinTR-20. One is estimating the Manning n for the channel flow segment for the calculation of travel time and time of concentration. The other is estimating the Manning n for representative cross sections used for routing reaches.

The channel flow segment for the calculation of travel time and time of concentration is concerned primarily with the Manning n for the bankfull cross section, whereas the Manning n for the representative cross section for a routing reach applies to the complete cross section including channel and flood plain. Estimating Manning n for representative cross sections for reach routing is discussed in this section.

The Manning roughness coefficient is a very difficult parameter to estimate and can cause significant changes in the estimates of peak discharge. Even if estimates are based on carefully measured field data, the n would probably change if the measurements were made at a different discharge or at another cross section.

A study conducted by the US Army Corps of Engineers Hydrologic Engineering Center (USACE, 1986) explored the question of uncertainty in roughness coefficient estimates by asking their staff and training course participants to estimate roughness coefficients for several natural streams given photographs and descriptions of the streams. This effort found that the estimates by the participants were approximately lognormally distributed with a standard deviation given by the equation

$$SD = n(e^{[0.582+.10\ \ln(n)]2} - 1)^{0.5} \tag{3.16}$$

The equation indicates that an average estimate of n = 0.04 has a standard deviation of 0.011. Thus, if the average estimate of a group of experienced designers is n = 0.04, we can anticipate that their estimates will scatter, with approximately 68% of their predictions being between n = 0.029 and n = 0.051.

A number of tables list Manning roughness coefficients for different types of artificial and natural channels. The table presented by Chow (1959) in Chapter 5 is an excellent source. Chow points out that these values should be adjusted to reflect local conditions such as channel irregularity, alignment, silting and scouring, obstructions, meandering, suspended material and bed load. These and other corrections are discussed in considerable detail in Chow’s Chapter 5. Supplement B of NRCS National Engineering Handbook Section 5 “Hydraulics” (1956) provides a manual procedure to estimate Manning’s n value for stream cross section. Other references include Arcement and Schneider (1984), Fasken (1963) and Barnes (1967).

Still another problem arises when field investigations indicate that the roughness varies significantly from one section of the stream to another. In these cases, it may be necessary to break the stream into segments and compute the travel time for each. In the absence of field investigations, an initial Manning n value of 0.05 should be used for the bankfull cross sections for estimating the time of concentration.

3.4.10 Bankfull Cross Section

Another factor contributing to changes in the peak flow prediction is the "typical" bankfull cross section selected to determine the velocity and, therefore, one component travel time of the time of concentration. For example, selection of a cross section near the outlet of the watershed may result in a channel velocity that is significantly different from that predicted by the use of a cross section chosen from a point about half-way up the stream. Increasing the hydraulic radius will result in a higher velocity and corresponding shorter the time of concentration. Because the cross section varies from point to point along the channel, it is quite difficult to decide which is the representative cross section. Thus, the user must recognize the importance of the representative cross section when calibrating the WinTR-20 model as described in Chapter 4.

If it is not practical to survey bankfull cross sections, an alternative is to use regional regression equations that relate the bankfull depth, width and cross sectional area to the area of the upstream drainage basin. Figure 3-5 shows an example of channel cross-section regional regression equations developed for MDOT SHA by McCandless and Everett (2002), and McCandless (2003a and 2003b). Appendix 4 presents the equations that are accepted by MDOT SHA and MDE WSA. Dunne and Leopold (1978) present a similar set of relations and Rosgen (1996) includes several examples of findings similar to Figure 3-5.

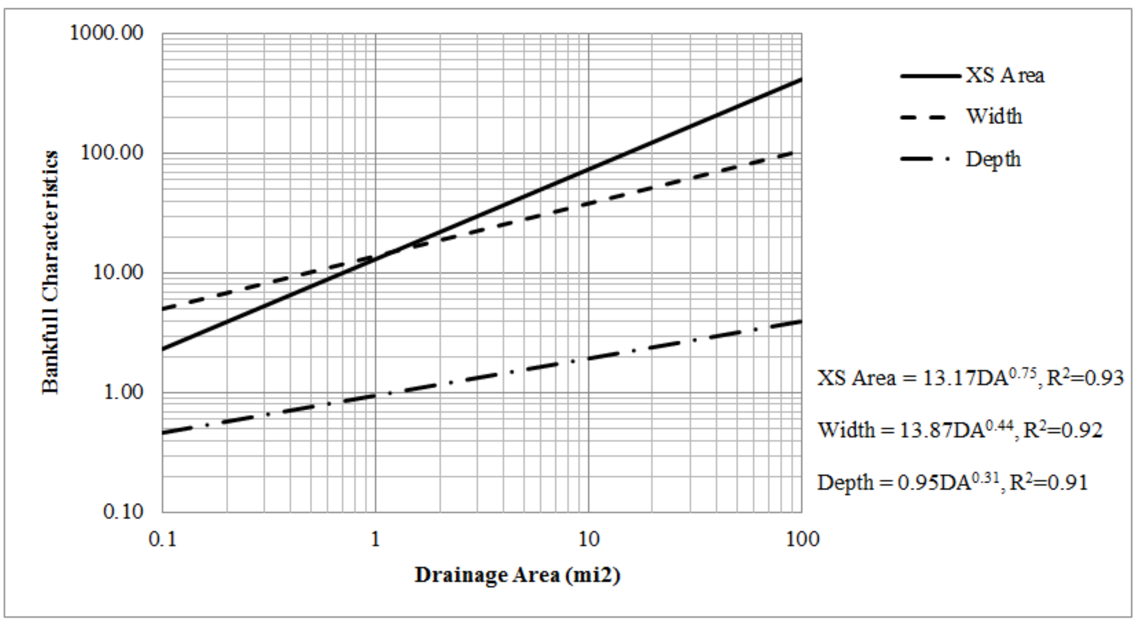

Figure 3-5: Bankfull Characteristics for Selected USGS Sites in the Maryland Piedmont

Figure 3-6 and Figure 3-7 indicate that time of concentration differences associated with cross-sections defined through the use of regional regression equations, as opposed to surveyed cross sections, may be less than the differences associated with different roughness coefficients. In Figure 3-6, the Siebach (1987) S-curve (time-area curve) defining time of concentration used travel times computed with surveyed, bankfull cross sections. The Dunne and Leopold curve used cross sections that were defined with their regional regression equations that estimated bankfull width, area and depth as functions of the watershed area. The S-curves used to estimate the time for concentration in Figure 3-7 used surveyed cross sections with the Manning roughness coefficient being varied.

The two figures indicate that errors in the Manning roughness coefficient can cause larger errors in the time of concentration than the changes associated with differences between surveyed and regression defined bankfull cross sections. This is to be expected because the channel velocity varies linearly with the roughness coefficient and to the 0.667 power of the hydraulic radius.

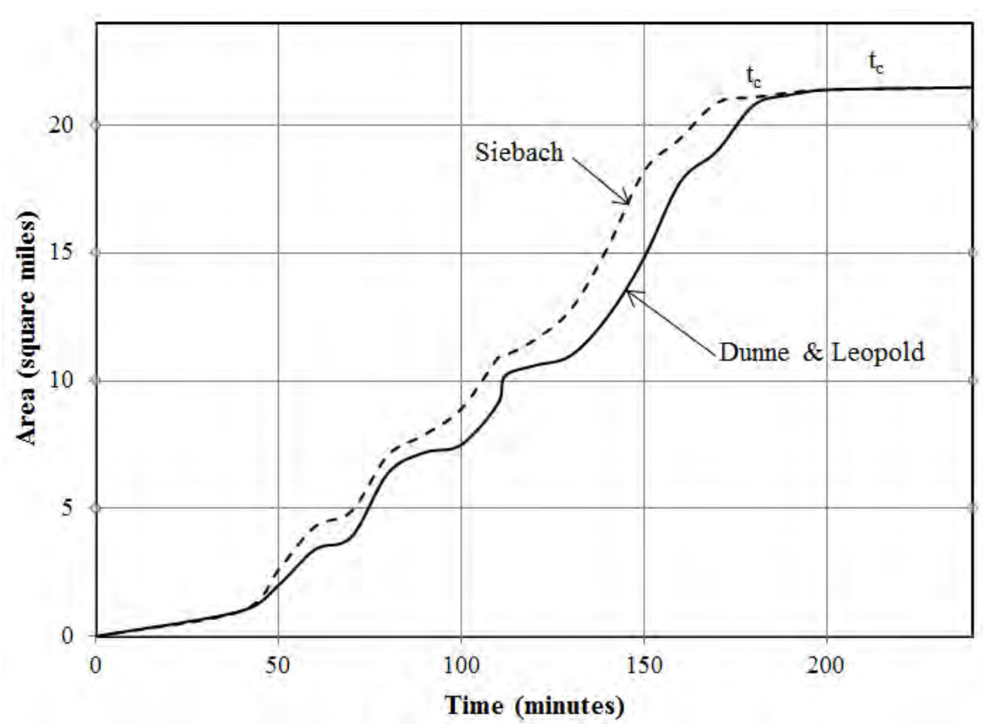

Figure 3-6: Time-Area Curves Using Surveyed and Regression Equation Defined In-Bank Cross Sections (n= 0.04)

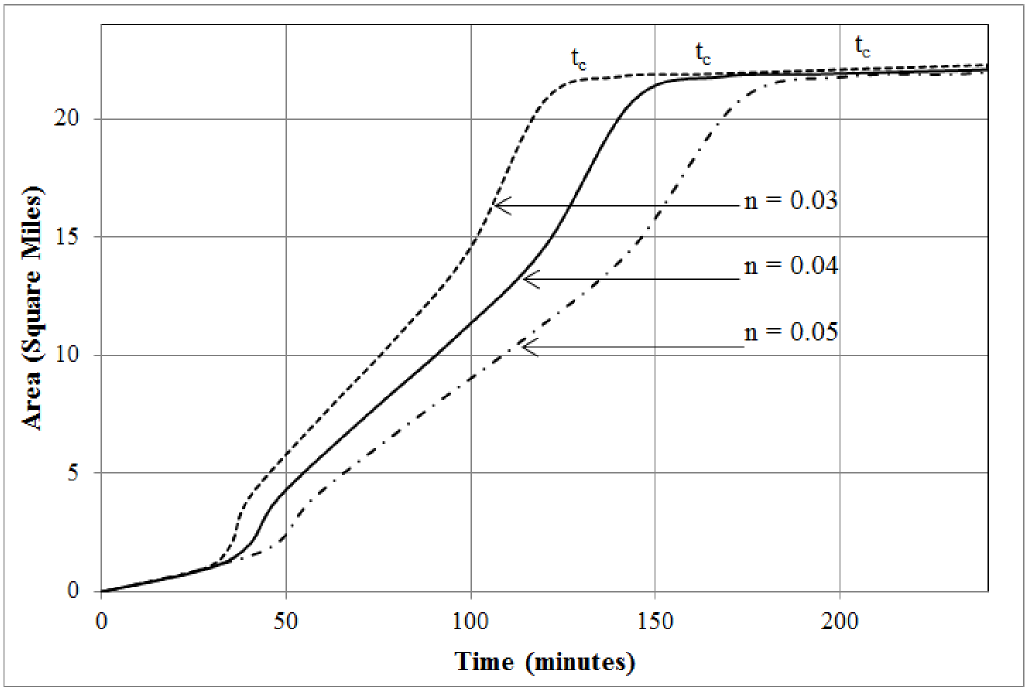

Figure 3-7: Time-Area Curves Using Surveyed In-Bank Cross Sections and Indicated Manning Roughness Coefficients

As can be seen from the above discussion, accurate estimates of the time of concentration are difficult to obtain because of the large uncertainty in the parameters used to compute the time of concentration. Thus, there needs to be an alternative approach that can serve to define upper and lower bounds for time of concentration. Regression models that estimate time of concentration based on watershed characteristics provide an attractive approach. Regression equations described in Appendix 6 can be used to check realistic bounds for the time of concentration.

3.5 SUBDIVIDING INTO SUB-WATERSHEDS AND ROUTING

If the watershed is large or has tributary drainage areas that have land/soil complexes that differ from each other, the watershed may be divided into sub-watersheds. In this approach, the dimensionless UHG uses the area, curve number and time of concentration for each sub-watershed to develop storm hydrographs. The hydrograph for each sub-watershed is then routed through the stream network to the outlet of the overall watershed. Even if the watershed is not especially large or heterogeneous, calibrating to the FRRE (as described in Chapter 4) may require subdivision in order to model the attenuation provided by the flood plain.

No “magic number” exists to define a small versus a large watershed. A watershed might be considered small if the land phase processes — overland and shallow confined flow — dominate the peak discharge and the shape of the runoff hydrograph. A watershed might be considered large if the translation and storage provided by the stream network provide significant attenuation or modification to the storm hydrograph. A large watershed by this definition could require subdividing and flow routing.

3.5.1 How Many Sub-watersheds

Part of the decision controlling the subdivision of the watershed is tied to the heterogeneous nature of the watershed. A watershed should be subdivided if peak discharges or hydrographs are needed at points within the watershed in addition to the peak or hydrograph at the watershed outlet. In the past NRCS has used a size criterion: if the drainage area exceeds 20 square miles, subdivision should be considered.

There does not appear to be a rule that one can apply to determine an optimal number of subdivisions for a watershed of a given size or set of topographic characteristics. Maryland designers must calibrate against the FRRE (as described in Chapter 4) to ensure that their subdividing approach is appropriate. The Panel recommends the paper by Casey and others (2015) for further guidance on subdivision when using WinTR-20.

3.5.2 The Representative Routing Cross Section

Bankfull and over-bank cross sections often show tremendous variations along a stream reach. Selecting the representative cross section for use in developing the required stage-area-discharge relation for the routing reach is a very difficult task. If the flood plain is too narrow, the peak will be too high and if it is too wide, the peak will be subject to too much attenuation.

An alternative to the use of field surveys to define typical cross sections is to digitize along transects drawn on maps, perpendicular to the stream. In many areas, 1:2,400 or similar scale maps are available. Transects on these maps can provide an excellent base for routing sections. The bankfull portion of the section is generated by the regression equations discussed in Section 3.4.8. As shown by Figures 3-8 and 3-9, even a 1:24,000 scale map can be used in areas where there is good topographic definition.

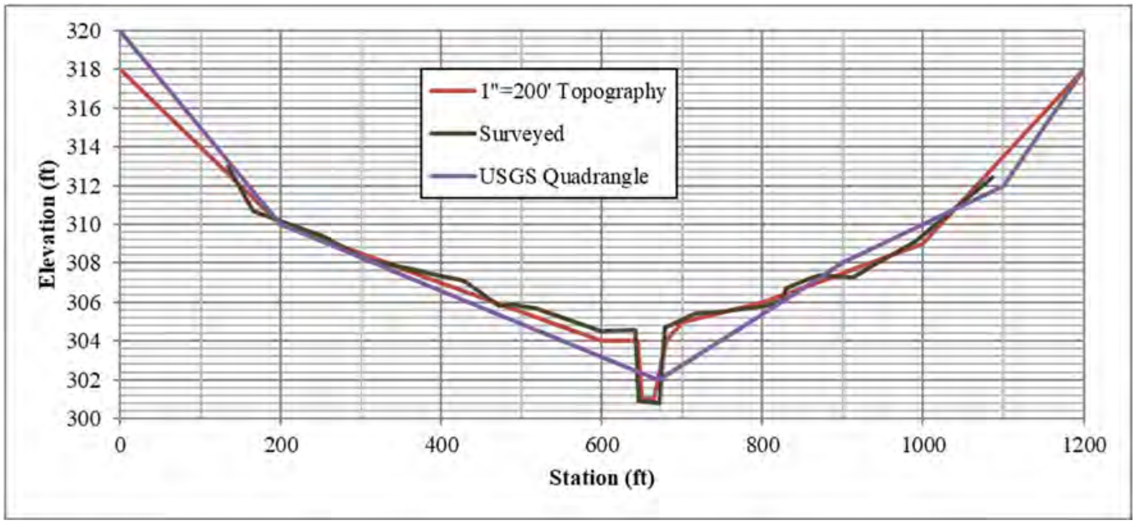

Figure 3-8: Stream cross-section (transect) as determined from in-situ survey (black), USGS quadrangle map (1:24,000) and 1:2,400 topographic data.

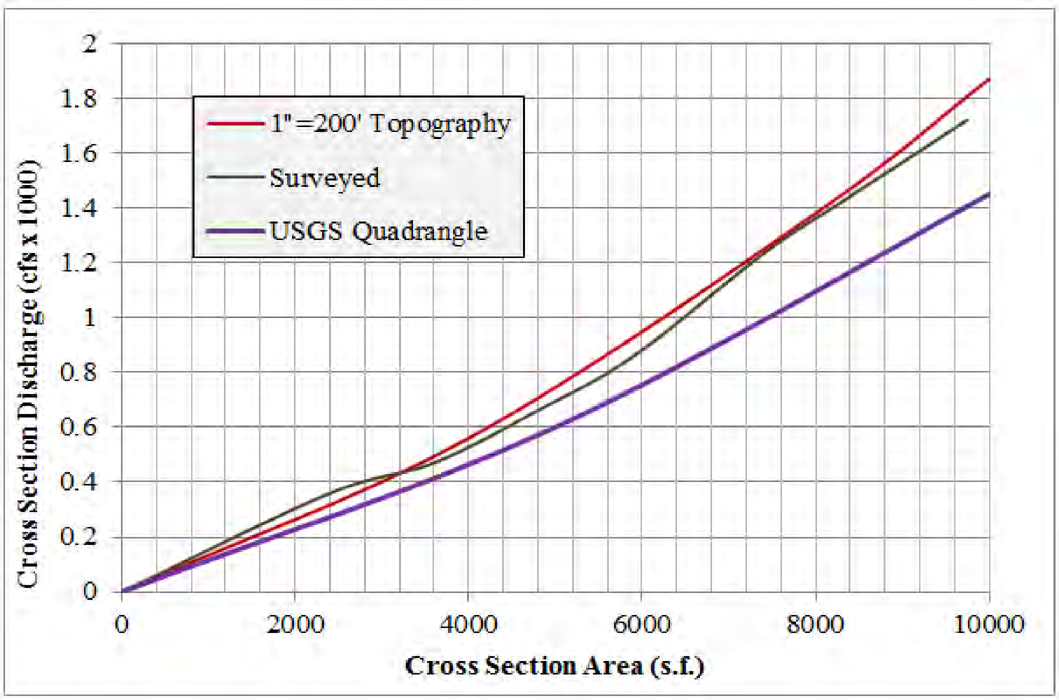

Figure 3-9: Discharge-Area Curves for Surveyed and Contour Defined Synthetic Cross Sections

Assume that we are confident that the "correct" representative cross sections for the flood routing component of the WinTR-20 have been chosen. We are now faced with the problem of selecting the Manning roughness coefficients required for the stage-area-discharge relationship. Section 3.4.9 discussed the difficulties associated with the definition of the in- bank roughness and illustrated the impact of the roughness on the time of concentration.

3.5.3 Manning n for the Representative Routing Cross Section

Estimating the over-bank roughness involves more uncertainty than the in-stream bankfull coefficient because of the extremely limited amount of data collected for flow in a flood plain. Chow's (1959) table suggests flood plain Manning roughness coefficients that range from 0.02 to 0.20.

For the representative cross section for reach routing, different Manning n values are estimated for the channel and overbank areas to the left and right of the channel. Arcement and Schneider (1984) include photographs of flood plains with Manning n estimates from 0.10 to 0.20.

3.5.4 Channel Routing Techniques

The WinTR-20 routing module is the Muskingum-Cunge (M-C) approach. The M-C method is a variation of the Muskingum method that has been used for many years in river forecast operations by the National Weather Service, U.S. Army Corps of Engineers and similar organizations. Both the M-C and Muskingum methods use a series of routing coefficients that are defined by the routing period, Δt, a travel time constant for the routing reach, K, and a weighting factor, X. In the traditional river forecast environment, there are usually recorded inflow and outflow hydrographs that can be used to define K and X and earlier experiences on the river can evolve the optimal value of Δt. Concise summaries of the two routing methods can be found in Bedient and Huber (1992).

In most cases there will be no records of inflow and outflow hydrographs at the point of interest that can be used to determine K and X. Without historic records of inflow and outflow hydrographs, K is estimated by the length of the routing reach and the celerity of a small gravity wave moving through the reach. The length of the routing reach is a decision made by the user. The celerity of the small gravity wave requires an estimate of the average velocity, width and depth of flow through the routing reach. The first major difference between the Muskingum and M-C procedures is that the M-C procedure includes an equation to estimate X from cross section hydraulic properties and reach length. The value of X is defined from the routing reach length, average width, average slope, celerity of a gravity wave, and the peak discharge entering the reach. The second major difference between the Muskingum and M-C is that with the M-C there is a possibility of breaking the reach into a number of routing steps.

The M-C method was selected by NRCS because it was concluded that it would overcome some of the problems associated with the former Modified Att-Kin module in TR-20. A paper by Merkel (2002) outlines the studies that NRCS made before selecting the M-C procedure. The M-C procedure was compared to the full dynamic wave routing for a large number of cross section shapes, reach lengths, and slopes. Note that all the parameters in the previous paragraph have feedbacks involving many of the same issues that impact the performance of the previously-applied Modified Att-Kin method. For example, to estimate the coefficients K and X, the user must have decided on the length of the routing reach and must still make judgment decisions on the Manning n and "average cross section" so that the celerity can be computed. The values for each of these elements are difficult to determine.

3.6 THE DESIGN STORM

The WinTR-20 model requires that the user define the total depth of rainfall, the duration of the storm, and time distribution of cumulative rainfall depth within the storm. Before NOAA Atlas 14 Volume 2 was published, the usual approach was to accept one of the "standard" design storms such as the NRCS Type II, 24-hour storm. With publication of Atlas 14 Volume 2, new temporal distributions were developed (Merkel and others, 2006).

A major assumption used in the development of the design storm is that the 5-minute through 24-hour rainfall values have the same return period. In other words, the 5-minute 100-year rainfall, 10-minute 100-year rainfall, etc, up to the 24-hour 100-year rainfall occur within the same storm. A second assumption is that the durations are nested, with the most intense rainfall at the storm center (12 hours) and the intensity gradually reducing symmetrically from the storm center to the starting and ending times (zero and 24 hours), (Merkel and others, 2006). Details on the procedure and an example based on Howard County, Maryland are included in Appendix 7. This procedure has been incorporated into the WinTR-20 software so the user does not need to do significant amounts of hand or spreadsheet calculations.

The WinTR-20 software uses the watershed area and time of concentration to convert the dimensionless UHG to a UHG. Then the cumulative rainfall distribution and runoff curve number are used to generate a series of cumulative runoff values. The cumulative runoff values for the design storm are then convolved with the UHG to produce a storm hydrograph. If the 100-year, 24-hour depth of rainfall is used to define the intensities in the design storm distribution, the "design expedient" typically accepts the peak discharge generated by the WinTR-20 as an estimate of the 100-year frequency peak discharge to be used in design. **It must be emphasized that the WinTR-20 computes an estimate of the peak discharge caused by a synthetic 100-year storm that is based on rainfall records and not an estimate of the peak discharge based on streamflow records. The two discharges may differ significantly. The Panel's recommended calibration against one of the methods described in Chapter 2 of this report (as described in Chapter 4) is intended to reconcile some of the disagreement.**

Decisions that define the storm input are very important because the performance of WinTR-20 is very sensitive to the structure of the rainfall input.

Segments of the NRCS 24-hour design storm should be used to develop synthetic storms having different durations. When developing a synthetic storm having a duration that is shorter than 24 hours, one should use the period that is distributed equally on each side of the steepest portion of the mass curve. For example, a six-hour storm would be based on the dimensionless intensities between T = 9.0 and T= 15.0 hours on the NRCS 24-hour storm distribution. A 12-hour storm would be based on the dimensionless intensities between T = 6.0 and T= 18.0 hours on the NRCS 24-hour storm distribution. Figure 3-10 illustrates the NRCS 24-hour storm used to generate the storm distributions having durations of 6 and 12 hours for a location in Howard County, Maryland (longitude -76.9862 and latitude 39.2922). An example of development of a 6-hour and 12-hour storm distribution based on a location in Howard County, Maryland is included in Appendix 7.

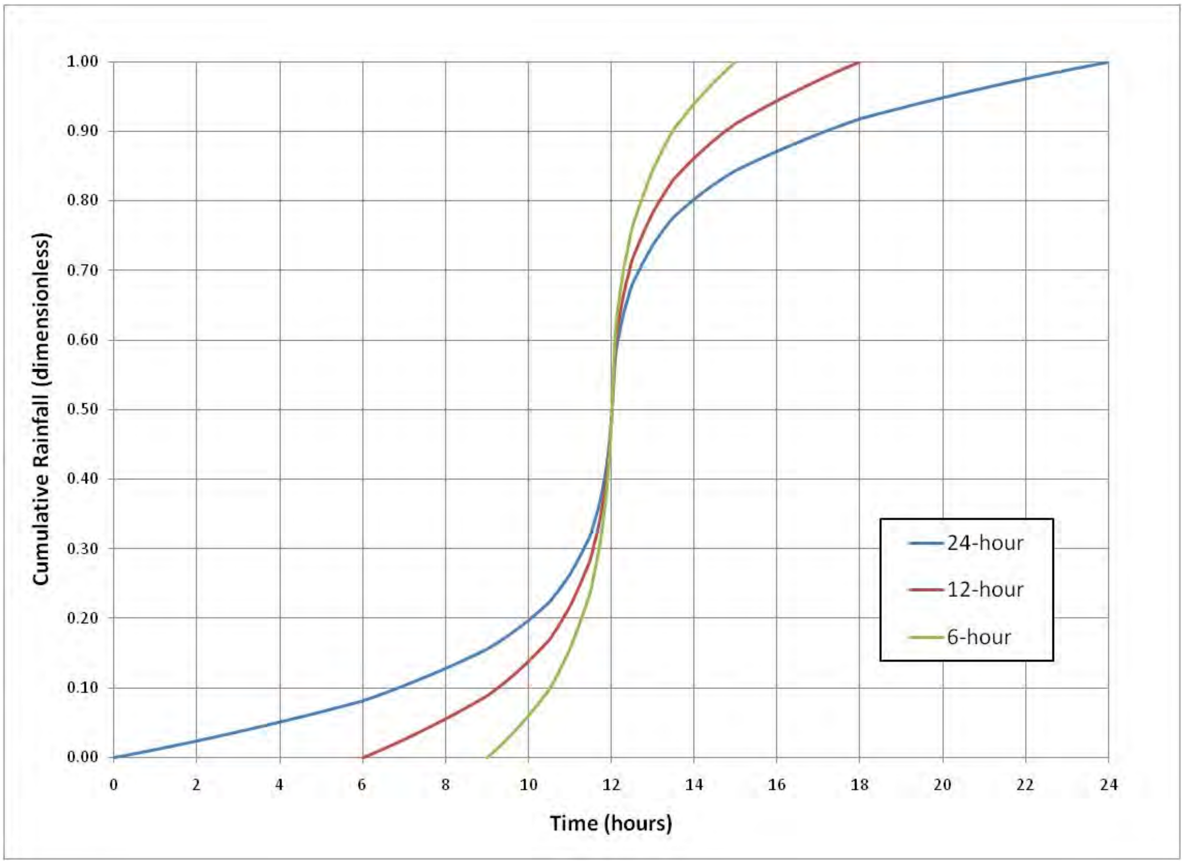

Figure 3-10: 6, 12, and 24 Hour Storm Distributions Howard County MD

Design storms having similar structures, but different durations, produce significantly different hydrographs and peak discharges when input to WinTR-20. As a consequence, there is uncertainty as to what storm duration should be used. The traditional practice in Maryland in the past has been to use the 24-hour Type II storm in all cases. However, the Type II storm distribution does not fit the data from NOAA Atlas 14 for the entire state of Maryland nor does it fit the NOAA Atlas 14 data for the 1-year to 500-year return periods. If ratios of shorter duration to 24-hour rainfall are computed at a point, there can be significant differences when compared to the ratios within the Type II storm distribution. For example, at a point in Howard County (longitude −76.9862 and latitude 39.2922) rainfall ratios are included in Table 3-4. The rainfall data used to develop this table are based on the partial duration series.

Table 3-4: Rainfall ratios based on NOAA 14 and Type II for a point in Howard County

Duration	Type II Ratio	1-year NOAA ratio	10-year NOAA ratio	100-year NOAA ratio
5 min	0.114	0.129	0.110	0.085
10 min	0.201	0.208	0.177	0.135
15 min	0.270	0.261	0.224	0.170
30 min	0.380	0.356	0.324	0.261
60 min	0.454	0.443	0.422	0.359
2 hour	0.538	0.530	0.511	0.456
3 hour	0.595	0.568	0.548	0.496
6 hour	0.707	0.708	0.682	0.636
12 hour	0.841	0.867	0.849	0.826
24 hour	1.000	1.000	1.000	1.000

Table 3-5 shows that the 100-year rainfall intensity is much less for the distribution based on NOAA 14 data than the Type II. The rainfall intensity for the NOAA 14 1-year storm is relatively close to the intensity of the Type II.

Table 3-5: Comparison of peak discharges between NOAA 14 and Type II storm distributions.

	Peak Discharge cfs	**Peak Discharge cfs**	**Peak Discharge cfs**	**Peak Discharge cfs**	**Peak Discharge cfs**	**Peak Discharge cfs**
	NOAA 14	Type II	NOAA 14	Type II	NOAA 14	Type II
Time of Con. hours	1-year	1-year	10-year	10-year	100-year	100-year
0.75	825	845	2715	3065	5166	7151
1.25	585	582	1984	2123	3933	4970
2.0	420	408	1450	1487	2968	3488
3.0	314	300	1090	1083	2299	2549

Table 3-5 was developed at the same location in Howard County, Maryland. It is based on a drainage area of 3.0 square miles and curve number 75. Short to long times of concentration were used to show the sensitivity of storm distribution to changes in time of concentration. As expected, the 1-year peak discharges are not significantly different between the two storm distributions. However, the NOAA 14 distribution produces 100-year discharges that are significantly lower. These results may not be generalized for the entire state of Maryland because a storm distribution based on NOAA 14 data depends on the relationship of 5-minute through 24-hour rainfall data at each location and return period.

Experiments conducted by the Panel demonstrate that the 25-, 50-, and 100-yr flood peaks predicted by the WinTR-20 model, using the 24-hour design storm duration and appropriate estimates of watershed parameters, agree reasonably well with the flood peaks predicted by the regression-based equations. However, such is not the case for more frequent storm events. The Panel's experiments indicate that the 2-, 5-, and 10-yr flood peaks generated by the WinTR-20 model using the 24-hour design storm duration are often significantly higher than those predicted by the regression-based equations. When shorter duration design storms, based upon center-peaking period of the Atlas 14 storm and meeting all of the conditions imposed by the Maryland IDF curve, are used for the 2-, 5-, and 10- year flood peaks, the WinTR-20 and regression estimates may be brought into closer agreement. Obviously, more research using NOAA Atlas 14 data is warranted. In the interim, the 10-, 5-, and 2-year storm events should be derived using either the 6-hour or 12-hour design storm duration (depending on time of concentration) if needed during the calibration process.

The depths of precipitation (partial duration) of a given frequency and duration vary considerably across Maryland. The depth of precipitation in a 100-year 24-hour storm varies from 5.4 inches in western Garrett County to 9.3 inches in Calvert, St. Mary's, Wicomico, and Worcester counties.

There appears to be a problem in applying WinTR-20 models in western Maryland. Peak flood flows predicted by WinTR-20 are often significantly higher than the estimates based on regression equations. Many of the USGS stream gages have operated in that region for more than 70 years. These gages simply have not measured peak flows as high as those measured in the central portion of the State. Analysis of eleven USGS gages in the Maryland Appalachian Plateau and Valley and Ridge provinces demonstrates that the observed maximum flows range between 83 and 300 cfs per square mile, with an average of 167 cfs per square mile. The minimum length of record is 17 years and the maximum length is 50 years. The average watershed area is 23 square miles. The same analysis conducted on six gages in the Maryland Piedmont indicates that the maximum flows vary from 319 to 780 cfs per square mile, with an average of 452 cfs per square mile. The minimum length of record is 12 years and the maximum length is 60 years. The average watershed area is 22.3 square miles. Based upon watershed characteristics alone, one would expect the steep mountain areas in western Maryland would yield higher peak flows than the Piedmont. However, indications are that flood producing rainfalls in western Maryland may be shorter in duration than those farther east. More specific research using NOAA Atlas 14 data is warranted in this regard. Therefore, if the flood estimates using the 24-hour storm do not lie between the regression estimate and the upper 68% limit, the analyst should use the 12-hour storm for the 25-, 50- and 100-year events and the 6-hour storm for the 2-, 5- and 10-year events.

NOAA Atlas 14 provides the user a choice between precipitation intensity based on Annual maximum or Partial duration time series. Partial duration precipitation values from NOAA Atlas 14 are recommended for design purposes.

Precipitation values available from NOAA Atlas 14 are point estimates. The typical storm is spatially distributed with a center area having a maximum rainfall and a gradual reduction of intensity and depth away from the storm center. The spatial distribution of rainfall within a storm generally produces an average depth over an area that is a function of watershed area and storm duration. Figure 3-11 is based on the areal reduction curves from US Weather BureauTP-40. The Panel recommends that the hydrologist adjust the design storm rainfall to reflect spatial distribution.

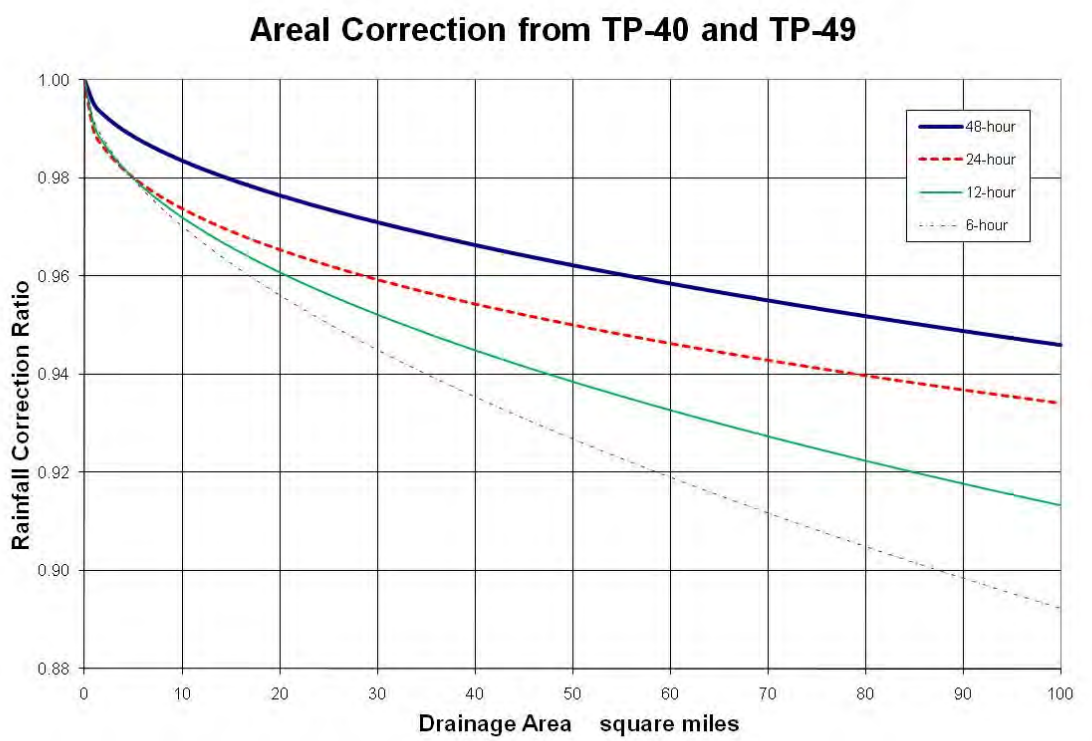

Figure 3-11: Areal Reduction curves based on TP-40

If the hydrologist is using GISHydro the adjustment is an option presented as a screen prompt and should be implemented for all watershed studies. If the hydrologist is conducting a study outside the GISHydro environment, the adjustment for spatial distribution should be made using equations 3.17 – 3.20.

$RF = 1 - \alpha A^{\beta}$	(6 hour)	(3.17)
$RF = 1 - (\alpha/2)A^{\beta} - (\phi/2)A^{\rho}$	(12 hour)	(3.18)
$RF = 1 - \phi A^{\rho}$	(24 hour)	(3.19)
$RF = 1 - \gamma A^{\kappa}$	(48 hour)	(3.20)

where A is the area [square miles], $\alpha = 0.008245$, $\beta = 0.558$, $\phi = 0.01044$, $\rho = 0.4$, $\gamma=0.005$, and $\kappa=0.5169$.

CHAPTER FOUR

4 Calibration of WinTR-20 with Statistical Methods

4.1 OVERVIEW

The hydrologic analysis of MDOT SHA bridges and culverts must examine the behavior of the structure and local stream conditions under both existing and ultimate development watershed conditions. Because two land cover and flow path conditions are involved, the basis for these hydrologic analyses must be a deterministic model that can simulate the major runoff processes for both existing and future conditions. The recommended approach is to first select field and map defined parameters that describe the runoff processes for existing watershed conditions. After the designer is satisfied that the model provides a realistic representation of the existing watershed conditions, the impact of ultimate conditions can be simulated by adjusting the input parameters to reflect future land cover and flow path modifications.

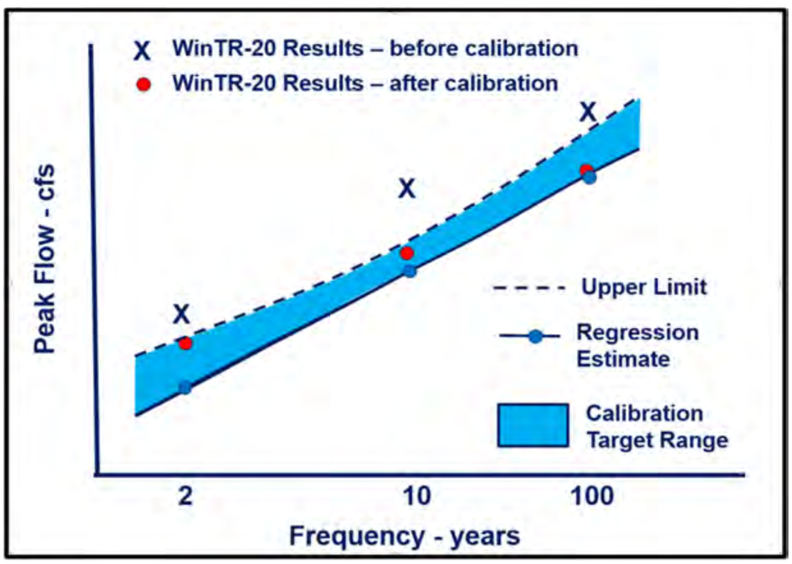

Figure 4-1: Over-prediction behavior of WinTR-20 for all return periods

The NRCS WinTR-20 computer program is a well-established deterministic model that has an extensive history of use in Maryland. However, the WinTR-20, as with all deterministic models, is sensitive to the values of the input parameters. In most instances, the input parameters are difficult to accurately determine. As discussed earlier, the WinTR-20 model historically has a tendency to over predict peak flows for most return periods. This behavior is illustrated by Figure 4-1. The Panel has concluded that this tendency to over predict can be reduced through calibration. Thus, in order to provide the designer with confidence that the input parameters selected are representative of the existing watershed conditions, the Panel recommends that the WinTR-20 peak discharges for existing watershed conditions be calibrated against one of the statistical methods described in Chapter 2. In most cases, the statistical method most applicable is the regression equations since most hydrologic analyses are performed on ungaged streams. The WinTR-20 model will be accepted as calibrated if the peak discharges for the design frequency event are in the window between the statistical best estimate and an upper limit of plus one standard error of prediction as defined in Chapter 2. The calibration window is based on the historical tendency of the TR-20 model to predict larger discharges than the regression equations. If the watershed conditions are such that a calibration cannot be achieved in accordance with the procedures defined below, the designer will explain why the calibration cannot be accomplished and what approach will be followed to generate the required flows.

In recent years, the regression equations were updated more frequent and the equations were based on both rural and urban watersheds in the Piedmont-Blue Ridge and Western Coastal Plain Regions. Sometimes the TR-20 model will predict discharges less than the regression equations. Under these conditions, the input data to the TR-20 model should be adjusted within reasonable bounds to get the flood discharges close to the regression estimates. The reasons why the TR-20 model is predicting lower discharges should be documented.

In many cases, the designer will not be able to choose one calibration adjustment for the WinTR-20 to bring the peak flow rates within the regression equation target range for all storm frequencies. For example, a calibration adjustment needed to bring the 100-year storm within the target range may not be sufficient to bring the 50-, 10-, or 2-year storms within their respective target ranges. In these cases, it will be necessary to use a progression of calibration adjustments in a logical sequence

Table 4-1 suggests a logical progression of calibration steps for multiple storm frequencies. It can be used as a guide for the designer with the understanding that there may be other logical calibration progressions that are more suitable for a particular watershed.

The Panel emphasizes that all input parameters to WinTR-20 must be consistent with accepted hydrologic practice. Thus, all WinTR-20 computations will be supported by documentation that lists the values of (1) category curve numbers; (2) the quantities used to define the time of concentration, and (3) the watershed segmentation and stage-area-discharge relations if routing is involved. This documentation will explain the decision-making process behind the selection of each input quantity.

The following sections examine the types of errors that may occur in the definition of inputs to the WinTR-20 model and the procedures to follow in making adjustments to achieve calibration. Because so few watersheds of concern to the MDOT SHA are located at a USGS gage or at a point that will allow gage transposition, the emphasis of this chapter is on calibration against the Fixed Region Regression Equations documented in Appendix 3. Figure 4-1 illustrates the situation that often occurs where the WinTR-20 model estimates are higher than the regression estimates. The WinTR-20 estimates in Figure 4-1 are actually greater than the regional regression estimates plus one standard error of prediction. The objective of the calibration of the WinTR-20 model is to modify the model input parameters to produce estimates of the flood discharges that are between the regression line and the upper limit represented by plus one standard error of prediction. This chapter provides guidance on modifying the model input parameters.

Table 4-1: Logical Progression of Calibration for Multiple Storm Frequency Models

Calibration Variable/ Input Element	Application
T_c (Time of Concentration variables)	Same for all storms
RCN conditions (good-fair-poor)	Same for all storms
Reach Length	Actual channel and flood plain lengths may be greater than values measured from maps or digital terrain data
Dimensionless Unit Hydrograph	Same for all storms
Rainfall Table – 24-hr duration	Use for 25-year to 500-year storms
Rainfall Table – 12-hr duration	May use for the 2- through 10-year storms if the time-of-concentration is between 6 and 18 hours.
Rainfall Table – 6-hr duration	May use for 2-, 5- and 10-year storms if time-of-concentration is less than 6 hours
Rainfall depth	May use the upper 90-percent confidence limit for rainfall depth as an alternative to best estimate
ARC (Antecedent Runoff Condition)	Use 2 for 25-year and greater return period storms. May use < 2 for the 2-year to 10-year storms provided that it does not decrease for greater return period storms. ARC of > 2 may be considered for storms of 200+ years.

4.2 SIZE AND CHARACTERISTICS OF THE WATERSHED

For watersheds greater than about 300 square miles in size, WinTR-20 models are not recommended. The NRCS developed the dimensionless UHG from data collected on relatively small watersheds. On most large watersheds, significant peak flow attenuation caused by the channel network may not be incorporated into the NRCS dimensionless UHG. Also, the assumption of homogeneous rainfall over the watershed becomes less likely for very large areas. Thus, the validity of WinTR-20 applications on large watersheds is questionable. Moreover, the effects of ultimate land use conditions on peak flows generally are muted on very large watersheds.

Before any calibration of the WinTR-20 is attempted, care should be exercised to ensure that the characteristics of the watershed are within the limits of the statistical data set used to develop the regression equations. Calibration will not be valid if there are other factors that are not accounted for in the Fixed Region Regression Equations such as ponds, wetlands storage, or structures that significantly change the natural flow characteristics of the watershed. For the Appalachian Plateau and Eastern Coastal Plain Regions, the regression equations are not valid if existing impervious area exceeds 10%. This is because these regions contain insufficient gage data for urban (≥ 10% impervious) watersheds for the development of regression equations with an urban factor.

4.3 UNDERSTANDING ERRORS

The construction of any deterministic model involves the selection of certain input values. The selection estimate or measurement of any value includes the possibility of several types of errors. These can be labeled: Random (sometimes more and sometimes less), Systematic (always more or always less), and Cumulative (small systematic errors that add up to large systematic errors). Each variable entered in the WinTR-20 model can have one or more of these errors. As with the regional regression equations, the selected value for any WinTR-20 input variable represents the “best educated guess.” Unfortunately, unlike the standard error of the regional equation, the standard errors of WinTR-20 input variables are not defined. However, with experience and the guidelines of standard practice, designers can estimate the range of reasonable WinTR-20 input values and confine their choices to those within this range. For example, a Manning’s roughness coefficient for a natural stream channel might be 0.05. Estimates that are 0.07 and 0.03 still appear to be within a reasonable range while 0.3 and 0.002 are not. In general, the designer should select the variables with large potential systematic errors as the most likely values to calibrate or adjust.

The WinTR-20 input variables and a description of the types of errors that are inherent in their estimate follows, along with recommendations regarding adjustments for calibration to more closely simulate the results of the Fixed Region Regression Equations. Table 4-2 is a summary of these variables and their inherent errors. It also shows the limits of calibration adjustments of the input variables. They are guidelines only and not intended as absolute limits.

4.3.1 Drainage Area

Assuming that both the map or digital terrain data used to delineate the drainage area and the measuring approach are accurate, the estimation of the drainage area includes a random error. When delineating areas, the designer should check for random errors by ensuring that the sum of all sub-areas equals the total drainage area. Adjusting the size of a drainage area is seldom justified unless the watershed includes Karst topography or non-contributing drainage areas. In some unusual cases such as for extractive land use (mining), depression areas will not contribute to watershed runoff at the 2-year event but may contribute at the 50- or 100-year event.

4.3.2 Runoff Curve Number

The error in selection of a runoff curve number (RCN) value is random. The NRCS handbook (NEH Part 630, Hydrology, Chapter 9) shows the acceptable range of values for each land cover. Those for croplands and natural ground cover are based on hydrologic conditions such as fair, poor, or good. In cases where one land cover is predominant, a potential for a systematic error exists because of the impact of the selection of one significant value rather than the distribution of small random errors in a varied land cover model.

RCN value(s) can be adjusted to match a measured runoff volume provided that the resulting RCN falls within the logical limits of their respective ARC (Antecedent Runoff Conditions) limits. Consideration should be given to the use of ARC ≤ 2 for the frequent events (1- up to 10-year storms). The reasoning is that these small storms are usually the result of short duration summer thunderstorms without the preceding ground wetting light rain. Greater storms (10-year and larger) are generally related to cyclonic storms of 12- to 48-hour duration where several hours of rain precede that of the flood producing rain intensities. In this case, the ARC value is set at 2. An ARC > 2 may be considered for storms of 200-year frequency or greater.

4.3.3 Land Use Categories and RCN Values

Land use categories such as those used in GISHydro, are defined by the Maryland State Department of Planning. They are intended to be used for planning studies that extend beyond hydrologic modeling. The term <u>land use</u> is intended to describe a function rather than a hydrologic response. Because of this, there are several categories of land use that are not sufficiently descriptive of their corresponding hydrologic response and, if other than an insignificant part of the watershed, may require a more detailed evaluation and sub-classification. The following is a list of those land use categories that have these characteristics.

1. *Low Density Residential.* Residential lots of 2 acres and greater may produce a hydrologic response that is characteristic of other dominant land cover such as forest (or woods), meadow, grass, cropland, etc. If this land use is a significant portion of the watershed, an examination of aerial photographs may help better define the ground cover conditions.

2. *Institutional.* Institutional land use incorporates a wide range of uses including governmental offices, educational facilities, health facilities, etc. that exhibit land cover ranging from parking lots to woods. It is important to examine available

mapping and aerial photographs to subdivide this category to better simulate the hydrologic response.

3. *Extractive.* Extractive land use is defined by mining operations. There is a potential of a wide range of hydrologic responses depending on the nature of the type of mining. In particular, strip mining may respond as bare ground while a limestone quarry may act as a reservoir without an outlet. If this land use is a significant part of the watershed, the analyst should determine the particular type of mining. Many large mining operations include areas of active disturbance, areas of reclaimed land, and undisturbed areas of future excavations. More significantly, the hydrologic response of a mining operation is often determined by the way runoff is handled at the site. This could include peak storage, pumping, diversion swales and berms. To conform to the environmental regulations, each active mining operation must have a stormwater, sediment control, and drainage plan that will define these elements. These plans are filed with the Maryland Department of the Environment, Bureau of Mines.

4. *Transportation.* Transportation includes major highways, interchanges, storage and maintenance yards for government highway agencies, Metro facilities, rail yards, and similar uses. Large interstate highway interchanges may include higher proportions of grass than pavement as compared to the highway right-of-way alone. Storage yards may be predominantly impervious surface while rail yards may be compacted gravel. Aerial photos and site inspections will enable the analyst to subdivide this category to better define the hydrologic response.

The default values of RCN for the above land uses in GISHydro have been derived using assumed percent imperviousness. These default values may not affect the runoff hydrograph if the corresponding areas are insignificant relative to the total watershed area. However, the engineer must decide if this is the case or provide more appropriate RCN values as described above.

5. Cropland. GISHydro lumps all cropland into a single land use category. Curve numbers for row crops (such as corn) are significantly different from those for small-grain (such as wheat). Inspection of satellite imagery may indicate the most common crop type. Satellite imagery may also indicate if cropland has been changed to residential, forest, or other land use. GISHydro also allows the selection of good/fair/poor hydrologic condition. This choice applies to all curve numbers selected in a GISHydro application.

4.3.4 Time of Concentration (overland/sheet flow component)

The application of several methods to calculate the overland component to the time of concentration can contain both random and systematic errors. This overland flow variable, by experience, has shown to be the most difficult to quantify of any of the input variables. The potential for a systematic error is high, which may be related to the experience or application techniques of the designer. This is one of the variables that should be examined for adjustment, especially if the sub-basins are small and the times of concentration are short.

4.3.5 Time of Concentration (shallow concentrated flow component)

Calculation of this portion of the T_c often generates a systematic error that results in underestimation of the flow time. The shallow concentrated flow portion of the time of concentration is generally derived using Figure 3-1 of the TR-55 manual (USDA, 1986) or similar graphs. Another approach for estimating flow velocities is Figure 15-4 of NEH Part 630, Chapter 15 Time of Concentration (USDA, 2010).

Use of Figure 15-4 in Chapter 15 Time of Concentration (USDA, 2010) may underestimate the travel time by overestimating the flow velocity for upper reaches of the shallow concentrated flow path. For shallow depth, the hydraulic radius approaches the depth of flow. In this shallow flow range the n value should represent a higher resistance than that which would be used for channel flow. Consider, for example, a wide grass swale with flow depths of less than 0.5 feet and grass 6-inches high or more. The Manning n value may fall between the 0.2 value for sheet flow and the 0.05 value for channel flow. In this case the designer might select a Manning n value of 0.10, which better represents this shallow concentrated flow. For specific shallow concentrated flow conditions, the designer can develop a new relationship of velocity to slope for more appropriate values of Manning n and the hydraulic radius.

4.3.6 Time of Concentration (channel flow component)

The selection of the channel component of the time of concentration can produce a systematic error that shortens the travel time. This can be attributed to three factors: incorrect estimates of the channel length, the Manning roughness coefficient and the bankfull cross-section.

Measuring the length of channel flow generally involves a scale error. Larger scale maps such as the USGS quad maps or digital terrain data at 1:24,000 do not account for all the bends or meanders of a natural stream channel. Using a smaller scale map (1 in = 200 ft) will help reduce this error, but it will always be systematic. **Adjustments in channel lengths up to 25% when measuring from a USGS 1:24,000 map or digital terrain data can be reasonable, provided the designer documents the decision.**

A single Manning n value selection to represent full cross-sectional flow should be higher than an n value used for just the channel in a hydraulics model like HEC-RAS. This single n value must account for all hydraulic losses including high resistance overbanks, expansion and contraction losses, gradient changes, debris in flow, and local obstructions such as culverts. An increase of up to 50% in the n value is appropriate when using a simple trapezoidal cross section and single n value as is most often done when calculating the channel flow portion of the travel time.

The NRCS recommends that the velocity defined by the bankfull cross section be used to estimate the channel component of the time of concentration. The channel velocity is a function of the two-thirds power of the hydraulic radius. Because the cross section and, therefore, the hydraulic radius change from point to point along the channel, it may be difficult to determine the "typical" bankfull section. Care must be taken in the definition of the "typical" section because an error can lead to a significant overestimate or underestimate of the time of concentration in a large watershed that has a relatively long channel component.

4.3.7 Representative Reach Cross Section for Reach Routing

The selection of a representative cross section for reach routing can produce large systematic errors. WinTR-20 models with many reaches may exhibit cumulative systematic errors that will significantly affect the peak flow estimation. Since the WinTR-20 model is sensitive to the timing of hydrographs routed through long reaches, the typical routing cross section is a likely choice for adjustment.

Systematic errors in the selection of a "representative cross section" often produce reach routing that underestimates the hydrograph travel and underestimates the attenuation. The n value selection and length of reach are again suspect as in the time of concentration channel flow component described earlier.

Generally, representative cross sections are derived from contour maps supplemented by estimates of the channel geometry from field reconnaissance. In most cases surveyed cross sections are not available. GISHydro uses the digital terrain data supplemented with empirical equations for the channel geometry.

The effect of stream storage is often underestimated. A good method to derive a representative cross section, if the data are available from prior FEMA studies, is to use the results of multiple HEC-RAS runs. For each flow rate, the cumulative volume in the reach is divided by the total reach length. This results in a representative cross-sectional area for each flowrate. However, cross sections for a hydraulic model such as HEC-RAS are usually taken so as to eliminate ineffective flow areas. These ineffective flow areas, while not contributing to the stream conveyance in the hydraulic model, do affect the attenuation of the hydrograph in the reach routing computation. This is most common in reaches that are characterized by wide, flat flood plains and wetlands. If stream storage is expected to be underestimated, the designer may be justified in increasing the area for each flow rate value on the WinTR-20 cross section table.

4.3.8 Reach Length

Reach lengths measured on large-scale maps (USGS Quad, 1:24,000 digital terrain data) commonly underestimate the true length of a stream. Topographic maps of a scale of (1:2,400) and smaller will show more meanders and yield longer measurements. The effective stream length may not be the same for minor and severe events (2-year vs. 100-year). This is due to the fact that the more extreme events are conveyed over floodplains rather than in the channel, resulting in shorter flow paths. For minor events, such as 5-year and less, a longer reach length is appropriate due to the longer flow path in the meandering channel.

WinTR-20 accepts input for channel length and flood plain length. The designer may choose to use this option when channel and flood plain lengths are significantly different.

Figure 4-2 shows the relationship of total time of concentration to drainage area for gaged watersheds in Maryland (data from Appendix 6). Three different regional curves are defined in Figure 4-2 for the Appalachian Plateau, Piedmont-Blue Ridge and Coastal Plain regions. These curves can be used as a guide for comparison to calculated T_c values.

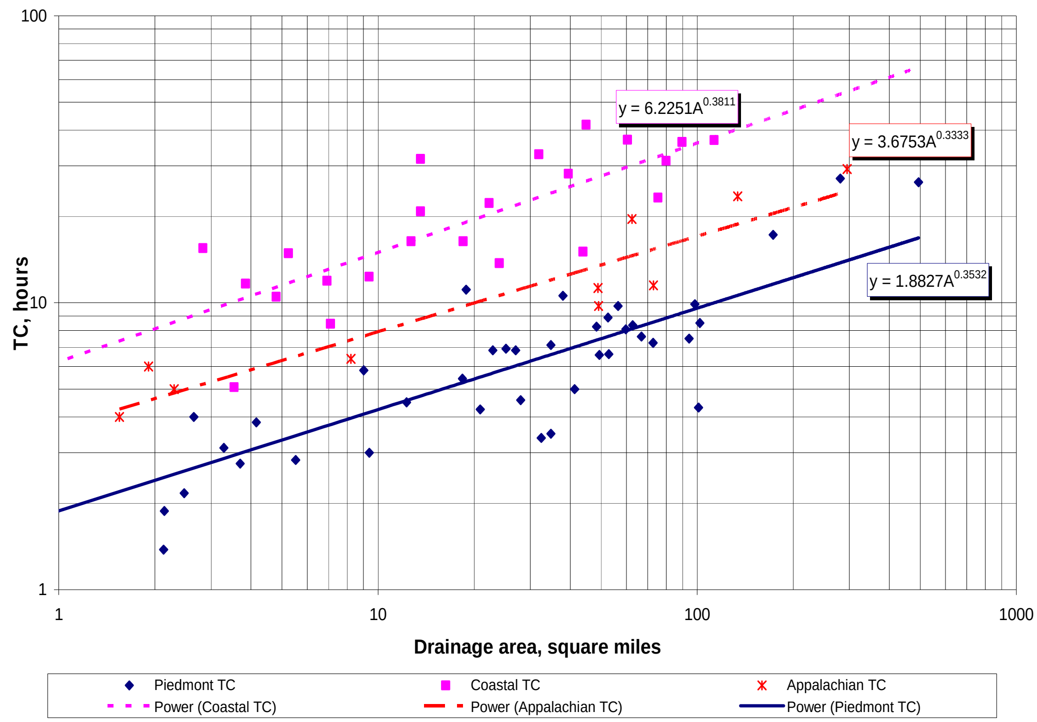

Figure 4-2: Time of concentration versus drainage area in Maryland

4.3.9 Storage at Culverts

Experience shows that if the storage behind a culvert is less than 10% of the volume of runoff of the contributing drainage area, storage routing may be ignored without significant impact in the peak flow rate prediction. However, an accumulation of several culverts, each having storage potential near 10%, could affect the peak flow prediction and should be examined.

The measurement of storage behind a culvert is sometimes subject to systematic error, which tends to underestimate storage, especially for low flows. Less detailed topographic maps or digital terrain data will not show small depressions and ditches that may contain storage that can affect the peak flow prediction of small storms.

4.3.10 Antecedent Runoff Condition (ARC)

Most applications will use the recommended value of ARC = 2 to represent the preliminary wetting of the ground surface and filling of small depressions. The ARC = 2, which represents the average watershed conditions when flooding occurs, is appropriate for severe storms such as the 10-, 25-, 50-, and 100-year events because they are generally related to the longer duration cyclonic events such as hurricanes and tropical storms with a longer duration**.** An ARC = 1, which is the dry soil condition, may be more applicable to short duration summer thunderstorms in dry weather for the more frequent 2- to 10-year rainfall events.

One calibration procedure that may be employed for the more frequent storms of 10-year frequency and less is the global change in RCN values for fractional ARC conditions. The WinTR-20 program accepts integer values of 1, 2 or 3 for ARC and also fractional ARC values between 1 and 3.

4.3.11 Dimensionless Unit Hydrograph

The dimensionless unit hydrograph varies by region. Refer to Table 3-1 for recommended peak rate factors. The peak rate factor determines the generalized shape of the runoff hydrograph. In a subdivided watershed, the subarea runoff hydrographs are routed downstream and added to other runoff branches at various intervals that influence the shape of the downstream hydrograph. Therefore, the influence of the unit hydrograph selection diminishes as the watershed is subdivided. Conversely, the total stream hydrograph shape for single area watersheds or those with a few large subareas are more influenced by the selection of the unit hydrograph.

4.3.12 Rainfall Tables

The 24-hour rainfall distribution used in the WinTR-20 model has been shown to approximate closely most of the Maryland statistical rainfall data for large cyclonic storms. However, there is justification for selecting storm durations of less than 24 hours in certain circumstances. Until new research on storm structure is complete, the 25-, 50-, and 100-year storm events should be derived using the 24-hour design storm duration. The 2-, 5-, and 10-year storm events may be derived using either the 6-hour or 12-hour design storm duration. For watersheds having a total time of concentration of less than six hours, the 6-hour design storm duration may be more appropriate. For watersheds having a total time of concentration between 6 and 18 hours, the 12-hour design storm duration may be more appropriate. Therefore, if the flood estimates using the 24-hour storm do not lie between the regression estimate and the upper prediction limit, the analyst should use the 12-hour storm for the 25-, 50-, and 100-year events and the 6-hour storm for the 2-, 5- and 10-year events provided that the T_c to the design point is not greater than 6 hours. See Table 1-1 for recommended storm durations.

Rainfall total depths for various frequency storms can be found in NOAA Atlas 14, Volume 2, dated 2006. This information is also available on the Web at: http://hdsc.nws.noaa.gov/hdsc/pfds/ .

4.3.13 Rainfall Depths

The uncertainty in estimating the rainfall depths in NOAA Atlas 14 is quantified by the upper and lower 90-percent confidence limits, which are reported along with the best estimate of the rainfall depth. This implies there is a 90-percent chance of the rainfall depth being between the lower and upper confidence limits. For Volume 2 of NOAA Atlas 14, the confidence limits are rather narrow with the upper 90-percent confidence limit being approximately 8 to 13 percent higher than the best estimate of rainfall depth depending on location, storm duration and return period.

NOAA Atlas 14, Volume 2, is based on rainfall data through 2000. Several large storms have occurred in Maryland since 2000 and there is the possibility that the Atlas 14 rainfall depths may be underestimated in certain parts of the state. The National Weather Service is working on an update of Volume 2 for Pennsylvania, Maryland, Delaware, Washington, DC, Virginia, North and South Carolina but it will a few years until the new study is available. In the meantime, the Panel recommends the use of the upper 90-percent confidence limit, in lieu of the best estimate, as an aid in getting the WinTR-20 discharges within the calibration window if all other adjustment procedures are not successful. The upper confidence limit data were incorporated into GISHydro and the program was revised to include the upper confidence limit as an option for the rainfall depths. The use of the upper 90-percent confidence limit could also account for any possible future change in climate. As discussed earlier, if it not possible to get the WinTR-20 discharges within the calibration window, the analyst should explain why and provide the rationale for the recommended discharges.

Table 4-2: Table of WinTR-20 Variable Adjustment Limits for Calibration

Variable	Error Type	Error Source Variable	Common Error Trend	Effect On Peak Q	Note	Adjustment Limits of variable in column 3
Area	Random	Area	High or Low	Increase or Decrease		Not Recommended, check for non-contributing areas
RCN	Random	Table Selection	High or Low	Increase or Decrease	4	± 10% for each category and within the limits of the NRCS guidelines.
T_c (Overland)	Systematic	n_o, L	Low	Increase	3	n_o up to 25%, L max = 100’
T_c (shallow conc.)	Systematic	Length, n	Low	Increase	3	Increase L up to 25%, n to ± 50%
T_c (channel)	Systematic	Length, n	Low	Increase	3	Increase L up to 25%, n to ± 50%
Representative X-section	Systematic	Area, n	Low	Increase	3	Area to ± 25%, n to ± 50%
Reach Routing Length	Systematic	Length	Low	Increase	3	Up to 25% for 1:24,000 maps, up to 19% for 1:2,400 maps
Storage at culverts	Systematic	Volume	Low	Increase	1	Up to 15%
ARC	Random	N/A	N/A	N/A	2	ARC= 2 is base value. See note below.
Dimensionless Unit Hydrogr.	Systematic	Peak Rate Factor	High or Low	Increase or Decrease		Regional values of PRF in Maryland
Rainfall Tables	Systematic	Increment, intensity, & duration	High or Low	Increase or Decrease		48, 24, 12 and 6 hr. distributions
Rainfall depths	Systematic	N/A	Low	Increase		Upper 90-percent confidence limit

Definitions:

Random (errors) = either high or low from an expected mean value.

Systematic (errors) = always higher or always lower than the calculated value.

Low = calculated value lower than probable “actual” value.

High = calculated value higher than probable “actual” value

Notes:

1. If the total volume of “reservoir” storage in the watershed is less than 10% of the total runoff volume, the effects of storage may be ignored.
2. ARC < 2 may be more appropriate for estimating the 10-year or more frequent storms. ARC > 2 may be appropriate for severe storms of 200 year and above.
3. Primary calibration variable.
4. Do not adjust the <u>weighted</u> RCN.

Table 4-2 is presented as a guide to assist the designer in reevaluating WinTR-20 input parameters that might be causing the peak discharges to fall outside the recommended fixed region regression equation bounds. The table is a guide suggesting that, because of the difficulties in the estimation process, the parameters of column 3 could be in error by as much as the value listed in the last column. The selected values of all parameters in column 3 must be supported by field and map investigations, be consistent with

4.4 SENSITIVITY OF WINTR-20 RESULTS TO VARIATION IN INPUT VARIABLES

Experience has shown that the variables that affect hydrograph timing contain the greatest potential error of estimation and are, therefore, those that should be calibrated first. The hydrograph timing variables include each of the time of concentration components, the representative reach cross section, and the reach length.

If further calibration is necessary, re-evaluate the watershed storage by adding storage routing at culverts and other structures that create backwater. In particular, railroad culverts and embankments frequently cause backwater and reservoir storage. In very flat areas only a small rise in backwater may generate substantial amounts of storage that should be included as reservoirs in the WinTR-20 model. Occasionally, urban watersheds may experience a cumulative effect of storage from multiple road culverts. It may be practical to combine a series of small culverts with backwater into one reservoir to simplify modeling if accurate flows between these culverts are not needed.

Calibration of RCN values involves selecting values within the range recommended by NRCS for each land cover and soil type. Generally, the designer will be changing the RCN value for woods, meadows, or croplands from average to good or poor condition to adjust the peak discharge**. However, these changes must be documented.** In limestone regions, there may be some justification for a further reduction in RCN values.

The designer must compare the appropriate Fixed Region Regression Equations with the peak flow rates computed by the WinTR-20 model. In some circumstances, a decision may be made to adjust the WinTR-20 model input variables to yield peak flows that are closer to the results of the regional equation. In most instances, the adjustment of the WinTR-20 input variables should fall within the ranges shown in Table 4-2. However, the following factors should be evaluated before adjusting the WinTR-20 input:

Does the WinTR-20, using map and field study defined input parameters that are within the bounds of sound hydrologic practice, estimate peak discharges that fall between the regression estimate plus one standard error of prediction? If it does, adjustment of the WinTR-20 may not be necessary.

1. Are the values of the input variables used for the Fixed Region Regression Equations within the limits prescribed? Do the study watershed conditions lie within the bounds of the data from which the regional regression was derived? If the answer to either of these equations is no, then the regional equation results may not be valid.

2. If part of the study watershed lies within different regions, has the proportional regional equation been computed using procedures that account for the drainage area in each region?

3. Have the Fixed Region Regression Equation input variables been measured from the data source as used in the derivation of the regional equations (i.e., 30-meter DEM data or MDP land use data? If not, the designer should determine if there is a possible bias by utilizing the same data source as used in developing the regression equations.

4. Are there reservoir storage, wetlands, quarries, or other features that may invalidate the regional equations? If these areas have been accounted for in the WinTR-20 model, there would be no benefit in a comparison to regional equation estimates.

5. Is the study area more than 10% impervious? If so, then the regional equations in the Eastern Coastal Plain and Appalachian Plateau Regions may not be valid.

If it is determined that the regional equation has been applied correctly and is valid for the study watershed, these results then may be used to adjust the input parameters of the WinTR-20 program. However, these WinTR-20 input parameter adjustments must be map- and/or field-justified and within the range of sound hydrologic practice. The designer will provide documentation that explains the selection and adjustment of each input parameter.

4.5 SPECIAL PROBLEMS WITH SMALL URBAN WATERSHEDS

Recent MDOT SHA experience has shown that the calibration of the WinTR-20 models to the Fixed Region Regression Equations for some small urban watersheds having drainage areas of less than two square miles may be problematic. In particular, small urban watersheds with predominant Type A or B soils may generate WinTR-20 peak discharges that are well below the target range calculated by the Fixed Region Regression Equations. In these cases, the Panel suspects that the standard RCN table values may not satisfactorily describe this urban condition and recommends one or more of the following additional calibration adjustments:

1. Use RCN values for urban land that are derived using “fair” or “poor” hydrologic conditions rather than “good”. (The urban RCN values in TR-55 were derived using proportions of impervious RCN = 98 and open space RCN based on soil type and “good” hydrologic condition.) See Table 4-3 below.

2. Subdivide generalized land use categories. Predominant land use in particular categories may result in a false hydrologic response. Refer to Section 4.3.3 for further discussion.

3. Some small urban watersheds may respond in more complicated ways than those accounted for in standard hydrologic applications. For instance, a watershed model that is highly urban may produce higher peak discharges when the shorter

"dominant" time of concentration from large impervious areas is applied rather than the longest T_c that is computed from non-impervious upland areas. Similarly, using the "paved" rather than the "non-paved" option for computation of the shallow concentrated flow segment of the T_c may be more appropriate where a significant proportion of non-stream channel flow is carried in pipes and street gutters.

Table 4-3: Urban Curve Numbers

Good conditions

Type	Impervious %	A soil	B Soil	C Soil	D Soil
1/8 acre	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	85
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acre	12	46	65	77	82
Commercial	85 *	89	92	94	95
Industrial	72 *	81	88	91	93

Fair conditions

Type	Impervious %	A soil	B Soil	C Soil	D Soil
1/8 acre	65	81	88	91	93
1/4 acre	38	68	80	86	89
1/3 acre	30	64	78	85	88
1/2 acre	25	61	76	84	88
1 acre	20	59	75	83	87
2 acre	12	55	72	81	86
Commercial	85 *	91	94	95	96
Industrial	72 *	84	90	93	94

Poor conditions

Type	Impervious %	A soil	B Soil	C Soil	D Soil
1/8 acre	65	88	91	94	95
1/4 acre	38	79	86	91	92
1/3 acre	30	77	85	90	92
1/2 acre	25	76	84	89	91
1 acre	20	74	83	88	91
2 acre	12	72	81	87	90
Commercial	85 *	94	95	96	97
Industrial	72 *	90	93	95	95

*Impervious values are based on buildings, parking lots, driveways, and related landscaped edges. Open space and woods are not included.

4.6 DERIVING ULTIMATE DEVELOPMENT PEAK FLOW RATES USING THE ADJUSTED WINTR-20 MODEL

In most cases, the designer will derive the "Ultimate Development" peak flow rates by only changing the RCN values in the calibrated existing land cover model. The new RCN values for each sub-basin are computed to reflect the future conditions using zoning maps or comprehensive planning maps. The other existing land cover model parameters usually remain unchanged. There may be instances where ultimate development channelization, enclosure, or restoration will result in velocities that are significantly different from those under existing conditions. In that situation the changed time of concentration would have to be incorporated. The focus on stream water quality, stormwater management, and wetland and habitat preservation in Maryland and the relatively few large river flood prone areas has inhibited the construction of major channel improvements, long large diameter pipe systems, and flood conveyance channel-levee systems. Of course, there may be exceptions to this assumption, which should be examined on a case-by-case basis. If justified, the hydrograph timing parameter can also be modified to reflect expected significant changes to stream channel hydraulic characteristics. Figure 4-3 below describes this procedure.

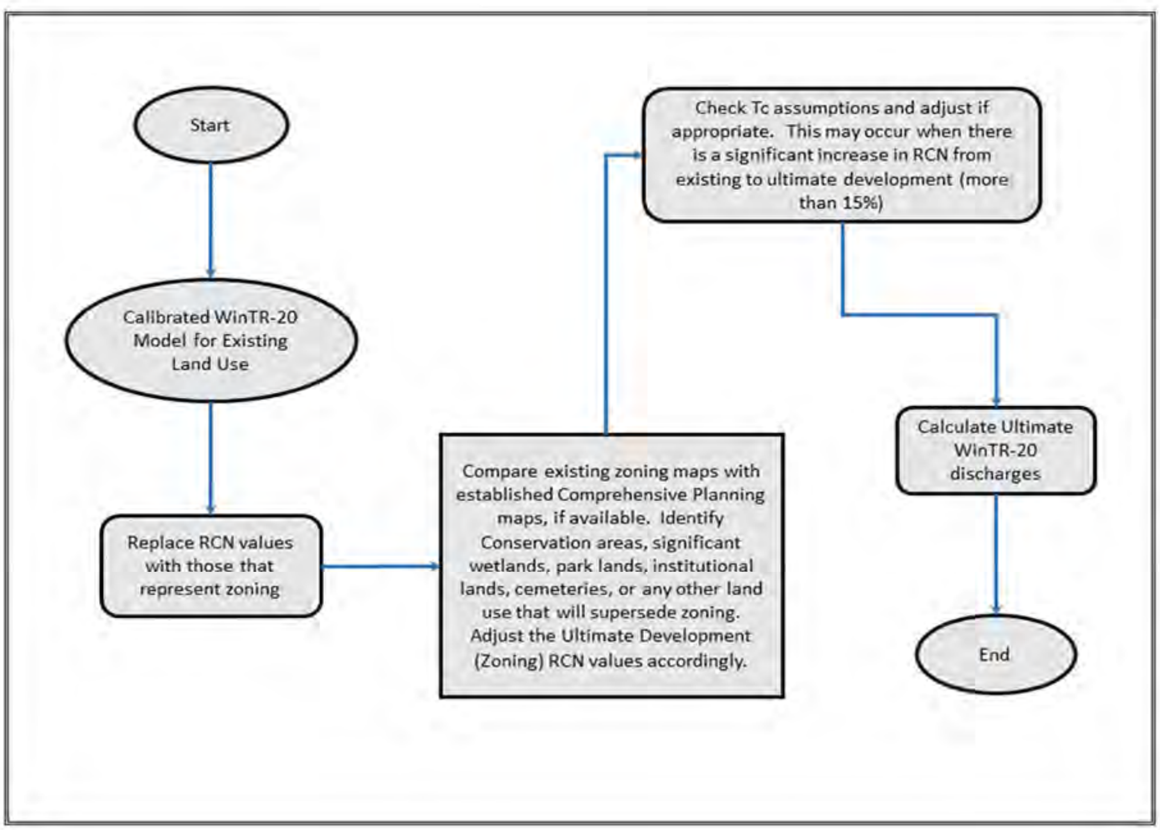

Figure 4-3: Flow chart for changing existing land use to ultimate development

4.6.1 Ultimate Development as Defined Under COMAR

The Code of Maryland Regulations (COMAR), Title 08, Subtitle 05, Chapter 03, "Construction on Non-Tidal Waters and Floodplains," states:

> *F. Unless waived by the Administration, hydrologic calculations shall be based on the ultimate development of the watershed assuming existing zoning.*

In the creation of a WinTR-20 hydrologic model for ultimate conditions, it is common practice for the designer to derive RCN values for each zoning type for the jurisdiction of the watershed. These "ultimate development" RCN values are substituted for the "existing" RCN values and an "ultimate development" model is constructed. This model, when the regulatory 2-, 10-, and 100-year rainfall is applied, results in "ultimate development" peak flow rates. These peak flow rates then are used for structure design or floodplain delineation and become the benchmark for regulatory evaluation. However, there are several pitfalls that both the practitioner and regulator should consider in its application. They are:

Many zoning districts cover a wide range of permitted uses that have significant variability in hydrologic characteristics. There are two methods of accounting for the wide variation: (1) use more subdivision of the zoning divisions into more homogeneous areas; (2) use weighted RCN for the zoning district based on the actual land use and hydrologic soil group.

1. Existing agricultural areas that are zoned for large multi-acre lots may yield lower RCN values under "ultimate development" than under the existing conditions of active croplands. Common practice has been to select the higher of the two RCN values. In some cases, this situation may be realistic if the hydrologic condition of the area was poor. However, this case is often unidentified or ignored in large, variable land use models.

2. Many modern zoning types do not lend themselves to simple conversion to an RCN value. Several of these zoning types are related to ecological and historic preservation or recreation that have a wide range of possible future RCN values.

Many jurisdictions permit clustered or planned unit development that typically creates high density mixed development interspersed with natural preservation areas. The resulting land cover then bears no resemblance to the originally described zone type; hence, the ultimate RCN value derived from it is unreliable.

3. The creation and editing of zoning maps is a political process and is not intended to represent future hydrologic conditions. A jurisdiction wishing to promote industrial development, for example, may designate large areas for that zoning classification to

attract industry, yet have no realistic expectation that all such zoned land will be developed. Similarly, rural jurisdictions may find it politically preferable to label vast areas as “agricultural” or “conservation” but expect to re-zone specific sites if a non-conforming, intensive use is deemed desirable. In all such cases the direct conversion from zoning type to RCN is invalid as a prediction of future peak flow rates.

4. Current environmental regulations inhibit full build out of many residential and other intensive use zoning districts. For example, a district that may permit 16 units per acre seldom achieves full density. This is due to restrictions such as wetlands, road systems, forest conservation, and recreational or open space reservations.

While these pitfalls are known to many in the hydrologic profession, the common rationalization of the use of zoning is that it is the best, or simplest, way to derive a future development model that will ensure that newly designed hydraulic structures are not under-designed. In other words, the regulation requiring the use of “ultimate development” peak flow rates for design is simply a hydrologic safety factor. Unfortunately, because of the unreliable nature of the future land use – zoning relationship, the use of existing zoning to derive “ultimate” peak flow rates will result in undefined and highly variable factors of safety for different watersheds. This is not a correct application of factors of safety in a hydrologic analysis.

The selection of a factor of safety to apply to a calculated peak flow rate should be based on the following considerations:

1. The potential for land use changes
2. The timing of land use change
3. The potential risk of failure of the hydraulic structure
4. The economic life and useful life of the hydraulic structure
5. The reliability of the computational method

Item number 5 is usually addressed in the selection of input values for each method and is discussed in other chapters of this report. Items 3 and 4 are often considered by selecting the flow or storm frequency. In general, large, expensive structures or ones that could endanger the public are designed for flows of lesser frequency such as the 100-year (1% annual chance of exceedance) for major interstate highways. Minor drainage systems are designed using the 10-year (10% annual chance of exceedance) event.

Item numbers 1 and 2, as discussed above, are not reliably estimated by zoning district. A better estimate of Items 1 and 2 can be derived from comprehensive planning maps. Comprehensive planning maps are prepared for most major jurisdictions in the state. Most plans include a 20-year projection and are available in both map and digital GIS form.

4.6.2 Using Comprehensive Planning Maps for Future Hydrologic Conditions

Comprehensive planning maps, if available, offer a better tool for the designer to predict the future land use of a watershed than the zoning map. They incorporate the key elements of time and spatial distribution that are not apparent in zoning maps. The designer can compare these maps to the zoning maps to determine the following:

1. Does the 20-year comprehensive plan approach complete build-out as defined by the zoning maps? If not, it may be better to use the comprehensive plan as the more realistic future projection.

2. Does the comprehensive plan define specific land use within a general zoning type? Comprehensive plans will show areas of likely growth based on existing and planned transportation networks, proximity to growth centers, and water and sewer service areas. They will also account for special environmental or historic areas and buffers, critical areas, unfavorable terrain, proximity to uninviting land use such as landfills and airports, and similar conditions that are likely to inhibit growth.

3. Will the intensively urbanized areas induce in-fill type development according to zoning or will the general character of the urban area change? Comprehensive plans may account for the trends for more urban green space or the conversion from heavy industrial to office parks, recreation/tourism, or mixed residential/commercial use.

The current regulation permits the Administration (now Maryland Department of the Environment) to waive the requirement of current zoning to define ultimate development. This requirement should be waived in favor of the Comprehensive Planning Maps, wherever appropriate.

4.6.3 Estimating Ultimate Development for Large Watersheds

Ultimate development in large watersheds over 300 square miles is often similar to existing conditions because it is not likely that urbanization and development can cover a significant portion of the upstream watershed over the design life of the structure. Guidance is provided on how to determine ultimate development in large watersheds over 300 square miles.

The objectives of this guidance include:

1. Use a relation between impervious area and population to illustrate that ultimate development will be similar to existing development for most watersheds greater than 300 square miles.
2. Identify all watersheds in Maryland that have a total drainage exceeding 300 square miles and use the impervious area-population density relation to determine

those watersheds where ultimate development may differ significantly from existing development.

Impervious area is used as a measure of development in the watershed. Figure 4-4 illustrates data for 96 gaged watersheds in the Piedmont−Blue Ridge Region; it indicates that impervious decreases quickly as the size of the watershed increases. This outcome is logical because urban development is concentrated in residential and commercial areas and as the watershed size increases it is less likely that urbanization will cover a significant portion of the watershed.

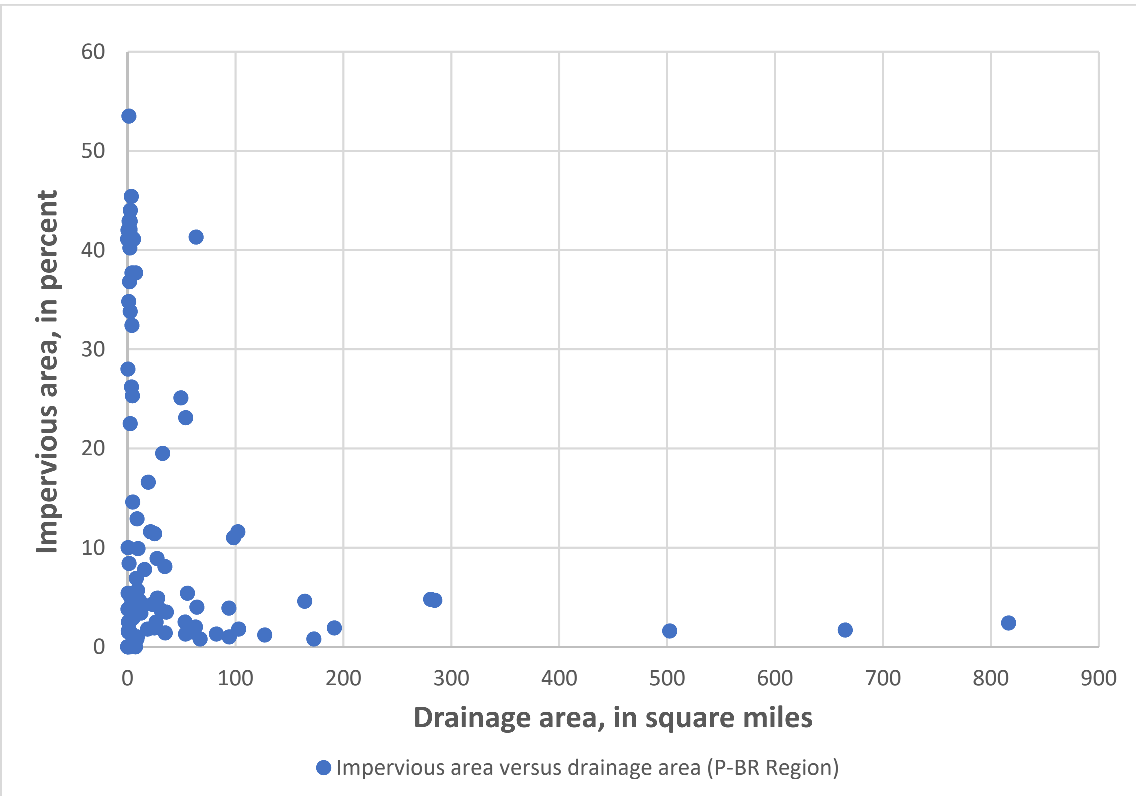

Figure 4-4: Relation of impervious area and drainage area for 96 gaged watersheds in the Piedmont−Blue Ridge Region of Maryland

As defined in the Code of Maryland Regulations (COMAR), "Unless waived by the Administration, hydrology calculations shall be based on ultimate development of the watershed assuming existing zoning." A reasonable assumption is that the time frame of the zoning maps or comprehensive planning maps is consistent with the life span of most hydraulic structure (e.g., 75-100 years). In this context, ultimate development would be land use conditions around 2100.

Impervious area is obviously highly related to population density. Impervious cover or area is a result of human settlement, and thus, population density should be a reasonable predictor of impervious area arising from residential development and the commercial areas that directly support them. Several studies have used population density to estimate impervious area; some are illustrated in Figure 4-5 (Exum and others, 2005).

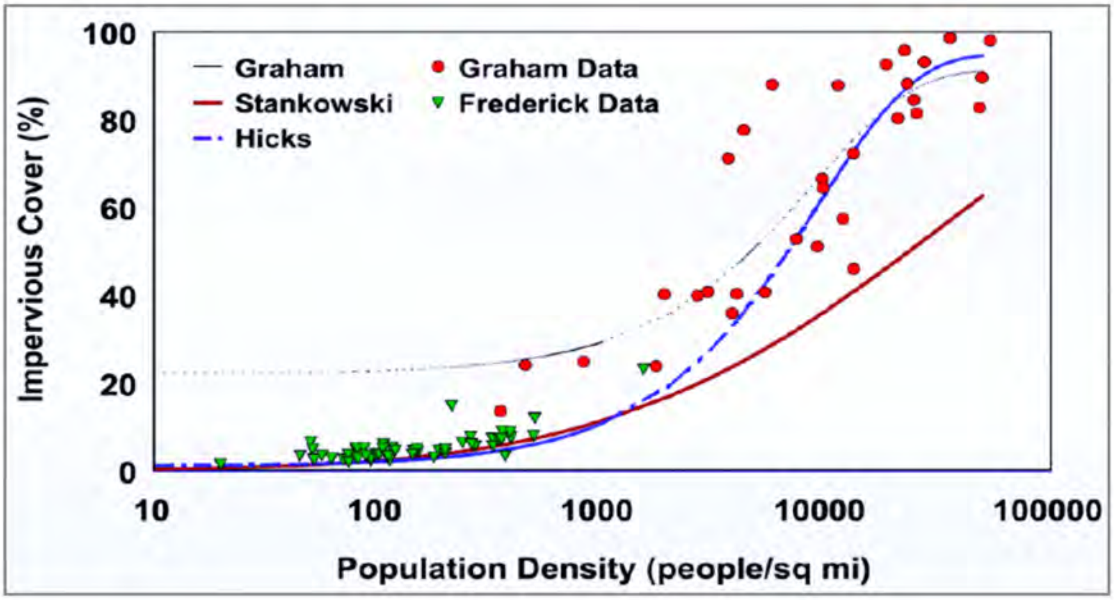

Figure 4-5: Impervious cover versus population density for several studies (from Exum and others, 2005)

The data and relations in Figure 4-5 are briefly defined as:

- Hicks – based on land use data in Vancouver, British Columbia, Canada
- Graham – based on land use data in Washington, DC
- Stankowski – based on land use data in New Jersey
- Frederick data – data in Frederick County, Maryland estimated from USGS Orthophoto Quadrangles dated 1989.

Additional details are provided in Exum and others (2005). The relations developed by Hicks, Graham, and Stankowski were based on land use zoning categories similar to procedures in GISHydro for estimating impervious area. The most reasonable relation in Figure 4-5 is the Hicks relation (heavy dotted blue line) based on land use data in Vancouver, British Columbia. The Hicks relation also agrees reasonably well with the data compiled by Graham for the Washington, DC area. The equation for the Hicks relation is:

$$IA = 95 - 94 \exp(-0.0001094\ PD) \qquad (4.1)$$

where IA is impervious area in percent and PD is population density in people per square mile. The relation between IA and PD from Equation 4-1 is quantified in Table 4-4.

Table 4-4: Relation between impervious area (IA) and population density (PD) from the Hicks relation (Equation 4.1)

Population density in people per square mile	Impervious area in percent
100	2.0
500	6.0
630	7.3
1,000	10.7
5,000	40.6
6,760	50.1
10,000	63.5

Using Equation 4.1 and estimates of existing and future population density, a determination was made as to which large watersheds in Maryland would have or would not have ultimate development significantly different from existing development.

Excluding the Potomac and Susquehanna Rivers, the watersheds in Maryland with total drainage areas greater than 300 square miles include:

1. Choptank River, 1,004 square miles with 22 percent of the watershed in water (large tidal area),
2. Chester River, 368 square miles with 20 percent of the watershed in water (large tidal area),
3. Nanticoke River, 828 square miles with a large tidal area,
4. Patapsco River, 680 square miles with 7 percent of the watershed in water,
5. Patuxent River, 937 square miles, largest and longest river entirely within Maryland,
6. Monocacy River, 960 square miles, largest Maryland tributary to the Potomac River,
7. Conococheague Creek, 566 square miles where only 65 square miles are in Maryland, and
8. Youghiogheny River, 294.1 square miles at gaging station at Friendsville, MD. No bridges in Maryland over the river downstream of Friendsville before reaching the Maryland-Pennsylvania state line where the drainage area exceeds 300 square miles.

There are eight watersheds in or partially in Maryland where the total drainage area exceeds 300 square miles. There are only five gaged watersheds in Maryland where the drainage area exceeds 300 square miles and three of them are in the Monocacy River watershed. The lack of gaging stations on large streams in Maryland is related to the fact that the lower reaches of all the large streams in the coastal plains are tidally affected and subject to backwater conditions or reverse flow.

The eight large watersheds in Maryland fall into two categories:

1. Rural watersheds where population will not increase enough to significantly increase impervious area by 2100: Choptank River, Chester River, Nanticoke River, Monocacy River, Conococheague Creek and Youghiogheny River, and
2. Urban watersheds where population is increasing more rapidly and where ultimate development will differ from existing conditions: Patuxent River and Patapsco River.

For the watersheds in category 1, the ultimate development will not differ from existing conditions. For these watersheds, ultimate development is assumed to be the same as existing conditions. However, the Patuxent River and Patapsco River watersheds in category 2 have a relatively high level of urbanization in 2022 and are undergoing rather rapid urbanization. For this reason, the ultimate development will differ from existing land use conditions. Future projects in these watersheds should consider ultimate development. Both watersheds are entirely within Maryland and ultimate development data are available.

4.7 CALIBRATING INDIVIDUAL SUB-AREAS IN LARGE WATERSHEDS

Generally, the calibration of a WinTR-20 model is accomplished at the design point (outlet) of the watershed. However, there may be logical reasons to calibrate large subareas of a watershed individually to regression equations derived from the data for those subareas. Some of these circumstances could be:

1. A watershed with subareas that have significant differences in the regression predictor variables. A large subarea may be highly urbanized or contain a large percentage of carbonate bedrock or dense forest compared to the other subareas. In such cases, the assumption of homogeneous properties may not adequately model the subarea timing.

2. The watershed's stream network does not have a typical branching shape, i.e. it may have large subareas converging near the outlet making subarea hydrograph timing critical to the combined peak flow development.

3. There is a stream gage in a major subarea that can be used to better model that segment of the overall model.

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CHAPTER FIVE

5 Regression Equations for Estimating Low Flows and Flow Duration Percentiles for Fish Passage in Maryland

5.1 EXECUTIVE SUMMARY

The Maryland Department of Transportation State Highway Administration (MDOT SHA) requires low flow values for designing culverts to accommodate fish passage. Regression equations were developed for estimating the 2- and 10-year 90- and 120-consecutive day annual low flows for streams in Maryland with drainage areas less than 10 square miles (Thomas and others, 2014). The low flow frequency analyses at the 50 gaging stations were performed using the U.S. Army Corps of Engineers (USACE) HEC-SSP program (USACE, 2010) and were based on the annual minimum 90- and 120-consecutive day low flows. The statistically significant explanatory variables in the regression equations were drainage area, in square miles; impervious area in percent of watershed area; and land slope in feet per foot. The regression equations are applicable for rural and urban streams. The standard errors of the regression equations ranged from 45.1 to 53.3 percent. The regression equations are given below:

2-year 90-day low flow (Q_{2_90}):

$$Q_{2\_90} = 0.635\,DA^{0.979}\left(IA+1\right)^{0.160}\,LANDSL^{0.242} \qquad SE = 52.2 \text{ percent}$$

10-year 90-day low flow (Q_{10_90}):

$$Q_{10\_90} = 0.420\,DA^{0.816}\left(IA+1\right)^{0.177}\,LANDSL^{0.232} \qquad SE = 53.3 \text{ percent}$$

2-year 120-day low flow (Q_{2_120}):

$$Q_{2\_120} = 0.670\,DA^{1.019}\left(IA+1\right)^{0.147}\,LANDSL^{0.208} \qquad SE = 45.1 \text{ percent}$$

10-year 120-day low flow (Q_{10_120}):

$$Q_{10\_120} = 0.463\,DA^{0.851}\left(IA+1\right)^{0.193}\,LANDSL^{0.236} \qquad SE = 50.2 \text{ percent}$$

where Q_{T_D} is discharge [cfs] for return period T [yr] and duration D [days]
DA is drainage area [sq mi]
IA is impervious area [percent of watershed area]
LANDSL is average watershed land slope [ft/ft]
SE is standard error [percent]

The watershed characteristics used in defining these regression equations are more indicative of flood flows and improvements in the regression equations could be realized through further research on:

- Development of geologic or groundwater characteristics that should be highly correlated with low flows, and
- Investigation into seasonal flow characteristics that might be more indicative of fish spawning and migration in Maryland streams.

A limited analysis of seasonal streamflow characteristics was performed at 16 gaging stations scattered throughout the State. The regression equations from this analysis are provided for informational purposes.

Regression equations based on drainage area were also developed for estimating flow duration percentiles using data for 57 gaging stations less than 50 square miles across all hydrologic regions of Maryland. The flow duration percentiles analyzed were those where the daily flow was exceeded 10-, 50- and 90-percent of the time over the period of record. The regression equations and data used in this analysis are described in Section 5.10.

5.2 INTRODUCTION

Estimates of discharges are needed in the design of culverts in Maryland to facilitate fish passage. This study was undertaken to develop regression equations for estimating design flows for small watersheds in Maryland for which culverts are used as the hydraulic structure of choice. The original intent of the analysis was to develop regression equations for estimating various recurrence intervals of the annual or seasonal 7-day low flow (lowest 7-day consecutive daily flow on an annual or seasonal basis). An analysis of the U.S. Geological Survey (USGS) gaging station data in Maryland revealed about 90 gaging stations with 10 or more years of daily flow data where the drainage area was less than 50 square miles. An analysis of annual 7-day low flows for 16 stations with drainage areas less than 50 square miles indicated that the 2- and 10-year 7-day low flows were often zero or close to zero. Therefore, it was decided to analyze low flows with durations of 14 to 120 days to obtain larger flows that might be useful in designing culverts for fish passage.

MDOT SHA indicated that culverts are primarily used on watersheds with drainage areas less than 10 square miles. There are 50 gaging stations in Maryland with more than 10 years of daily flow record where the drainage area is less than 10 square miles. The 50 stations used in the analysis, and their locations are shown in Figure 5-1. As shown in Figure 5-1, most of these stations are in the Piedmont Region in the vicinity of the City of Baltimore, Baltimore County, or adjacent counties.

Guidance from the Federal Highway Administration on hydrology for fish passage is documented in Hydraulic Engineering Circular (HEC) No. 26, First Edition, *Culvert Design for Aquatic Organism Passage*, dated October 2010 (Kilgore and others, 2010). As defined in HEC-26, there are two flows of interest:

- High passage flow, Q_H, represents the upper bound of discharge at which fish are believed to be moving within the stream, and
- Low passage flow, Q_L, is the lowest discharge for which fish passage is possible, generally based on minimum flow depths required for fish passage.

The emphasis in this study was to estimate a low flow discharge that is similar to Q_L as defined in HEC 26. Low flow analyses for the annual minimum discharge for durations of 14, 30, 60, 90, and 120 consecutive days were performed for the 50 stations in Figure 5-1. The stations used in the analysis are listed in Attachment 5-1 in Section 5.8.

In the fall of 2013, MDOT SHA convened a conference call with fish biologists with the Maryland Department of Natural Resources and the University of Maryland to gain insight into what might be reasonable design flows to analyze. The conclusions from that call were:

- The two primary spawning periods for fish species in Maryland are March to June and September to November, and
- Fish passage is critical year round, and annual flow characteristics are needed.

Based on the conference call, it was not clear whether seasonal or annual flow characteristics were most important. Therefore, the decision was made to analyze annual minimum (not seasonal) n-day discharges at the 50 stations shown in Figure 5-1.

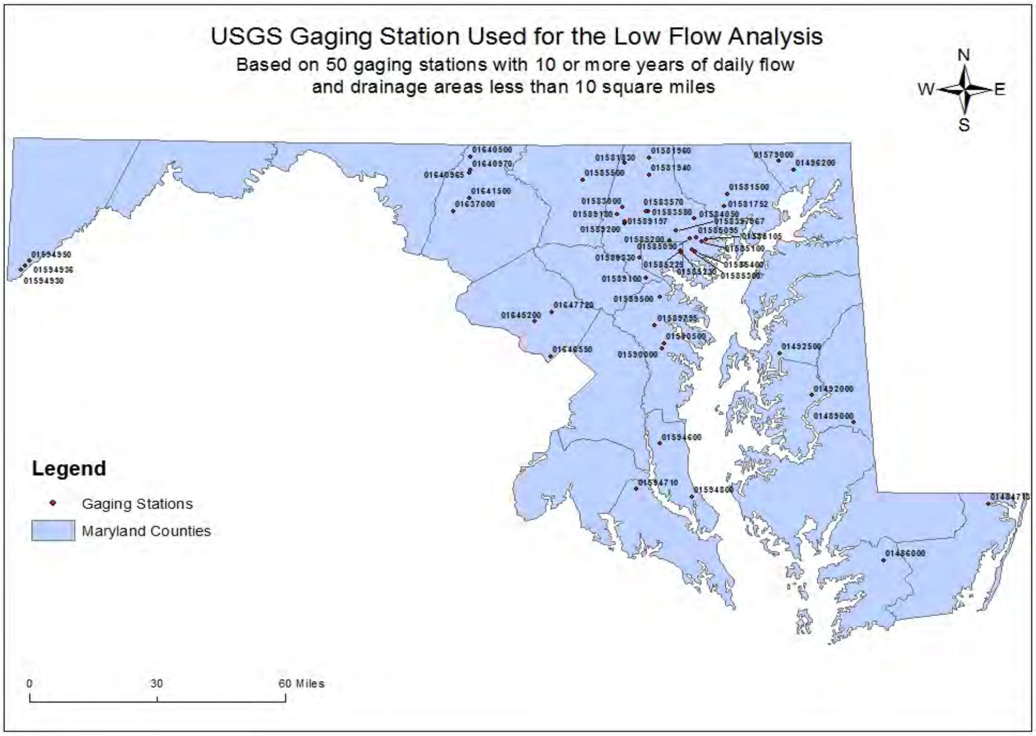

Figure 5-1: Location of 50 gaging stations used in the analysis where there are 10 or more years of daily flows and the drainage area is less than 10 square miles

5.3 LOW FLOW FREQUENCY ANALYSIS

Mean daily flows for the 50 stations in Figure 5-1 and Attachment 5-1 were retrieved from the USGS National Water Information System (NWIS) from the following web site (http://nwis.waterdata.usgs.gov/md/nwis/dv/?referred_module=sw). The U.S. Army Corps of Engineers (USACE) HEC-SSP Program (Version 2.0, dated October 2010) was used to estimate the annual minimum discharges for 14-, 30-, 60-, 90- and 120-consecutive day durations. The climatic year of April 1 to March 31 was used in this analysis to insure that the low flow period in the late summer and early fall (namely July to October) were within the same year. A Pearson Type III frequency distribution was fit to the logarithms of the annual minimums and the untransformed data (discharges in cubic feet per second [cfs]) using the HEC-SSP program. The Pearson Type III distribution was chosen because it is a 3-parameter distribution that is flexible in fitting low flow data and has been used historically by USGS for this type of analysis. Estimates of the 2- and 10-year discharge were summarized for durations of 14-, 30-, 60-, 90-, and 120-days for the 50 gaging stations.

An examination of the data revealed that the 2- and 10-year flows for durations of 14-, 30-, and 60-days were frequently zero or close to zero so the remaining analysis focused on the longer 90- and 120-day duration flows. In addition, the untransformed analysis sometimes resulted in the 10-year 90- or 120-day flow being negative so the analysis was limited to using the logarithms of the 90- and 120-day flows. The following four flow characteristics, based on the logarithms of the discharges, were used in the regression analysis:

- 2-year 90-day discharge,
- 10-year 90-day discharge,
- 2-year 120-day discharge, and
- 10-year 120-day discharge.

Generally, there was not a significant difference in the 2- and 10-year discharges based on the logarithmic transformed analysis and the untransformed analysis. A comparison is given in Figure 5-2 for the 2-year 120-day discharges based on the logarithmic transformation (logs) and the untransformed (cfs values) analyses. As shown in Figure 5-2, there is a slight tendency for the 2-year 120-day discharge to be higher when using the untransformed data. The trend line through the data in Figure 5-2 is nearly the equal discharge line (constant of 1 and exponent of 1). The results from the transformed analysis were used in the regression analysis because occasionally the untransformed analysis resulted in a negative flow for the 10-year flow as noted earlier. This was an artifact of fitting the Pearson Type III distribution to the untransformed low flow data. For the 2-year 120-day analysis, there were no negative flows for the untransformed analysis, so the data in Figure 5-2 are for all 50 stations.

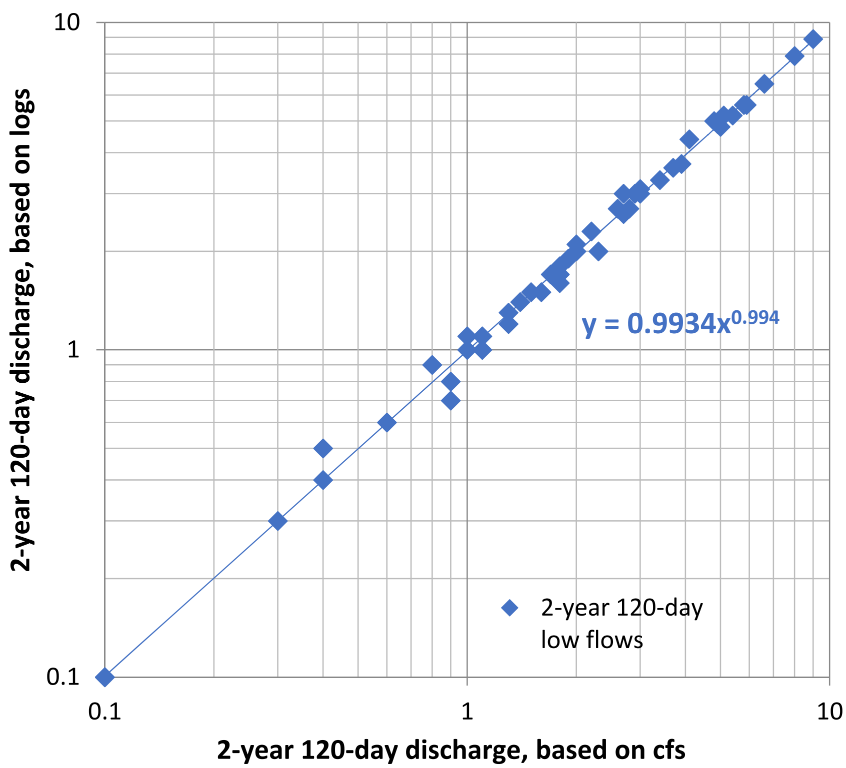

Figure 5-2: Comparison of the 2-year 120-day discharges based on the logarithmic transformed (logs) and the untransformed analysis (cfs values)

5.4 REGRESSION ANALYSIS FOR ANNUAL MINIMUM N-DAY LOW FLOWS

The watershed characteristics used in the regression analysis were obtained from the ongoing and previous flood discharge regression analyses for the State of Maryland. These watershed characteristics included:

- Drainage area, in square miles;
- Impervious area, in percent of the drainage area;
- Land slope, in feet per foot, slope of the watershed, not the main channel (The average land slope is the average of all neighborhood slopes determined along the steepest direction of flow. These are the local slopes determined from the upstream to downstream pixel for each pixel within the watershed.);
- Channel slope, in feet per mile, calculated as the slope between two points located at 10 and 85 percent of the distance along the main channel;
- Forest cover, in percent of the drainage area;
- Hydrologic soil groups A, B, C and D, in percent of the drainage area, based on SSURGO data;
- Basin relief, in feet, calculated as the average elevation of all points within the watershed minus the elevation at the outlet of the watershed; and
- Channel length, in miles, calculated as the distance along the main channel from the outlet of the watershed to the basin divide.

These watershed characteristics are relevant to flood runoff and are not necessarily the best suite of characteristics for estimating low flows. For example, no geological characteristics were determined as part of this analysis because it was beyond the scope of work. A future research effort should involve the determination of characteristics more related to geology and ground water contributions. None of the 50 small gaging stations are impacted by karst terrain, so the percentage of limestone in the watershed was not used as an explanatory variable.

Many of the watershed characteristics used in the regression analysis are correlated and the objective of any regression analysis is to use explanatory variables that are reasonably independent. For this regression analysis, all variables were converted to logarithms and a linear regression analysis was performed using the Statistical Analysis System (SAS) software (SAS Institute, Cary, North Carolina). Table 5-1is the correlation matrix for the logarithms of the watershed characteristics for the 50 gaging stations (N=50) and the following observations are pertinent (highlighted in Table 5-1):

- Drainage area (lda) and channel slope (lcsl) have a correlation of -0.428,
- Impervious area (lia) and forest cover (lfor) have a correlation of -0.725,
- Land slope (llandsl) and channel slope (lcsl) have a correlation of 0.808, and
- The sum of A and B soils (labsoil) and the sum of C and D soils (lcdsoil) have a correlation of -0.714.

Basin relief and channel length were not shown in Table 5-1 because these data are not available for all 50 stations. However, these variables are highly correlated with other variables shown in Table 5-1 as follows:

- Basin relief and land slope have a correlation of 0.884,
- Basin relief and channel slope have a correlation of 0.928,
- Channel length and drainage area have a correlation of 0.899.

Basin relief and channel length were obtained from the September 2010 Maryland Hydrology Panel report (http://www.gishydro.eng.umd.edu/panel.htm) and evaluated for possible use in the regression analysis but were not available for all 50 stations in the current analysis. Because they are highly correlated with other variables used in the analysis, no attempt was made to estimate these data for all stations, and basin relief and channel length were not used in the regression analysis.

Table 5-1: Correlation matrix for the watershed characteristics for the 50 gaging stations used in the regression analysis

Pearson Correlation Coefficients, N = 50
Prob > | r | under H_0: $\rho = 0$

	lda	lia	llandsl	lcsl	lfor	labsoil	lcdsoil
lda	1.00000	-0.22784 0.1115	-0.07963 0.5825	-0.42811 0.0019	0.23694 0.0976	-0.00722 0.9603	0.07624 0.5987
lia	-0.22784 0.1115	1.00000	-0.10369 0.4736	-0.00095 0.9948	-0.72478 < 0.0001	0.26816 0.0597	-0.10061 0.4869
llandsl	-0.07963 0.5825	-0.10369 0.4736	1.00000	0.80765 < 0.0001	0.23026 0.1077	-0.13750 0.3410	-0.16714 0.2460
lcsl	-0.42811 0.0019	-0.00095 0.9948	0.80765 < 0.0001	1.00000	0.06987 0.6297	-0.22202 0.1212	-0.00564 0.9690
lfor	0.23694 0.0976	-0.72478 < 0.0001	0.23026 0.1077	0.06987 0.6297	1.00000	-0.25901 0.0693	0.06355 0.6611
labsoil	-0.00722 0.9603	0.26816 0.0597	-0.13750 0.3410	-0.22202 0.1212	-0.25901 0.0693	1.00000	-0.71376 < 0.0001
lcdsoil	0.07624 0.5987	-0.10061 0.4869	-0.16714 0.2460	-0.00564 0.9690	0.06355 0.6611	-0.71376 < 0.0001	1.00000

Highlighting: sample correlation contradicts the null hypothesis of zero correlation.

If two variables are highly correlated, then they are explaining the same variability in the dependent variable (discharge) and, likely, one of the explanatory variables will not be statistically significant in the regression analysis. For the explanatory variables shown in Table 5-1, the four most statistically significant variables for estimating the 2- and 10-year 90-day discharge and the 2- and 10-year 120-day discharge were drainage area, impervious area, land slope, and channel slope, with channel slope being the least significant. For the 2-year 90-day and 120-day discharge analyses, the inclusion of channel slope reduced the standard error by 2.8 and 1.4 percent, respectively. Channel slope was not used in the final regression equations because of the small reduction in standard error and the significant correlation with both drainage area and land slope.

The 90- and 120-day discharges were chosen for the regression analysis because the discharges are larger and generally greater than zero. Very small values of discharges are not useful in designing a culvert. However, there were still two gaging stations (01583570 and 01589180) where the 10-year 90- and 120-day low flows were zero (see Attachment 5-1 in Section 5.8). Because all data were transformed to logarithms for the linear regression analysis, a small constant of 0.1 cfs was added to all the 10-year discharges to avoid taking the logarithm of zero. This implies that a constant of 0.1 cfs should be subtracted for the regression estimate for the 10-year equations. For all 50 gaging stations, the 2-year 90- and 120-day lows were greater than zero, so no constant was added in the 2-year analysis.

The regression equations for estimating the 2- and 10-year 90- and 120-day discharges are based on drainage area (DA), in square miles; impervious area (IA), in percent of the drainage area; and land slope (LANDSL), in feet per foot. A constant of 1 was added to impervious area to avoid taking the logarithm of zero. The equations are as follows:

2-year 90-day low flow (Q_{2_90}):

$$Q_{2\_90} = 0.635\ DA^{0.979}\ (IA+1)^{0.160}\ LANDSL^{0.242} \qquad \text{Std. error = 52.2 percent} \qquad (5.1)$$

10-year 90-day low flow (Q_{10_90}):

$$Q_{10\_90} = 0.420\ DA^{0.816}\ (IA+1)^{0.177}\ LANDSL^{0.232} \qquad \text{Std. error = 53.3 percent} \qquad (5.2)$$

2-year 120-day low flow (Q_{2_120}):

$$Q_{2\_120} = 0.670\ DA^{1.019}\ (IA+1)^{0.147}\ LANDSL^{0.208} \qquad \text{Std. error = 45.1 percent} \qquad (5.3)$$

10-year 120-day low flow (Q_{10_120}):

$$Q_{10\_120} = 0.463\ DA^{0.851}\ (IA+1)^{0.193}\ LANDSL^{0.236} \qquad \text{Std. error = 50.2 percent} \qquad (5.4)$$

The exponent on drainage area (DA) is close to 1.0 and higher than the exponents in the flood discharge regression equations. An exponent close to 1.0 implies the entire watershed is contributing discharge namely because the discharge is coming from ground water flow and not direct runoff. The exponent on impervious area (IA) is positive, implying that the more highly impervious watersheds are yielding more discharge than similar sized rural watersheds. This is likely related to the production of water in the more urban watersheds through lawn watering, residential and commercial water use, and businesses like car washes or car dealerships. The exponent on impervious area is larger for the 10-year equation than the 2-year equation, possibly implying that regulation by water use has a larger impact on the smaller discharges. Land slope has a positive exponent and may reflect the ground water gradient.

There were some outlier stations for all four equations but the outlier stations varied among the equations. There were both high and low outliers so the resultant equations do not appear to be biased. It was not obvious why certain stations were outliers so all 50 station were used in defining the regression equations.

As noted earlier, 34 of the 50 stations used in the analysis are in close proximity to the City of Baltimore, Baltimore County, or adjacent counties in the Piedmont Region. Because many of the stations are in urban areas, 21 of the 50 stations used in the regression analysis have impervious areas greater than 10 percent. There were only 16 stations in the Eastern and Western Coastal Plains and the Appalachian Plateau, so it was not possible to develop separate equations for the different hydrologic regions. Equations 5.1 to 5.4 are applicable statewide within the following limits:

- Drainage areas from 0.13 to 10 square miles,
- Impervious area from 0 to 45.4 percent, and
- Land slope from 0.0035 to 0.155 feet per foot.

Figure 5-3 illustrates the relation between the gaging station estimates of the 2-year 120-day discharge and drainage area for all 50 stations. Figure 5-3 illustrates that, on average, the 2-year 120-day discharge is 0.1 cfs at about 0.2 square miles, 0.54 cfs at 1.0 square mile and about 5 cfs for 10 square miles. Figure 5-3 further illustrates that there is a reasonable linear relation between the logarithms of the 2-year 120-day discharge and drainage area.

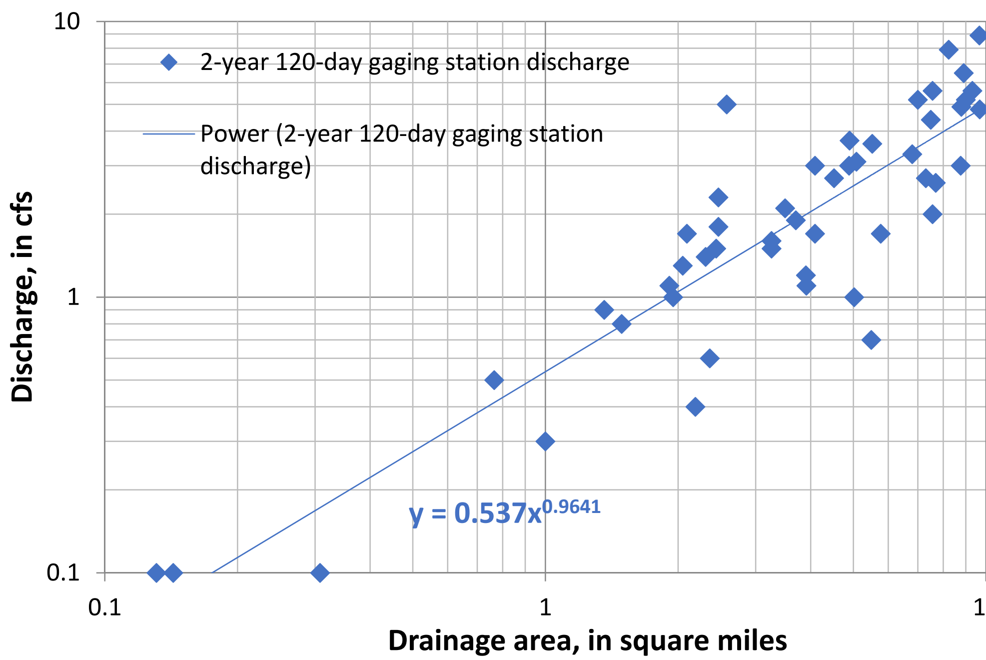

Figure 5-3: Relation between the gaging station 2-year 120-day discharge and drainage area for all 50 stations

Figure 5-4 compares the estimates of the 2-year 120-day discharge from Equation 5.3 (regression estimate) with gaging station estimates given in Attachment 5-1 in Section 5.8. The comparison is made for the 2-year 120-day discharge because these discharge values are the largest and potentially most useful for designing culverts for fish passage.

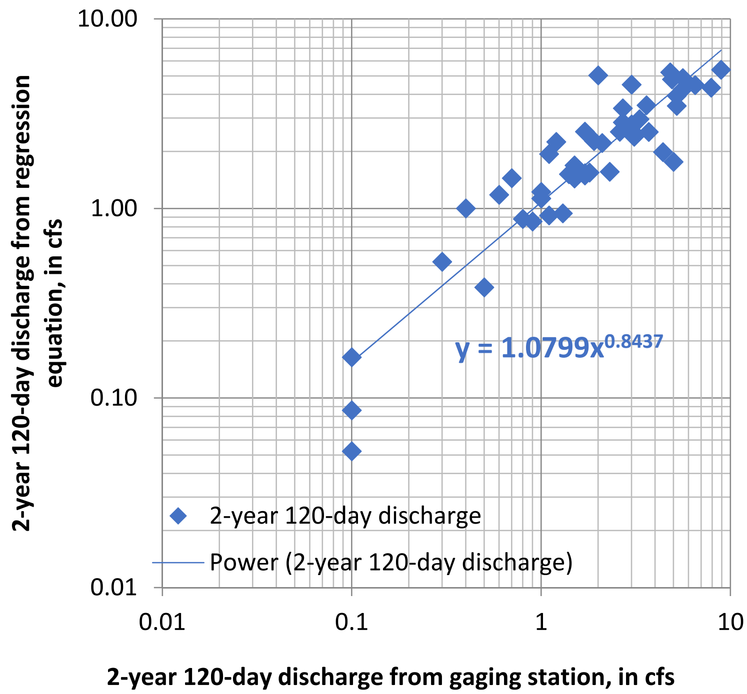

Figure 5-4: Comparison of the 2-year 120-day discharge from Equation 5.3 to the gaging station estimates

There are three points in Figure 5-4 where the gaging station estimate of the 2-year 120-day discharge is 0.1 cfs. The discharges are reported to the nearest 0.1 cfs in the USACE HEC-SSP program so the gaging station estimates are reported to that accuracy. The lower three points in Figure 5-4 give the impression there may be a non-linear relation between the regression and gaging station estimates. However, if the three gaging station estimates of 0.1 cfs are omitted from the figure, the remaining points define a reasonable linear trend (see Figure 5-5).

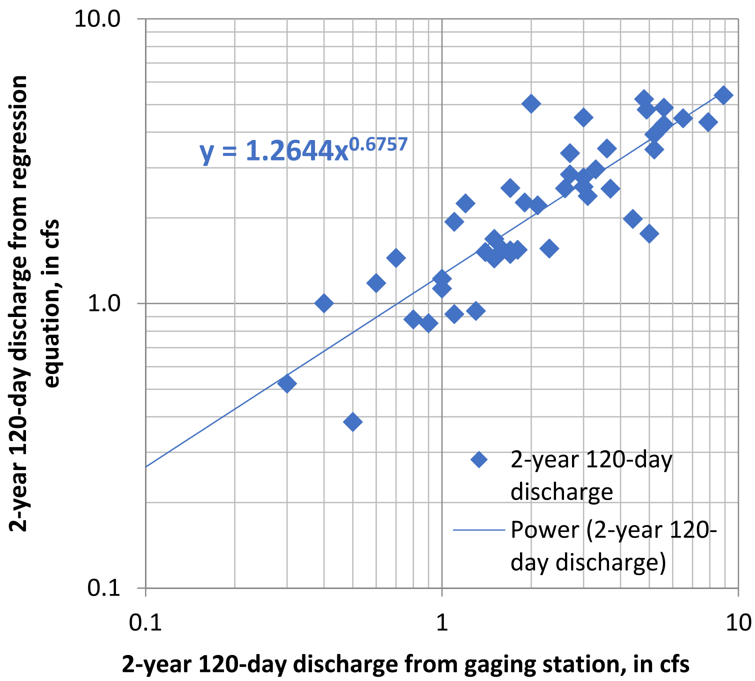

Figure 5-5. Comparison of the 2-year 120-day discharge from Equation 5.3 to the gaging station estimates without the three gaging station estimates of 0.1 cfs

5.5 REGRESSION ANALYSIS FOR SEASONAL LOW FLOWS

Based on interaction with fish biologists with the Maryland Department of Natural Resources and the University of Maryland, it was determined that fish migration in Maryland is prevalent in the Spring during the March to June time period (most species) and in the Fall during the September to November time period (trout). Flows during times of fish migration may be useful flow characteristics to analyze. Data for 16 gaging stations with drainage areas ranging from 1.49 to 48.9 square miles were used to evaluate selected seasonal flow characteristics. Of the 16 stations, only one station had an impervious area greater than 12 percent so these stations were basically rural watersheds. Using daily flow data at 16 gaging stations, the mean flows for the March to June period and September to November period were determined for each year. A Pearson Type III frequency distribution was fit to the logarithms of these annual flows for each time of the year. The following flow statistics were analyzed:

- March to June mean flow that has a 90-percent annual chance of exceedance,
- March to June mean flow that has a 10-percent annual chance of exceedance,
- September to November mean flow that has a 90-percent annual chance of exceedance, and
- September to November mean flow that has a 10-percent annual chance of exceedance.

Regression equations based only on drainage area were developed for estimating the seasonal flow characteristics described above and are provided in Attachment 5-2 for informational purposes. Because the regression equations were only based on 16 stations that were primarily rural stations, these equations have less applicability than Equations (5.1) to (5.4) described earlier.

5.6 FUTURE TOPICS FOR RESEARCH

Regression equations were developed for estimating the 2- and 10-year 90- and 120-day duration discharges for small watersheds in Maryland where the drainage areas were less than 10 square miles. The watershed characteristics used in the analysis were obtained from previous regression analyses for flood discharges. These watershed characteristics are more appropriate for predicting flood runoff rather than low flows. The accuracy of Equations 5.1-5.4 could be improved by developing explanatory variables that are based on geological or groundwater characteristics. For example, groundwater related variables like yields from wells or depth to groundwater would likely be statistically significant in estimating low flows like the 2-year 120-day discharge. Development of geologic and groundwater related variables was beyond the scope of the current project.

Future research should be conducted to estimate geologically-based explanatory variables that are more appropriate for estimating low flows and develop new regression equations. The USGS has performed several low flow regional analyses over the years and USGS reports that are documented at http://water.usgs.gov/osw/programs/nss/pubs.html were reviewed to determine geologically-based explanatory variables that were shown to be statistically significant in estimating low flows. The low-flow reports at the above cited USGS web site are those for which the low-flow regression equations have been incorporated into the USGS National Stream Statistics (NSS) Program (Ries, 2007).

For low-flow studies in Alabama and Tennessee, Bingham (1982, 1985) developed a streamflow recession index that was indicative of the rate of streamflow recession during base (low) flow and estimated in days per log cycle for discharge depletion (see Figure 5-6). The streamflow recession index is controlled by hydraulic characteristics of the aquifers and is highly correlated with low flows in the stream. Bingham (1982, 1985) determined the streamflow recession index at several gaging stations and then mapped this variable using a geologic map. The gaging station data were used to develop the regression equations, and the mapped value of the stream recession index was then used to estimate low flows at ungaged sites.

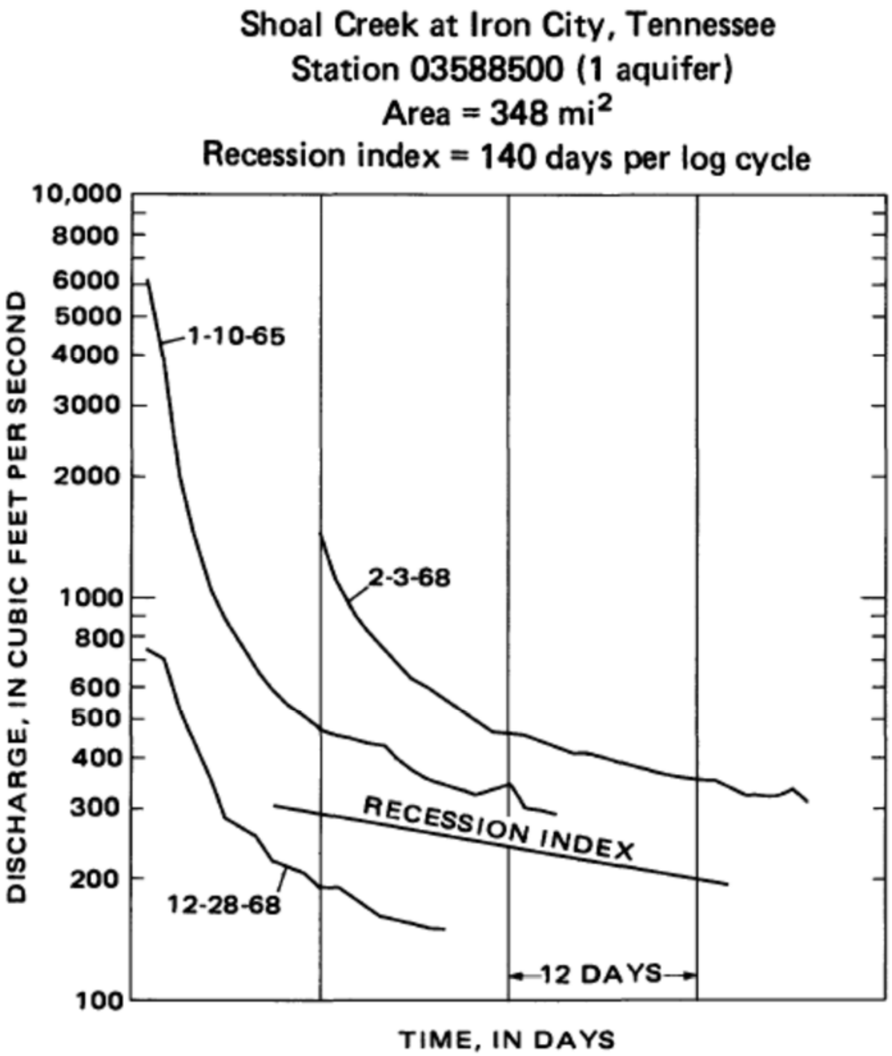

Figure 5-6: Schematic of determining the streamflow recession index

The streamflow recession index was found to be statistically significant for estimating low flows in other states as well. Funkhouser and others (2008) determined that the streamflow (baseflow) recession index was statistically significant for estimating low flows in Arkansas. Curran and others (2012) determined that the streamflow (baseflow) recession index was statistically significant for estimating low flows in Washington. For these studies, the streamflow recession index was mapped based on values estimated at gaging stations and using geologic maps.

In addition to the streamflow recession index, the following geological-based explanatory variables were found to be statistically significant in at least one hydrologic region in a given state:

- In Indiana, the ratio of the 20-percent flow duration value to the 90-percent flow duration value that was mapped using gaging station data and surficial geology maps (Arihood and Glatfelter, 1991),
- In Pennsylvania, stream density in sum of stream miles in the watershed divided by drainage area, soil thickness in depth to bedrock (in feet), percent glaciation of the watershed, and percent carbonate rock (Stuckey, 2006),
- In Ohio, an index of relative infiltration determined from by the fraction of the watershed covered by eight different soil groups, NOT related to A, B, C or D hydrologic soils (Koltun and Schwartz, 1987),
- In Idaho, percent of surficial volcanic rock (Hortness, 2006), and
- In Kentucky, streamflow variability index estimated as the standard deviation of the base 10 logarithms of 19 flow duration values from 5- to 95-percent (Martin and Arihood, 2010).

For the current analysis, the annual minimum 90- and 120-consecutive day discharges were used in developing the regression equations. The annual minimum n-day discharges normally occur in late summer and early fall, which roughly corresponds to the period of trout spawning and migration in Maryland streams.

Another topic for future research is to develop regression equations for seasonal flow characteristics that are consistent with fish spawning and migration in Maryland streams. During the course of this project, the use of seasonal flow characteristics was investigated using data for 16 gaging stations with drainage areas ranging from 1.49 to 48.9 square miles. The regression equations from this analysis are given in Attachment 5-2 of Section 5.9. Further research into seasonal flow analysis is warranted.

5.7 SUMMARY FOR LOW FLOW ANALYSIS

Daily flow data for 50 gaging stations with drainage areas less than 10 square miles and daily flow records in excess of 10 years were used to develop regression equations for estimating 2- and 10-year 90- and 120-consecutive day discharges. The USACE HEC-SSP program (USACE, 2010) was used to define the T-year N-day discharges at the 50 gaging stations by fitting a Pearson Type III frequency distribution to the logarithms of the annual minimum 90- and 120-day discharges. Drainage area, impervious area, and land slope were determined to be the most statistically significant watershed characteristics. The watershed characteristics used in defining Equations 5.1 to 5.4 are more indicative of flood flows and improvements in the regression equations could be realized through further research on:

- Development of geologic or groundwater characteristics that should be highly correlated with low flows, and
- Investigation into seasonal flow characteristics that might be more indicative of fish spawning and migration in Maryland streams.

There were two gaging stations where the 10-year 90- and 120-day discharges were zero. Since all data were transformed to logarithms for the linear regression analysis, a constant of 0.1 cfs was added to all 10-year discharges. Therefore, a constant of 0.1 cfs should be subtracted from the 10-year estimates in Equations 5.2 and 5.4. If the estimate becomes negative, then use zero as the estimated value.

The 2- and 10-year discharges for durations of 90- and 120-days (Equations 5.1 to 5.4) should be meaningful in designing culverts for fish passage in Maryland. The recommendation is to continue research in developing regression equations for low flows in Maryland.

5.8 ATTACHMENT 5-1. SUMMARY OF DATA USED IN THE REGRESSION ANALYSIS

Attachment 5-1 summarizes data for the 50 gaging stations used in the regression analysis. The three most statistically significant watershed characteristics in the regression analysis were drainage area, impervious area and land slope. The data in Attachment 1 include:

Station name
Station number
Drainage area, in square miles
Impervious area, in percent of drainage area
Land slope, in feet per foot, slope of the watershed, not the main channel slope
2-year 90-day low flow, in cubic feet per second (cfs)
10-year 90-day low flow, in cubic feet per second (cfs)
2-year 120-day low flow, in cubic feet per second (cfs)
10-year 120-day low flow, in cubic feet per second (cfs)

Station name	Station Number	Drainage area (mi^2)	Impervious area (%)	Land Slope (ft/ft)	90 day		120 day	
					Q2	Q10	Q2	Q10
Bacon Ridge Branch at Chesterfield	01590500	7	1.5	0.104	4.8	2.4	5.2	2.7
Baisman Run at Broadmoor	01583580	1.49	8.4	0.108	0.8	0.3	0.8	0.3
Basin Run at Liberty Grove	01579000	5.08	2.9	0.06	2.5	1.5	3.1	1.8
Beaverdam Run at Matthews	01492000	5.49	0.6	0.0069	0.4	0.2	0.7	0.3
Beetree Run at Bentley Springs	01581960	9.66	4.8	0.098	8.3	4.7	8.9	5.1
Birch Branch at Showell	0148471320	5.77	1.7	0.005	1.2	0.4	1.7	0.6
Brien Run at Stemmers Run	01585400	1.95	36.8	0.036	0.8	0.5	1	0.7
Bynum Creek at Bel Air	01581500	8.79	12.9	0.048	3.8	1.6	4.9	2.1
Cocktown Creek near Huntington	01594600	3.9	8.7	0.086	1	0.4	1.2	0.6
Cranberry Branch nr Westminister	01585500	3.26	4.2	0.081	1.3	0.5	1.5	0.6
Dead Run at Franklintown	01589330	5.52	41.1	0.047	2.8	1.3	3.6	1.7
East Branch Herbert Run at Arbutus	01589100	2.47	33.8	0.054	1.5	1	1.8	1.2
Faulkner Branch at Federalsburg	01489000	7.69	3	0.0104	2.1	1.1	2.6	1.2
Fishing Creek near Lewistown	01641500	7.3	0	0.141	2.3	1.4	2.7	1.5
Grave Run near Beckleysville	01581830	7.56	5.4	0.097	5.2	2.6	5.6	2.9
Gwynns Falls at Glyndon	01589180	0.308	42	0.026	0.1	0	0.1	0
Gwynns Falls near Delight	01589197	4.09	37.7	0.049	2.5	1.5	3	1.8
Gwynns Falls near Owings Mill	01589200	4.89	14.6	0.0559	2.7	1.6	3	1.8
Honeygo Run at White Marsh	01585104	2.44	22.5	0.054	1.2	0.5	1.5	0.6
Hunting Creek near Foxville	01640965	2.19	0	0.149	0.3	0.1	0.4	0.1
Hunting Creek Tributary nr Foxville	01640970	3.91	1.2	0.1188	0.8	0.4	1.1	0.6
Killpeck Creek at Huntersville	01594710	3.26	4.1	0.053	1.5	0.7	1.6	0.8
Laurel Run at Dobbin Road nr Wilson	01594930	8.23	1.1	0.155	6.3	3	7.9	3.5
Little Catoctin Creek at Harmony	01637000	8.76	0.8	0.152	2.2	1.1	3	1.4
Little Falls Branch near Bethesda	01646550	4.09	32.4	0.0517	1.4	0.7	1.7	1
Long Green Creek at Glne Arm	01584050	9.31	5.7	0.065	5	2.7	5.6	2.9
Manokin Branch near Princess Anne	01486000	5.02	1.5	0.0035	0.7	0.2	1	0.2
McMillian Fork near Fort Pendleton	01594950	2.36	1.2	0.13	0.4	0.1	0.6	0.1
Minebank Run near Glen Arm	0158397967	2.095	40.2	0.091	1.5	0.8	1.7	1
Mingo Branch near Hereford	01581940	0.765	2.5	0.105	0.5	0.1	0.5	0.2
Moores Run at Radecke Ave at Baltimore	01585230	3.5	45.4	0.045	1.7	1.1	2.1	1.3
Moores Run Trib at Baltimore	01585225	0.143	41.1	0.051	0.1	0.1	0.1	0.1
NB Rock Creek near Norbeck	01647720	9.68	9.9	0.0533	3.9	1.8	4.8	2.2
NF Whitemarsh Run nr White Marsh	01585095	1.36	42.9	0.049	0.7	0.4	0.9	0.5

Station name	Station Number	Drainage area (mi^2)	Imper-vious area (%)	Land Slope (ft/ft)	90 day		120 day	
					Q2	Q10	Q2	Q10
North Fork Sand Run near Wilson	01594936	1.91	0.5	0.144	0.9	0.3	1.1	0.4
North River near Annapolis	01590000	8.9	2.7	0.082	5.9	3.6	6.5	4.1
Owens Creek at Lantz	01640500	6.1	0.5	0.1263	1.5	0.6	2	0.8
Plumtree Run near Bel Air	01581752	2.47	42.9	0.048	2	1.1	2.3	1.4
Pond Branch at Oregon Ridge	01583570	0.131	0	0.101	0.1	0	0.1	0
Principio Creek nr Principio Furnace	01496200	9	1	0.0639	4.5	2.7	5.2	3
Sallie Harris Creek near Carmichael	01492500	7.49	0.1	0.009	4	2.4	4.4	2.6
Sawmill Creek at Glen Burnie, MD	01589500	4.9	23.5	0.026	3.7	0.5	3.7	0.7
SF Jabez Branch at Millersville	01589795	1	16.8	0.04	0.3	0.2	0.3	0.2
Slade Run near Glyndon	01583000	2.05	1.2	0.088	1.2	0.6	1.3	0.7
St Leonard Creek near St Leonard	01594800	6.8	0.3	0.088	2.8	0.9	3.3	1.2
Stemmers Run at Rossville	01585300	4.52	25.3	0.064	2	1	2.7	1.4
Watts Branch at Rockville	01645200	3.7	26.2	0.056	1.6	0.9	1.9	1.1
WB Herring Run at Idlewylde	01585200	2.31	42.1	0.059	1.2	0.7	1.4	0.9
White Marsh Run at White Marsh	01585100	7.56	37.7	0.061	1.7	2.4	2	2.8
White Marsh Run near Fullerton	01585090	2.58	44	0.068	4.2	0.8	5	0.9

5.9 ATTACHMENT 5-2. ANALYSIS OF SEASONAL FLOW CHARACTERISTICS

Based on interaction with fish biologists with the Maryland Department of Natural Resources and the University of Maryland, it was determined that fish migration in Maryland is prevalent in the Spring during the March to June time period (most species) and in the Fall during the September to November time period (trout). Flows during times of fish migration may be useful flow characteristics to analyze. Data for 16 gaging stations with drainage areas ranging from 1.49 to 48.9 square miles were used to evaluate selected seasonal flow characteristics. Of the 16 stations, only one station had an impervious area greater than 12 percent so these stations were basically rural watersheds.

Using daily flow data at 16 gaging stations, the mean flows for the March to June period and September to November period were determined for each year. A Pearson Type III frequency distribution was fit to the logarithms of these annual flows for each time of the year. The following flow statistics were summarized:

- March to June mean flow that has a 90-percent annual chance of exceedance,
- March to June mean flow that has a 10-percent annual chance of exceedance,
- September to November mean flow that has a 90-percent annual chance of exceedance, and
- September to November mean flow that has a 10-percent annual chance of exceedance.

5.9.1 March to June

The March to June mean flows with a 90- and 10-percent annual chance of exceedance are plotted in Figure 5-7 versus drainage area. The regression equations based on just drainage area in Figure 5-7 could be used to estimate the March-June mean flow with 90- and 10-percent chance exceedance. The equations for the March-June mean flows, based on only drainage area (DA) in square miles, are as follows:

$$\text{March-June } Q_{90\%} = 0.5993\ DA^{1.1038} \quad (5.5)$$

$$\text{March-June } Q_{10\%} = 1.9281\ DA^{1.0512} \quad (5.6)$$

The relation of the March to June mean flows is reasonably well described with just drainage area with Rsquare values of 0.9613 for the 10-percent annual chance exceedance flow and 0.9088 for the 90-percent annual chance exceedance flows. Both trend lines in Figure 5-7 have slopes slightly larger than 1.0, implying the discharges are directly proportional to drainage area. These flows correspond to the high fish passage and low fish passage discharges that are exceeded 10- and 90-percent of the time, respectively, during fish migration in the March to June time period as defined by Federal Highway Administration (FHWA) in HEC 26.

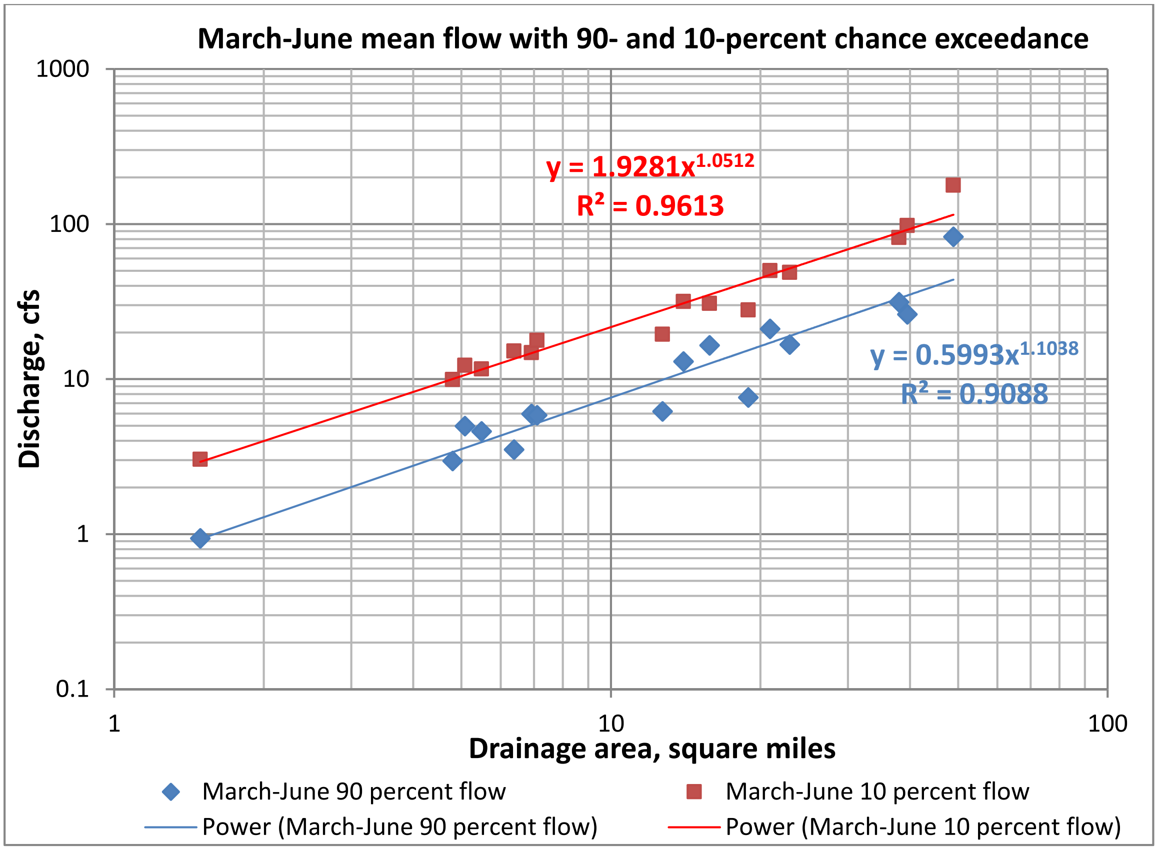

Figure 5-7: The March-June mean flow with 90- and 10-percent annual chance of exceedance plotted versus drainage area

5.9.2 September to November

The September to November mean flows with 90- and 10-percent annual chance of exceedance are plotted in Figure 5-8 versus drainage area. The regression equations based on just drainage area in Figure 5-8 could be used to estimate the September to November mean flow with 90- and 10-percent chance exceedance. The equations for the September-November mean flows, based on only drainage area (DA) in square miles, are as follows:

$$\text{September-November } Q_{90\%} = 0.223\ DA^{1.0621} \tag{5.7}$$

$$\text{September-November } Q_{10\%} = 1.464\ DA^{1.0265} \tag{5.8}$$

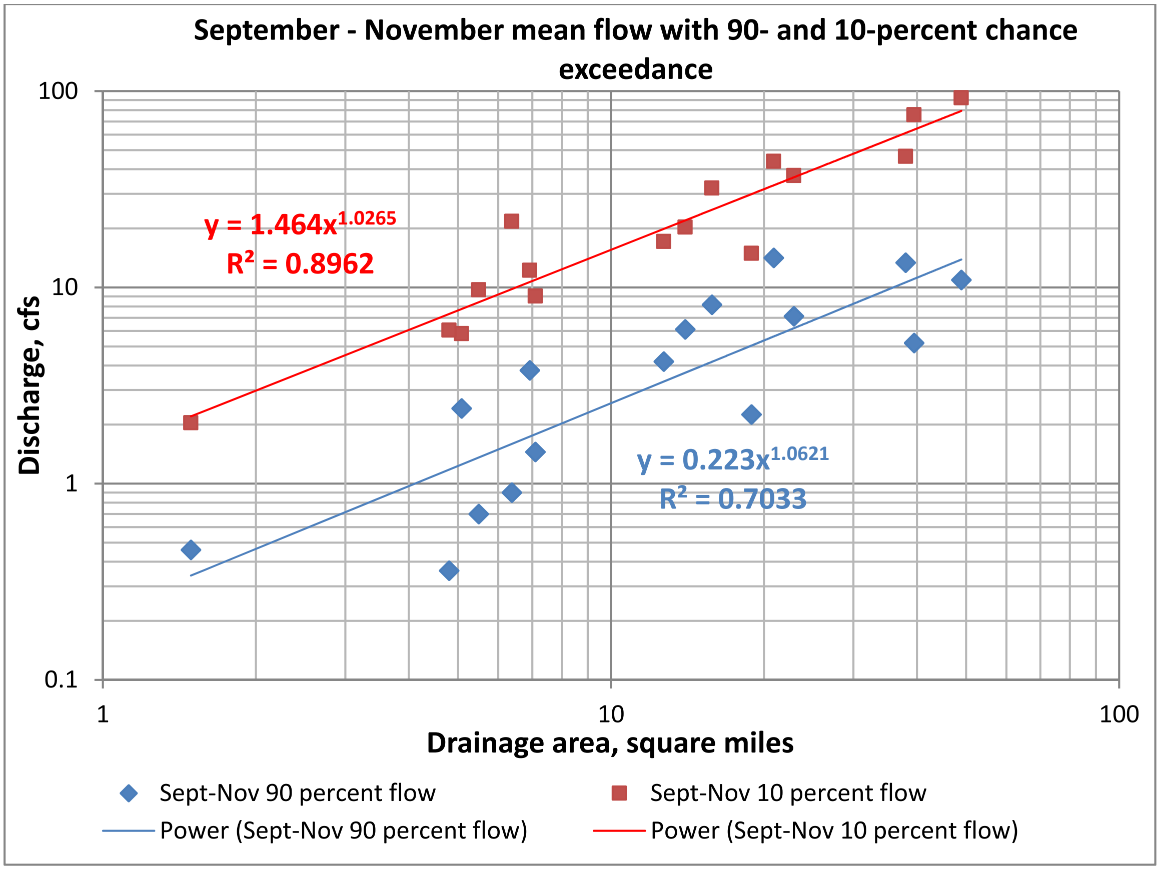

Figure 5-8: The September-November mean flow with 90- and 10-percent annual chance of exceedance plotted versus drainage area

The relation of the September to November mean flows is reasonably well described with just drainage area for the 10-percent annual chance exceedance flow with a Rsquare value of 0.8962. The 90-percent annual chance exceedance flow is not as well defined with a Rsquare value of 0.7033. The lower flows have more variability and are not as accurately estimated using only drainage area. Both trend lines in Figure 5-8 have slopes slightly larger than 1.0 implying the discharges are directly proportional to drainage area. These flows correspond to the high fish passage and low fish passage discharges that are exceeded 10- and 90-percent of the time, respectively, during fish migration in the September to November time period as defined by Federal Highway Administration (FHWA) in HEC 26.

The flows described in this section are analogous to low and high design flows recommended by the U.S. Fish and Wildlife Service (2019) and defined as:

- Low design flow: the mean daily flow that is exceeded 95 percent of the time during the migratory period of record based on a flow duration curve, and
- High design flow: the mean daily flow that is exceeded 5-percent of the time during the migratory period of record based on a flow duration curve.

5.10 FLOW DURATION DATA FOR MARYLAND STREAMS

Background

Regression equations for estimating the 2- and 10-year and 90- and 120-consecutive day low flows for small streams in Maryland less than 10 square miles are described in Section 5.4. The low flow regression equations are based on data for 50 gaging stations with 10 or more years of daily flow data through 2012 (if available) and the explanatory variables are drainage area, in square miles, percent of the watershed in impervious area, and land slope, in feet per foot. One anticipated use of the regression equations was to estimate low flows for fish passage in the design of culverts. The 90- and 120-consecutive day flows were used because these values tend to be substantially greater than zero. For durations less than 90 days, the low flows are often zero or very close to zero for small streams in Maryland and flows near zero are not useful in designing culverts. Flow duration data were developed for comparison to and to complement the low flow data for estimating flows for fish passage in the design of culverts.

Flow duration data describe the percent of the time that daily flows are exceeded during the period of record. These data are often used in stream restoration studies in concert with sediment transport curves (Bledsoe and others, 2016). A logical question is how do the 2- and 10-year and 90- and 120-conecutive day low flows compare to flow duration data. The following flow duration data are published annually by the U.S. Geological Survey (USGS) as part of their Annual Water Data Reports (https://wdr.water.usgs.gov/):

- The daily flow that is exceeded 10-percent of the time over the period of record (P10),
- The daily flow that is exceeded 50-percent of the time over the period of record (P50), and
- The daily flow that is exceeded 90-percent of the time over the period of record (P90).

The USGS published Annual Water Data Reports in paper format for many years including the daily flows and selected streamflow statistics. From 2006 to 2013, the USGS provided the annual water data reports available in electronic format on the internet (https://wdr.water.usgs.gov/). As of 2014, the USGS National Water Information System (NWIS) web server provides an on-demand, print-ready water-year summary as an annual water-data product.

The flow duration data for gaging stations with ending years of record from 2006 to 2013 are readily available from the internet without any data analysis. The USGS annual water data reports were accessed for the 50 small stream stations used to develop the low flow equations (Thomas and others (2014). Of the 50 stations, 19 were discontinued before 2006 and the flow duration were not readily available on the USGS web site. Data through the 2013 water year (or latest year available) were compiled for 31 stations with drainage areas less than 10 square miles. To represent a wider range of drainage areas, flow duration data for an additional 26 gaging stations with drainage areas less than 50 square miles were compiled. The flow duration data and watershed characteristics for 57 gaging stations used on the flow data analysis are given in Table 5-2 at the end of this section.

Comparison of Flow Duration Percentiles to T-year N-consecutive day Low Flows

In compiling the flow duration data, it became obvious that the daily flow exceeded 90-percent of the time (P90) was very similar to the 10-year 90-day low flow and the 10-year 120-day low flow. The 90-percent flow duration value was most comparable to the 10-year 90-day flow and that comparison is given in Figure 5-9.

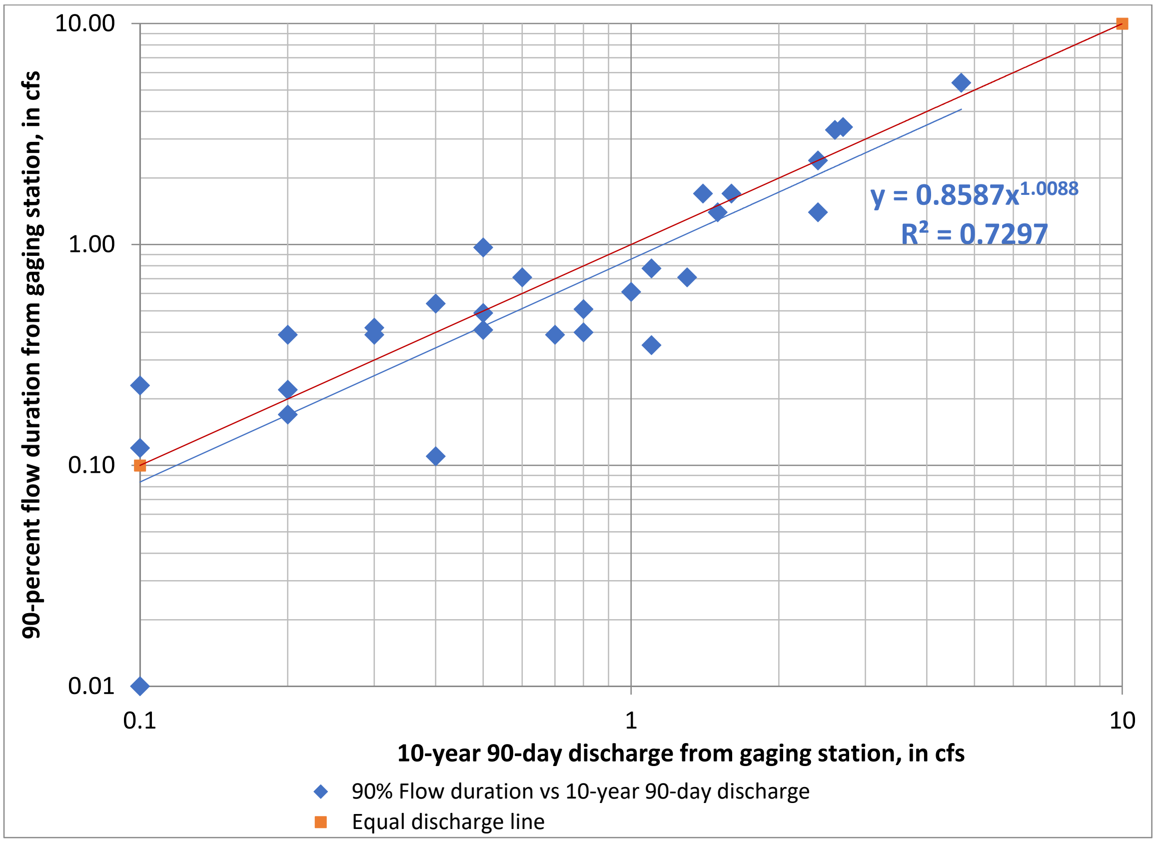

Figure 5-9: Comparison of the 90-percent flow duration value to the 10-year 90-day discharge for 31 small stream sites in Maryland with drainage areas less than 10 square miles

As shown in Figure 5-9, the best fit trend (blue) line relating P90 to the 10-year 90-day discharge has a slope of 1.0088 and is very close to the Equal Discharge (red) line. On average across all drainage areas, the 90-percent flow duration (P90) is about 15 percent less than the 10-year 90-consecutive day low flow for the 31 stations analyzed. The flow duration data are based on a water year (October 1 to September 30) over the period of record while the annual low flow statistic is based on a climatic year (April 1 to March 31) and annual minimum low flows. However, the comparison is still considered relevant.

Estimation of Flow Duration Percentiles

The 90-, 50- and 10-percent flow duration values were correlated with drainage area for the 31 small stream stations used in the low flow analysis and an additional 26 gaging stations with drainage areas from 10 to 50 square miles. Figure 5-10 gives the relation between the 90-percent flow duration (P90) and drainage area (DA) for 57 gaging stations in Maryland with drainage areas less than 50 square miles. The data plotted in Figure 5-10 are given in Table 5-2.

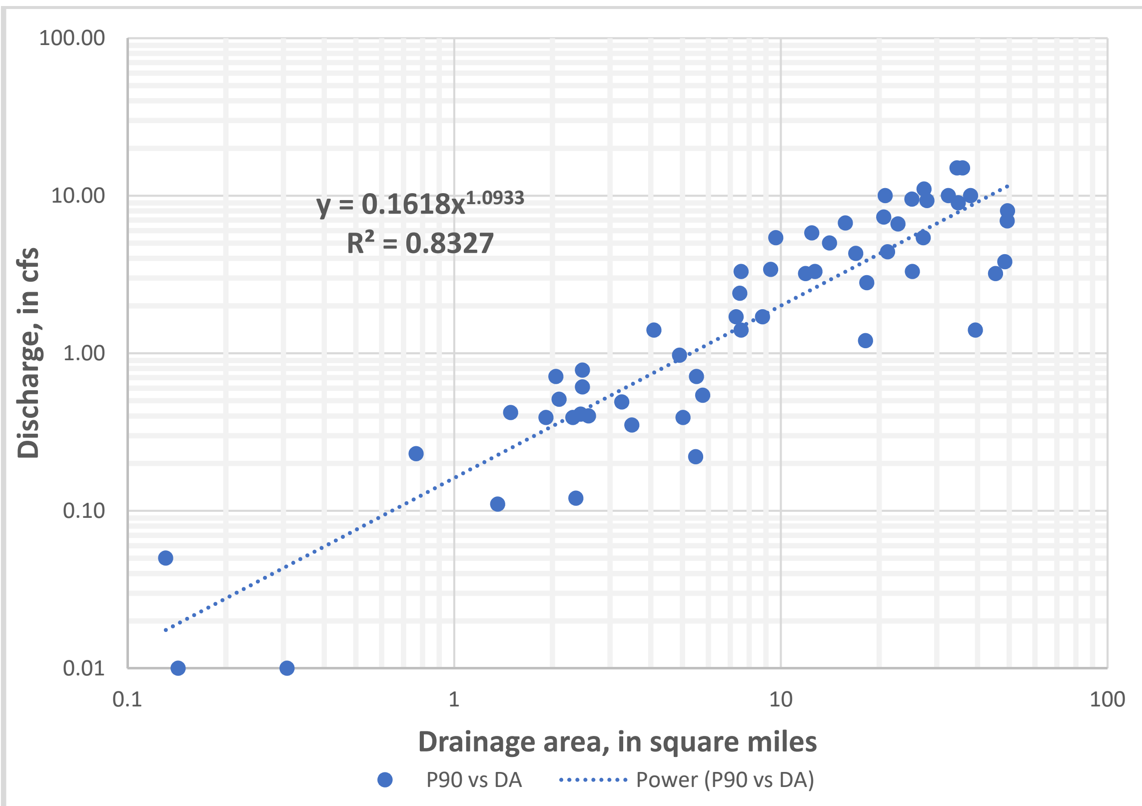

Figure 5-10: Relation between the 90-percent flow duration and drainage area for 57 gaging stations in Maryland with drainage areas less than 50 square miles

As shown in Figure 5-10, the 90-percent flow duration value is about 0.16 cfs, **on average**, for a one-square mile watershed and about 2.0 cfs for a 10-square mile watershed. The trend line in Figure 5-10,

$$P90 = 0.1618\ DA^{1.0933}, \qquad (5.9)$$

may be useful for estimating flow duration values in Maryland. The R^2 value is 0.83 and the standard error of estimate is 80.0 percent for Equation 5.9.

Figure 5-11 gives the relation between the 50-percent flow duration value (P50) and drainage area (DA). The data used in Figure 5-11 are given in Table 5-2.

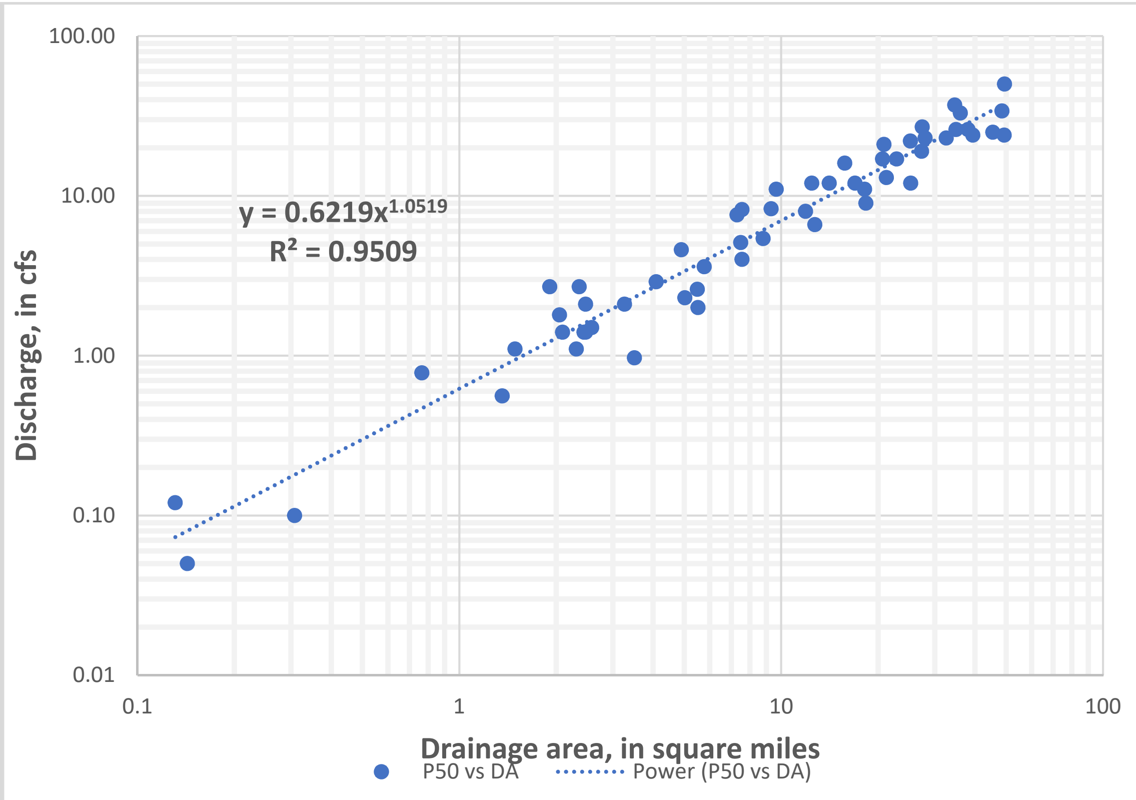

Figure 5-11: Relation between the 50-percent flow duration and drainage area for 57 gaging stations in Maryland with drainage areas less than 50 square miles

As shown in Figure 5-11, the 50-percent flow duration value (P50) is about 0.62 cfs, **on average**, for a one square mile watershed and about 7.0 cfs for a 10-square mile watershed. The trend line in Figure 5-11,

$$P50 = 0.6219\ DA^{1.0519}, \qquad (5.10)$$

may be useful in estimating flow duration values in Maryland. The R^2 value is 0.9509 and the standard error of estimate is 35.3 percent for Equation 5.10.

Figure 5-12 gives the relation between the 10-percent flow duration value (P10) and drainage area (DA). The data used in Figure 5-12 are given in Table 5-2.

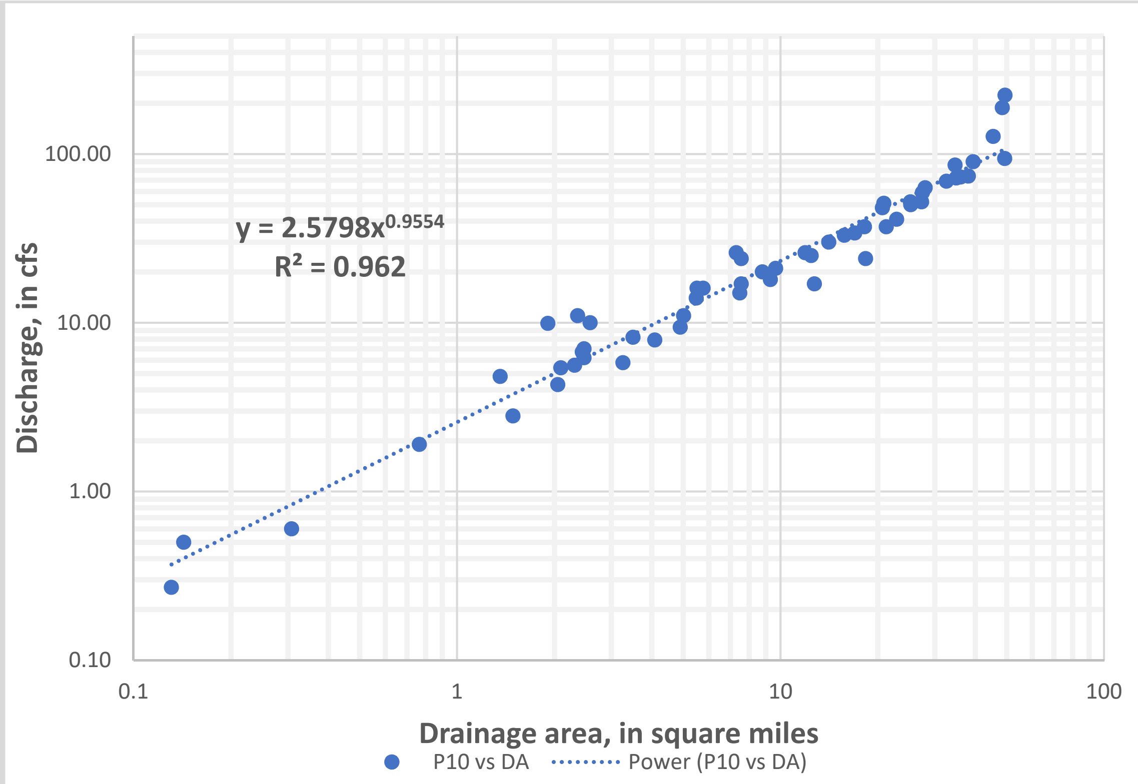

Figure 5-12: Relation between the 10-percent flow duration and drainage area for 57 gaging stations in Maryland with drainage areas less than 50 square miles

As shown in Figure 5-12, the 10-percent flow duration value is about 2.6 cfs, **on average**, for a one-square mile watershed and about 23.3 cfs for a 10-square mile watershed. The trend line in Figure 5-12,

$$P10 = 2.5798\ DA^{0.9554}, \qquad (5.11)$$

may be useful in estimating flow duration values in Maryland. The R^2 value is 0.962 and the standard error of estimate is 27.7 percent for Equation 5.11. As the flows get larger going from the 90th percentile to the 10th percentile, the standard errors of estimate decrease dramatically. Based on only drainage area, the 10- and 50-percent flow duration values has standard errors of 27.7 and 35.3 percent, respectively.

Comparison of Flood Discharges to Flow Duration Percentiles

The 10-percent flow duration percentile is compared to flood discharges to illustrate that the flow duration values are much less than even the more frequent flood discharges. The 1.25-year flood discharge, based on annual maximum instantaneous peak flows, has an 80 percent chance of being exceeded in any given year. In Figure 5-13, the 1.25-year flood discharge is compared to the 10-percent flow duration, that flow that has a 10-percent chance of being exceeded as a daily flow.

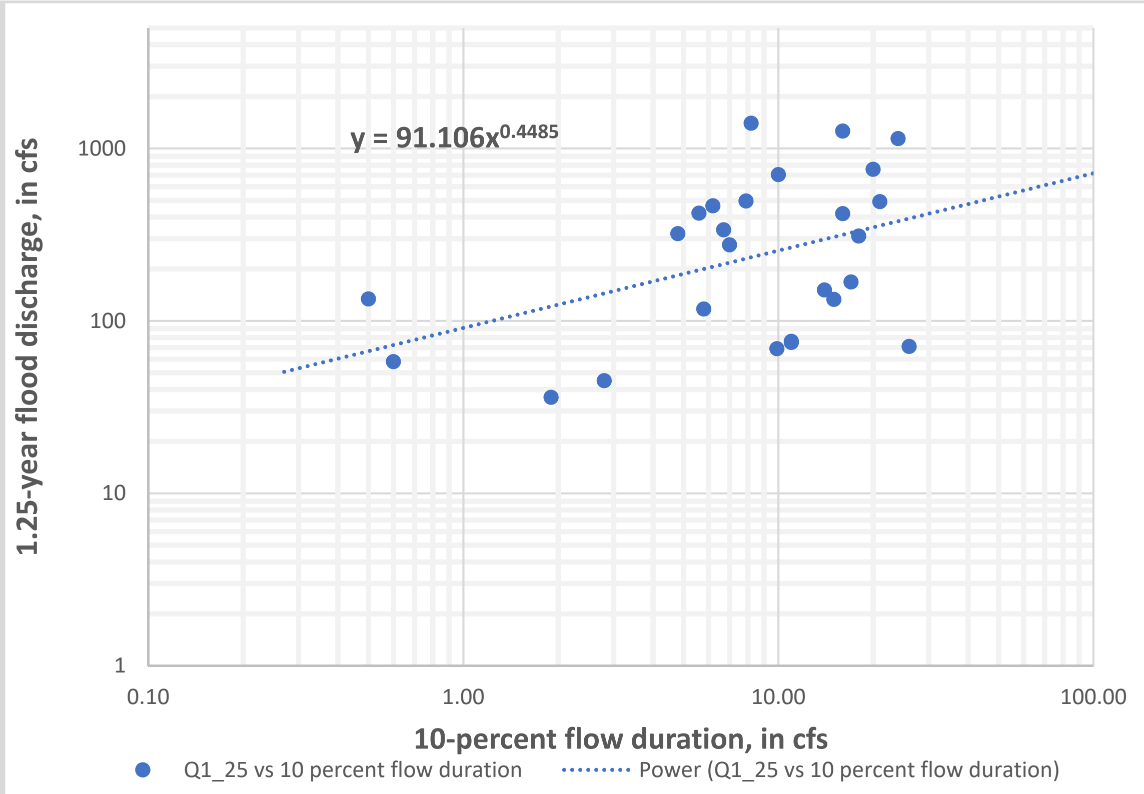

Figure 5-13: Comparison of 1.25-year flood discharge to the 10-percent flow duration percentile for 26 small watersheds in Maryland with drainage areas less than 10 square miles

As shown in Figure 5-13, when the 10-percent flow duration value is 1 cfs, the 1.25-year flood discharge is 91 cfs, on average. When the 10-percent flow duration value is 10 cfs, the 1.25-year flood discharge is about 256 cfs.

A similar comparison is given in Figure 5-14 for the 2-year flood discharge and the 10-percent flow duration value. The 2-year flood discharge has a 50-percent chance of being exceeded as an annual maximum instantaneous peak flow in any given year.

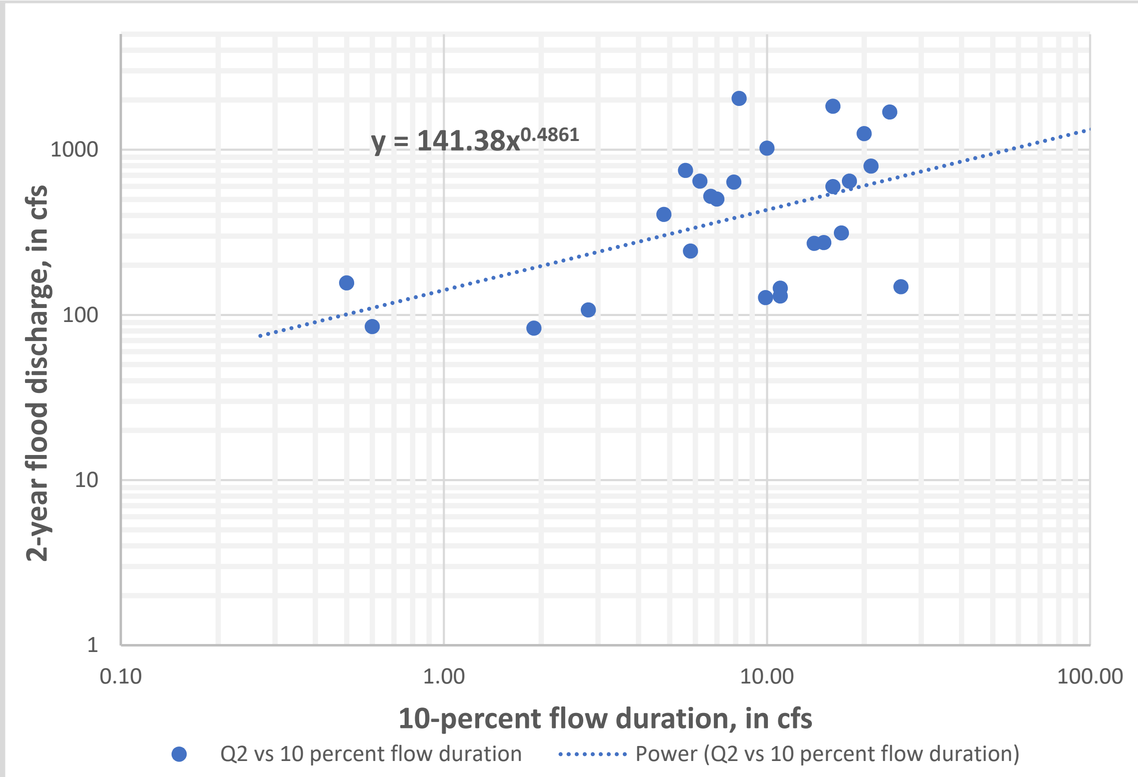

Figure 5-14: Comparison of 2-year flood discharge to the 10-percent flow duration percentile for 26 small watersheds in Maryland with drainage areas less than 10 square miles

As shown in Figure 5-14, when the 10-percent flow duration value is 1 cfs, the 2-year flood discharge is 141 cfs. When the 10-percent flow duration value is 10 cfs, the 2-year flood discharge is 433 cfs.

The purpose of Figures 5-13 and 5-14 is to illustrate the large difference in flow duration values and even the more frequent (small) flood discharges based on annual maximum instantaneous flows.

Summary

Some pertinent observations:

- The 90-percent flow duration value that is exceeded 90-percent of the time as a daily value is very comparable to the 10-year 90-consecutive day low flow based on annual minimum low flows.
- Equations based only on drainage area were developed for estimating the 10-, 50- and 90-percent flow duration values using flow duration data previously published by USGS. The equations for the 10- and 50-percent flow duration values have standard errors of 27. 7 and 35.3 percent, respectively, and may be useful for stream restoration projects in Maryland. The standard error of the 90-

percent flow duration values is 80 percent primarily because the flows are very small and even small deviations represent large percent errors.

- Even the larger 10-percent flow duration value is much less than even the more frequent flood discharges based on annual instantaneous maximum flows.

Table 5-2: Watershed characteristics and flow duration percentiles for flows exceeded 10-, 50- and 90-percent of the time for 57 gaging stations in Maryland with drainage areas less than 50 square miles.

Station name	Station Number	Drainage area (mi^2)	Impervious area (%)	Land Slope (ft/ft)	10% Flow (cfs)	50% Flow (cfs)	90% Flow (cfs)
Baisman Run at Broadmoor	01583580	1.49	8.4	0.108	2.80	1.10	0.42
Beaverdam Run at Matthews	01492000	5.49	0.6	0.0069	14.00	2.60	0.22
Beetree Run at Bentley Springs	01581960	9.66	4.8	0.098	21.00	11.00	5.40
Birch Branch at Showell	0148471320	5.77	1.7	0.005	16.00	3.60	0.54
Bynum Run at Bel Air	01581500	8.79	12.9	0.048	20.00	5.40	1.70
Cranberry Branch near Westminister	01585500	3.26	4.2	0.081	5.80	2.10	0.49
Dead Run at Franklintown	01589330	5.52	41.1	0.047	16.00	2.00	0.71
East Branch Herbert Run at Arbutus	01589100	2.47	33.8	0.054	6.20	1.40	0.61
Fishing Creek near Lewistown	01641500	7.3	0	0.141	26.00	7.60	1.70
Grave Run near Beckleysville	01581830	7.56	5.4	0.097	17.00	8.20	3.30
Gwynns Falls at Glyndon	01589180	0.308	42	0.026	0.60	0.10	0.01
Gwynns Falls near Delight	01589197	4.09	37.7	0.049	7.90	2.90	1.40
Honeygo Run at White Marsh	01585104	2.44	22.5	0.054	6.70	1.40	0.41
Long Green Creek at Glen Arm	01584050	9.31	5.7	0.065	18.00	8.30	3.40
Manokin Branch near Princess Anne	01486000	5.02	1.5	0.00349	11.00	2.30	0.39
McMillian Fork near Fort Pendleton	01594950	2.36	1.2	0.13	11.00	2.70	0.12
Minebank Run near Glen Arm	0158397967	2.095	40.2	0.091	5.40	1.40	0.51
Mingo Branch near Hereford	01581940	0.765	2.5	0.105	1.90	0.78	0.23
Moores Run at Radecke Ave at Balt	01585230	3.5	45.4	0.045	8.20	0.97	0.35
Moores Run Trib at Baltimore	01585225	0.143	41.1	0.051	0.50	0.05	0.01
NF Whitemarsh Run near White Marsh	01585095	1.36	42.9	0.049	4.80	0.56	0.11
North Fork Sand Run near Wilson	01594936	1.91	0.5	0.144	9.90	2.70	0.39
Plumtree Run near Bel Air	01581752	2.47	42.9	0.048	7.00	2.10	0.78
Pond Branch at Oregon Ridge	01583570	0.131	0	0.101	0.27	0.12	0.05
Sallie Harris Creek near Carmichael	01492500	7.49	0.1	0.009	15.00	5.10	2.40
Sawmill Creek at Glen Burnie, MD	01589500	4.9	23.5	0.026	9.40	4.60	0.97
South Fork Jabez Branch at Millersville	01589795	1	16.8	0.04	0.71	0.33	0.17
Slade Run near Glyndon	01583000	2.05	1.2	0.088	4.30	1.80	0.71
West Branch Herring Run at Idlewylde	01585200	2.31	42.1	0.059	5.60	1.10	0.39

Station name	Station Number	Drainage area (mi²)	Imper-vious area (%)	Land Slope (ft/ft)	10% Flow (cfs)	50% Flow (cfs)	90% Flow (cfs)
White Marsh Run at White Marsh	01585100	7.56	37.7	0.061	24.00	4.00	1.40
White Marsh Run near Fullerton	01585090	2.58	44	0.068	10.00	1.50	0.40
Nassawango Creek near Snow Hill	01485500	45.47	2.30	0.00841	127.00	25.00	3.20
Chicamacomico River near Salem	01490000	16.96	0.90	0.00757	34.00	12.00	4.30
Unicorn Branch near Millington	01493000	20.67	1.30	0.0127	48.00	17.00	7.30
Morgan Creek near Kennedyville	01493500	12.73	1.00	0.02445	17.00	6.60	3.30
Winters Run near Benson	01581700	34.64	8.10	0.07	86.00	37.00	15.00
Gunpowder Falls at Hoffmanville	01581810	27.46	4.90	0.112	59.00	27.00	11.00
Georges Run near Beckleysville	01581870	15.76	7.80	0.075	33.00	16.00	6.70
Piney Run at Dover	01583100	12.45	3.40	0.083	25.00	12.00	5.80
Beaverdam Run at Cockeysville	01583600	20.88	22.00	0.076	51.00	21.00	10.00
Little Gunpowder Falls at Laurel Brook	01584500	36.04	3.50	0.071	73.00	33.00	15.00
Beaver Run near Finksburg	01586210	14.11	11.90	0.079	30.00	12.00	5.00
Morgan Run near Louisville	01586610	28.01	4.90	0.089	63.00	23.00	9.30
Gwynns Falls at Villa Nova	01589300	32.59	19.50	0.056	69.00	23.00	10.00
Jones Falls at Sorrento	01589440	25.21	11.40	0.078	52.00	22.00	9.50
Patuxent River near Unity	01591000	34.95	1.40	0.092	72.00	26.00	9.00
Cattail Creel near Glenwood	01591400	22.86	4.30	0.08	41.00	17.00	6.60
Hawlings River near Sandy Spring	01591700	27.31	8.90	0.056	52.00	19.00	5.40
Little Patuxent River at Guilford	01593500	38.1	18.50	0.053	74.00	26.00	10.00
Dorsey Run near Jessup	01594400	11.91	16.70	0.051	26.00	8.00	3.20
Savage River near Barton	01596500	48.53	0.30	0.203	188.00	34.00	3.80
Marsh Run at Grimes	01617800	18.34	3.40	0.035	24.00	9.00	2.80
NW Branch Anacostia River nr Colesville	01650500	21.23	11.60	0.062	37.00	13.00	4.40
NW Branch Anacostia River nr Hyattsville	01651000	49.33	27.80	0.065	94.00	24.00	6.90
Piscataway Creek at Piscataway	01653600	39.43	11.60	0.057	90.00	24.00	1.40
St Clement Creek near Clements	01661050	18.18	3.40	0.059	37.00	11.00	1.20
St Marys River at Great Mills	01661500	25.29	6.10	0.041	50.00	12.00	3.30
Bear Creek at Friendsville	03076600	49.43	0.90	0.168	223.00	50.00	8.00

CHAPTER 6

6 Estimation of Discharges in Tidal Reaches

6.1 INTRODUCTION

Peak flows at tidal bridges on coastal streams in Maryland normally occur as a result of a combination of the following two elements of a hurricane or tropical storm event:

Storm surge hydrograph: Storm surge is the rising of the sea level due to the high winds, low atmospheric pressure and high waves associated with a hurricane or tropical storm. For design purposes, it can be characterized by a cosine curve with a high elevation as determined by Flood Insurance Studies of FEMA, a low elevation as determined by studies of the Hydrology Panel and a tidal period of approximately 24 hours. The difference in elevations from high elevation to low elevation is defined as the range of the storm surge, and the average of these two elevations is the mean storm surge elevation. The amplitude of the storm surge is equal to one-half the range. An example storm surge hydrograph for the Baltimore, MD tide station 8574680 for Hurricane Isabel (September 19, 2003) is shown in Figure 6-1. The mean storm surge elevation and tidal period is also illustrated in Figure 6-1. The peak storm surge elevation for Hurricane Isabel was 7.31 feet (NAVD88), the peak of record at this station in 113 years of record, and slightly greater than a 100-year event.

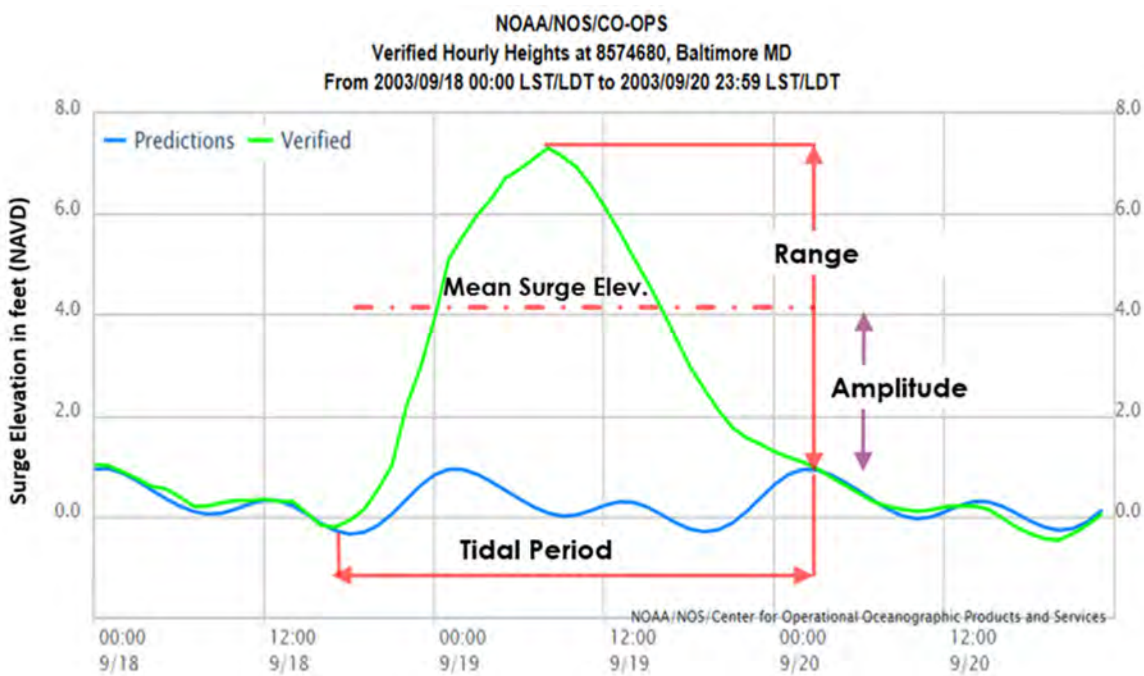

Figure 6-1: An example storm surge hydrograph for Baltimore, MD for September 18-20, 2003

An observed storm surge hydrograph for Hurricane Isabel in September 2003 (Figure 6-1) illustrates the characteristics of the surge hydrograph. For analyses at a bridge site, a T-year storm surge hydrograph is used for the analysis where the peak storm surge is the T-year Stillwater elevation from a FEMA Flood Insurance Study or similar studies. The tidal period in Figure 6-1 for the Hurricane Isabel event was about 30 hours and this varies by storm event. For analyses at tidal bridges, a 24-hour tidal period is assumed. The low point after the storm surge is the point at which the storm surge recedes to the normal tidal cycle. The low point elevation of the Hurricane Isabel event in Figure 6-1 was about 1.0 feet (NAVD88). An analysis of several storm surge hydrographs indicated that this is typical for major storm surge events in the Chesapeake Bay. The 1.0 ft elevation (NAVD88) is recommended as a tailwater elevation in scour computations where the tailwater elevation is influenced by the Chesapeake Bay.

Riverine hydrograph: The riverine hydrograph occurs as a result of the rainfall occurring during the hurricane or tropical storm that is falling on the drainage area of the tidal stream above the crossing of the structure under consideration. An example of an observed riverine hydrograph is given in Figure 6-2 for the Choptank River near Greensboro, MD (station 01491000) where the drainage area is 113.7 square miles. The hydrograph in Figure 6-2 represents the flooding for September 14-22, 1999, which occurred during Hurricane Floyd; the peak discharge of 6,420 cfs is a 20-year event (third highest flood in a 67-year record). The storm surge elevation during Hurricane Floyd was less than a 2-year event. The time base of the hydrograph is a function of the basin lagtime/time of concentration for the watershed.

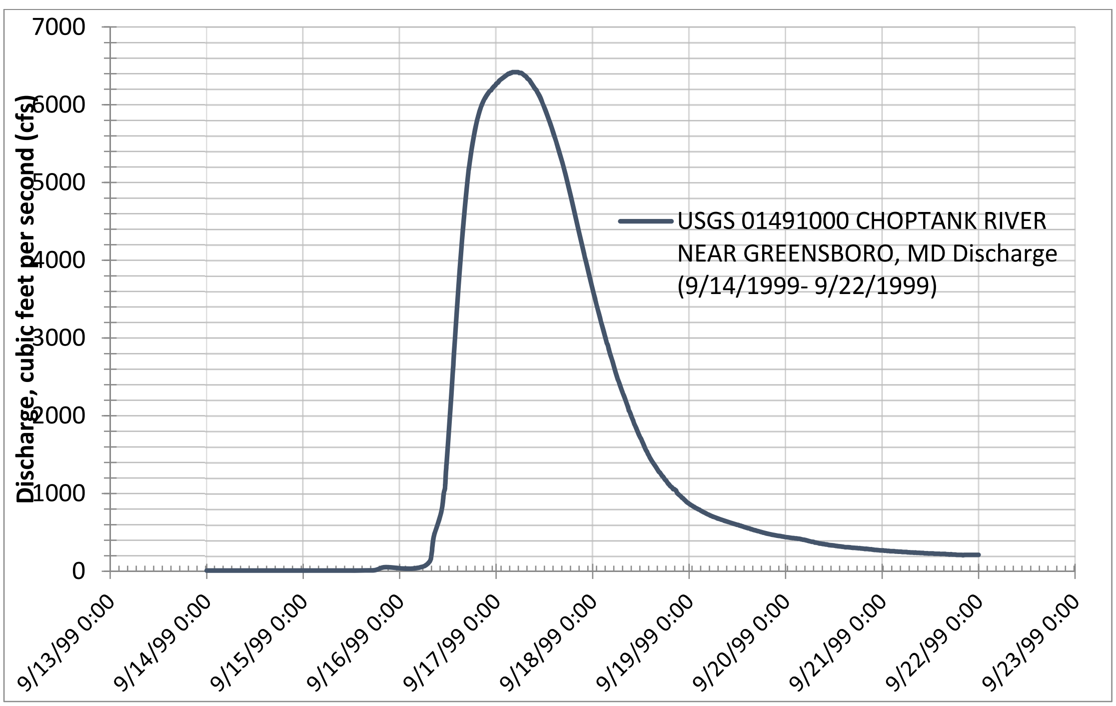

Figure 6-2. An example riverine hydrograph for the Choptank River near Greensboro, MD (station 01491000) for September 14-22, 1999

Many factors enter into the contribution of the storm surge and the riverine hydrograph and the peak flow through a structure for a given storm. The combined peak flow is needed to design the bridge opening and to evaluate scour. Determination of these flows is not subject to a rigorous analysis. Rather, the following guidance serves to provide for a conservative, yet reasonable, method for estimating peak flows at tidal bridges.

6.2 TIMING OF THE STORM SURGE AND RIVERINE HYDROGRAPHS

One important factor in determining the design discharges at tidal-affected bridges is the timing of the storm surge and riverine hydrographs. There are four long-term tide stations in Maryland; they are in the Chesapeake Bay (Baltimore, Annapolis, Cambridge and Solomons Island). A brief investigation was conducted with respect to the timing of peak storm surge elevations and peak riverine discharges and the frequency of those events. The long-term tide stations were utilized because sufficient data are available for determining the frequency (recurrence interval) of the events. Attachment 6-1 in Section 6.8 summarizes the timing and frequency of several surge and riverine flood events. There are a few constraints or data limitations in determining the relative timing of the storm surge and riverine flooding events:

- Concurrent data for flooding events need to be available for the tide and streamflow gaging stations; these data are not always available.
- The four long-term tide stations in Maryland are in the Chesapeake Bay and all stations experience the same storm surge events; therefore, a limited number of significant storm surge events are available for evaluation.
- The only storm surge event that approximated a 100-year event was Hurricane Isabel in September 19, 2003. The riverine events associated with Hurricane Isabel were about 2-year events. A significant rainfall-runoff event occurred four days after the storm surge event.
- Riverine hydrographs for 15-minute data or less are only available in electronic format for Maryland streams since October 1990. Before 1990, only annual maximum peak discharges and the day (date) of the peak discharge are available, which is not very helpful for evaluating the timing of peaks for the smaller streams.
- For events before 1990, the dates of the annual maximum peak discharge can be compared to the timing of the storm surge.
- Riverine gaging stations are generally located many miles upstream of the Chesapeake Bay to avoid the effects of tidal backwater; this complicates the evaluation of the timing of the peaks at bridges in tidal areas.

For evaluating the timing of the peak storm surge and peak riverine discharge, the candidate streamflow gaging stations were those that drained into the estuary of the tide station or an estuary close to the tide station. The drainage areas for the streamflow gaging stations varied from 1 square mile to 348.9 square miles.

The following observations resulted from the analysis described in Attachment 6-1:

- The major storm surge events generally do not produce major riverine peak discharges. That implies that significant storm surge events do NOT generally occur at the same time as significant riverine floods.
- Most of the annual maximum peak discharges occurred when the storm surges were low or during a time of year when hurricanes and tropical storms do not occur.
- For the larger streams (> 100 square miles) in the Coastal Plain regions, the riverine peaks occur long after the peak storm surge. In addition, the gaging stations are far upstream from the tide stations and have long times of concentration implying the peak riverine discharges occur much later than the peak storm surges at the tide stations.
- For the smaller streams (< 10 square miles) in the Coastal Plain regions, the timing of the peak storm surges and peak riverine discharges were much closer in timing with peaks sometimes occurring on the same day.
- During major storm surge events, watersheds greater than 100 square miles generally experienced events with recurrence intervals of 2 years or less. Conversely, during major riverine events, the storm surge events were on the order of a 2-year frequency.
- During major storm surge events, watersheds less than 10 square miles experienced riverine events with recurrence intervals ranging from 1.25 to 20 years.

6.3 APPROACH FOR ESTIMATING RIVERINE T-YEAR HYDROGRAPHS FOR WATERSHEDS GREATER THAN 300 SQUARE MILES

As described in Section 4.2, the WinTR-20 model is recommended for estimating riverine hydrographs for watersheds less than 300 square miles. The procedures for applying the WinTR-20 model are described in detail in earlier chapters of this report. For watersheds with drainage areas in excess of 300 square miles, a T-year riverine hydrograph can be estimated using the USGS dimensionless hydrographs described by Dillow (1998). The USGS dimensionless hydrographs for three hydrologic regions (Appalachian Plateau [AP], Piedmont [P], and Eastern and Western Coastal Plain [CP]) in Maryland are shown in Figure 6-3 [from Dillow (1998)] where the ordinate is the discharge divided by peak discharge and the abscissa is time divided by lagtime. Also shown in Figure 6-3 for comparison purposes are the NRCS dimensionless hydrograph and the Georgia dimensionless hydrograph, which is used in the USGS National Streamflow Statistics Program for flood hydrograph estimation. The lagtime as used by USGS is the time from the centroid of rainfall excess to the centroid of the runoff hydrograph. This differs from the NRCS lag time, which is defined as the time from the centroid of rainfall excess to the peak discharge. The USGS basin lagtime is, on average, about 95 percent of the time of concentration while the NRCS lag time is defined as 60 percent of the time of concentration.

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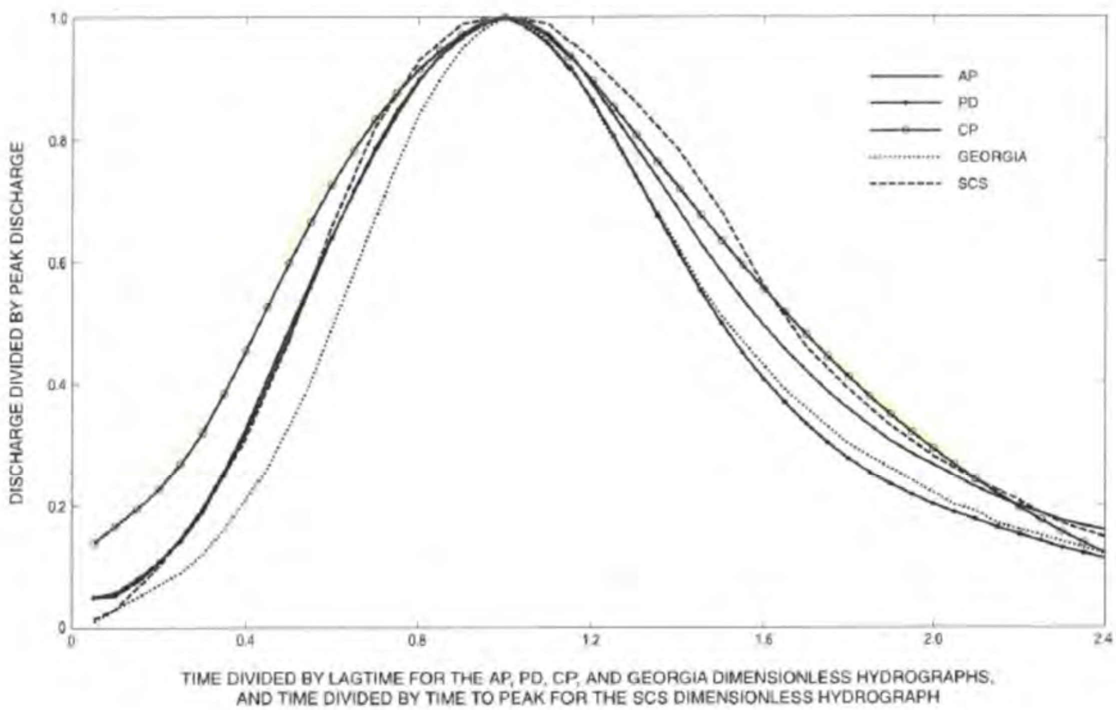

Figure 6-3: **Dimensionless hydrographs for the Appalachian Plateau (AP), Piedmont (PD), and Coastal Plain (CP) Regions in Maryland and the Georgia and NRCS (formerly SCS) dimensionless hydrographs with peaks aligned [from Dillow (1998)]**

The USGS dimensionless hydrographs for the three hydrologic regions are defined in tables in Dillow (1998). The estimation of a T-year hydrograph involves multiplying the ordinate by the T-year peak discharge and the abscissa by the basin lagtime and can be easily applied in a spreadsheet. The T-year peak discharge is estimated using the Fixed Region regression equations in Appendix 3 of this report and Dillow (1998) provides a regression equation for estimating the basin lagtime. The CP dimensionless hydrograph is mostly applicable for tidal streams because the tidal bridges are located in either the Western or Eastern Coastal Plain Regions.

An example of estimating the 100-year flood hydrograph for the Choptank River (station 01491000) using the CP dimensionless hydrograph is illustrated in Figure 6-4. The 100-year peak discharge of 10,400 cfs was estimated from a Bulletin 17B frequency analysis using the observed annual peak flows through 2011. A basin lagtime of 31.6 hours was estimated from observed rainfall-runoff events (Dillow, 1998). In order to evaluate the dimensionless hydrograph approach, the ordinates of the September 1999 flood hydrograph for Hurricane Floyd (Figure 6-2) were increased to match a peak discharge of 10,400 cfs to obtain an independent estimate of the 100-year flood hydrograph at the gaging station. The comparison in Figure 6-4 indicates that the USGS dimensionless hydrograph approach provides reasonable estimates of the T-year riverine hydrographs when compared to scaling up observed flood hydrographs. The USGS dimensionless hydrograph provides a quick approach for estimating T-year riverine hydrographs for large watersheds upstream of tidal bridges in Maryland.

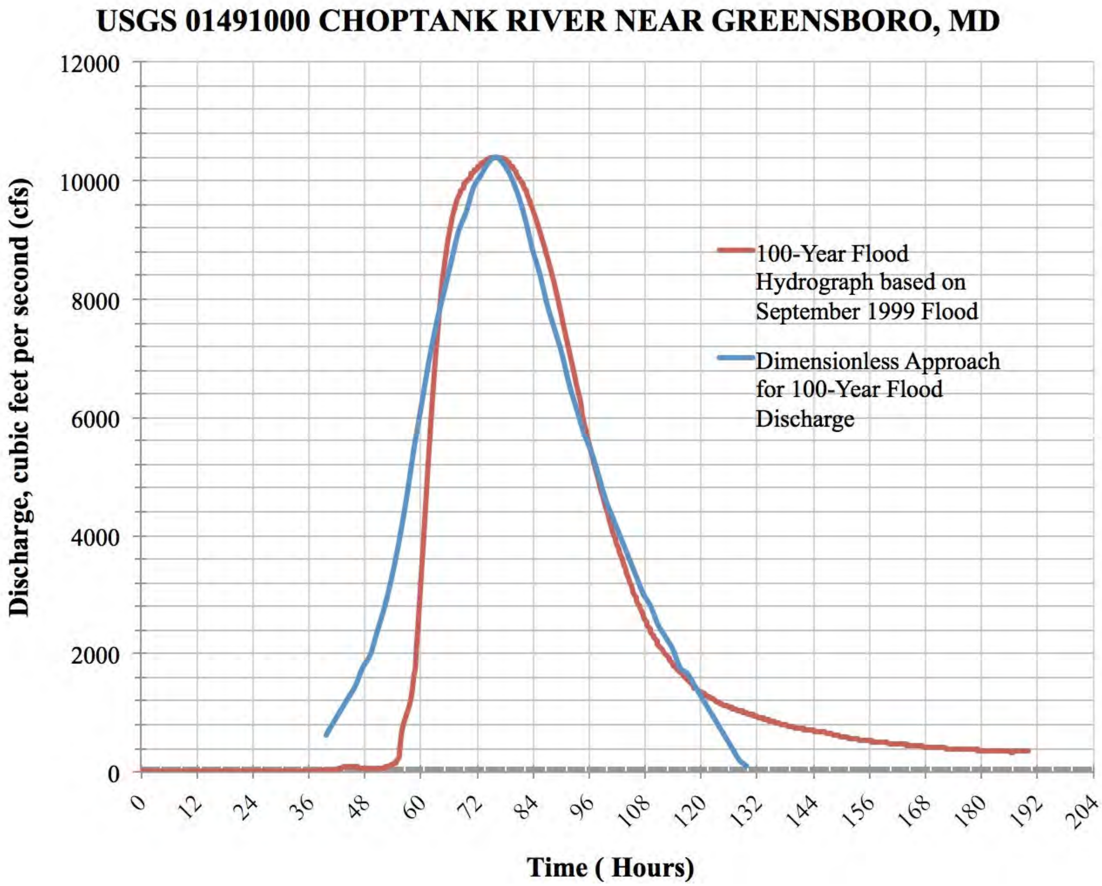

Figure 6-4: Comparison of 100-year hydrographs for the Choptank River near Greensboro, MD (station 014910000)

6.4 APPROACH FOR ESTIMATING MAXIMUM STORM SURGE DISCHARGE

The storm surge flow rate through a channel that is relatively unconstricted by a bridge opening depends on the rate at which the bay side of the bridge is filled or emptied since the head difference through the bridge is expected to be small. The maximum discharge occurs at an elevation halfway between the high and low storm surge elevation. Equation 6-1 can be used to estimate this maximum discharge:

$$Q_{max} = 3.14 \, (As * H) / T \qquad (6.1)$$

where
Q_{max} = maximum discharge in a tidal cycle in cubic feet per second,
As = surface area of the tidal basin at mean tide in square feet,

H = difference in elevation between high and low storm surge levels in feet, and
T = tidal period (24 hours) in seconds.

Equation 6.1 is used to estimate the maximum storm surge peak discharge that is combined with the riverine peak flow to estimate the total flow through the bridge opening. Using the data in Figure 6-1 to illustrate the use of Equation 6.1, the high elevation is the peak of the storm surge at 7.31 feet, the low elevation is about 1.0 foot, H is 6.31 feet and the mean tide is 4.16 feet. The surface area of the tidal basin upstream of the bridge at elevation 4.16 feet is the remaining variable to be determined. The surface area of the tidal basin upstream of the bridge is estimated using the best available topographic data.

6.5 MODELS FOR EVALUATING TIDAL FLOW

SHA uses two models for estimating discharges and water surface profiles in the vicinity of tidal bridges: TIDEROUT2 Scour and HEC-RAS. A brief purpose for each model is given but the user should consult the latest user's manuals for a more complete description of these programs.

<u>TIDEROUT2 Scour</u>

TIDEROUT2 Scour is a flood routing program developed by SHA (2015). Its primary purpose is for estimating discharges and scour at bridges in tidal waterways. The program can be used to route riverine flows from an upland watershed down to the tidal basin and then route the combined riverine/tidal flow through the bridge and over the road if needed. A riverine hydrograph can be entered into the program or estimated for a single watershed area using the NRCS dimensionless unit hydrograph. TIDEROUT2 Scour uses information on tidal characteristics, the volume of the tidal basin upstream of the bridge and the reservoir routing method (Inflow – Outflow = Δ Storage) to estimate a storm surge hydrograph for the bridge analysis. Recently, scour equations were incorporated into the TIDEROUT2 Scour program. Additional details on the program are provided in the TIDEROUT2 Scour program.

<u>HEC-RAS</u>

HEC-RAS is a hydraulic model developed by the U.S. Army Corps of Engineers (USACE) which is used to route riverine flows, estimate water surface profiles and compute bridge scour. The program implements Manning's equation in open channels, computes flow through bridges and culverts and flow over the road. MDOT SHA uses the steady state version of HEC-RAS that utilizes only peak discharges (no hydrographs). Additional details are provided on the program in the HEC-RAS User's Manual dated February 2016 (USACE, 2016).

6.6 RECOMMENDATIONS FOR COMBINING STORM SURGE AND RIVERINE DISCHARGES

The following recommendations are provided for estimating storm surge and riverine discharges at tidal bridges in Maryland. Each tidal bridge will present a different set of conditions to consider. Therefore, other approaches may be appropriate for specific tidal bridge site locations. However, approval of the Office of Structures is necessary prior to use of approaches different from the following guidance.

Tidal bridges in Maryland are located in the Eastern and Western Coastal Plain Regions. In those regions, the times of concentration for watersheds of 25 square miles are approximately 24 hours. If a watershed in one of the Coastal Plain regions exceeds 25 square miles, then it is likely the time of concentration is greater than 24 hours and the riverine and storm surge peaks will differ in time by at least 24 hours. The timing analysis in Attachment 6-1 verified that for the larger watersheds, the timing of the surge and riverine events generally differed by more than a day. However, the smaller watersheds sometimes experienced major storm surge and riverine peak discharges on the same day (within 24 hours). Therefore, different guidance is provided for analyzing watersheds less than and greater than 25 square miles.

The following assumptions are made about the coincidence of storm surge and riverine peak discharges:

- For watersheds 25 square miles or less, assume a 100-year or 500-year storm surge and a 10-year riverine peak discharge occur on the same event, and conversely, a 10-year storm surge and 100-year or 500-year riverine peak discharge occur on the same event.
- For watersheds greater than 25 square miles, assume a 100-year or 500-year storm surge and a 2-year riverine peak discharge occur on the same event, and conversely, a 2-year storm surge and 100-year or 500-year riverine peak discharge occur on the same event.

Limited data are available for gaging stations on very small tidal streams of a few square miles. For tidal bridges located close to the Chesapeake Bay for which the upstream drainage area is less than a few square miles, the same T-year event may occur on the same event. For example, the 100-year storm surge and the 100-year riverine peak discharge may occur at the same time. This is an example of a specific bridge for which the Office of Structures should be consulted.

Specific guidance is provided when using TIDEROUT2 Scour and HEC-RAS.

TIDEROUT2 Scour

Due to the lack of observed storm surge events on the order of a 100-year event, the same procedures are recommended for 100- and 500-year events because there are no data to indicate otherwise.

For watersheds of 25 square miles or less when estimating the 100-year event:

- Develop a storm surge hydrograph for the 100-year event and a 10-year riverine hydrograph and arrange the time of the riverine hydrograph to peak at the same time as the maximum surge discharge occurring at the mean surge elevation as illustrated in Figure 6-5.
- Develop a storm surge hydrograph for the 10-year event and a 100-year riverine hydrograph and arrange the time of the riverine hydrograph to peak at the same time as the maximum surge discharge occurring at the mean surge elevation as illustrated in Figure 6-5.
- Design for the worst case.

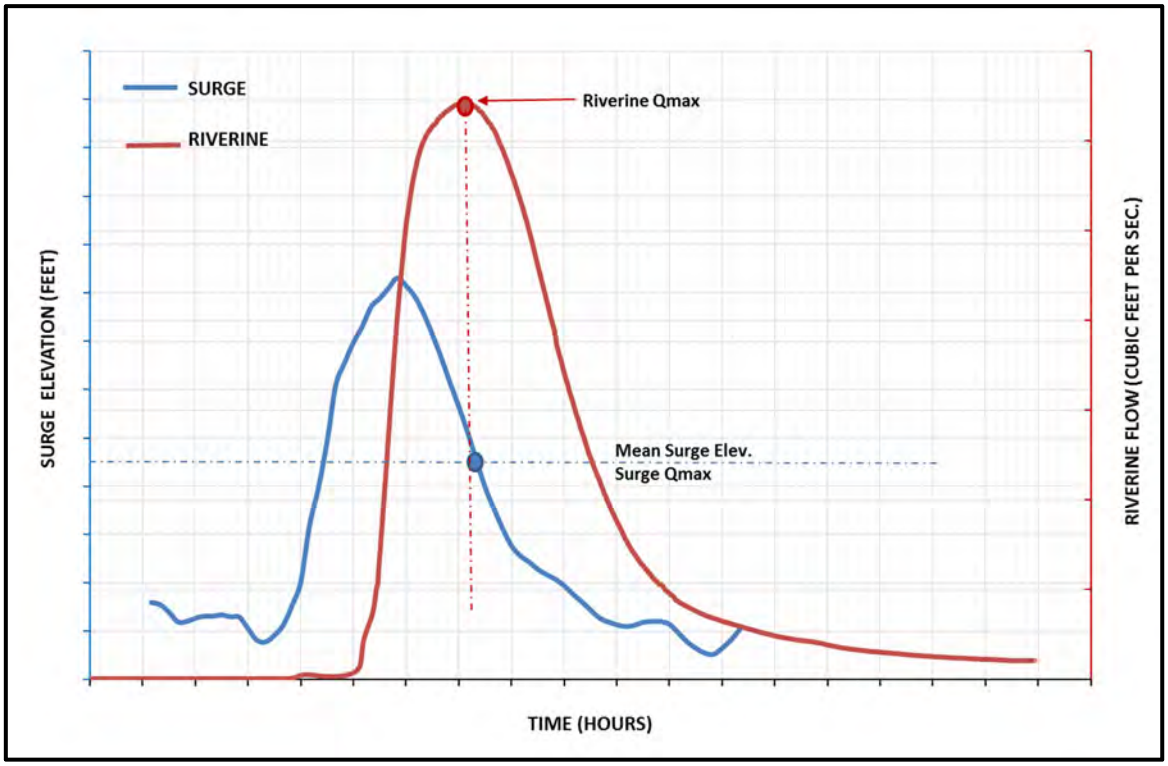

Figure 6-5: Illustration of the riverine peak discharge (Riverine Qmax) occurring at the same time as the maximum storm surge discharge (Surge Qmax)

For watersheds of 25 square miles or less when estimating the 500-year event:

- Develop a storm surge hydrograph for the 500-year event and a 10-year riverine hydrograph and arrange the time of the riverine hydrograph to peak at the same time as the maximum surge discharge occurring at the mean surge elevation as illustrated in Figure 6-5.
- Develop a storm surge hydrograph for the 10-year event and a 500-year riverine hydrograph and arrange the time of the riverine hydrograph to peak at the same time as the maximum surge discharge occurring at the mean surge elevation as illustrated in Figure 6-5.
- Design for the worst case.

For watersheds greater than 25 square miles and the 100-and 500-year events:

- Develop storm surge hydrographs for the 100-year and 500-year events and use a constant 2-year riverine discharge.
- Develop riverine hydrographs for the 100-year and 500-year events and use a 2-year storm surge hydrograph.
- Design for the worst case.

<u>HEC-RAS</u>

For watersheds of 25 square miles or less and 100-year event:

- Estimate the 100-year peak discharge for riverine flow; add the computed discharge for the storm surge for the 10-year event using Equation 6.1. For bridge sites impacted by the Chesapeake Bay, set the tailwater elevation at 1.0 feet (NAVD88). For bridge sites on the Potomac River or the open coast, additional analyses are warranted.
- Estimate a 10-year peak discharge for the riverine flow; add the computed discharge for the storm surge for the 100-year event using Equation 6.1. Set the tailwater elevation at the mean surge elevation for the 100-year storm surge.
- Design for the worst case.

For watersheds of 25 square miles or less and the 500-year event:

- Estimate the 500-year peak discharge for riverine flow; add the computed discharge for the storm surge for the 10-year event using Equation 6.1. For bridge sites impacted by the Chesapeake Bay, set the tailwater elevation at 1.0 feet (NAVD88). For bridge sites on the Potomac River or the open coast, additional analyses are warranted.
- Estimate a 10-year peak discharge for the riverine flow; add the computed discharge for the storm surge for the 500-year event using Equation 6.1. Set

the tailwater elevation at the mean surge elevation for the 500-year storm surge.
- Design for the worst case.

For watersheds greater than 25 square miles and the 100-year event:

- Estimate the 100-year peak discharge for riverine flow; add the computed discharge for the storm surge for the 2-year event using Equation 6.1. For bridge sites impacted by the Chesapeake Bay, set the tailwater elevation at 1.0 feet (NAVD88). For bridge sites on the Potomac River or the open coast, additional analyses are warranted.
- Estimate a 2-year peak discharge for the riverine flow; add the computed discharge for the storm surge for the 100-year event using Equation 6.1. Set the tailwater elevation at the mean surge elevation for the 100-year storm surge.
- Design for the worst case.

For watersheds greater than 25 square miles and the 500-year event:

- Estimate the 500-year peak discharge for riverine flow; add the computed discharge for the storm surge for the 2-year event using Equation 6.1. For bridge sites impacted by the Chesapeake Bay, set the tailwater elevation at 1.0 feet (NAVD88). For bridge sites on the Potomac River or the open coast, additional analyses are warranted.
- Estimate a 2-year peak discharge for the riverine flow; add the computed discharge for the storm surge for the 500-year event using Equation 6.1. Set the tailwater elevation at the mean surge elevation for the 500-year storm surge.
- Design for the worst case.

6.7 ESTIMATION OF THE 2-YEAR TIDAL ELEVATION

The 2-year tidal elevation is often needed at bridge sites to evaluate shear stress and to estimate bridge scour. There are basically two approaches for estimating the 2-year tidal elevation:

- Plot the Stillwater elevations (10-, 50-, 100-, and 500-year) from FEMA Flood Insurance Studies on graph paper and extrapolate down to the 2-year value. This approach tends to underestimate the 2-year value because the 2-year elevation is more influenced by tidal fluctuations whereas the 10- to 500-year elevations developed for Flood Insurance Studies are more storm surge oriented.

- Use frequency estimates at tide stations, for example, NOAA has frequency curves on their web site for the four long-term stations (Baltimore, Annapolis, Cambridge and Solomons Island).

A tidal frequency study conducted by FEMA in 2008 resulted in frequency analyses for several tide stations in the mid-Atlantic Region. The 2-year elevations (NAVD88) were estimated for the following stations in the Chesapeake Bay of Maryland and Virginia:

- Baltimore – 2-year elevation = 2.88 feet,
- Annapolis – 2-year elevation = 2.63 feet,
- Solomons Island – 2-year elevation = 2.46 feet,
- Lewisetta, VA – 2-year elevation = 2.65 feet.

In addition, the 2-year elevation for the Cambridge tide station from the NOAA web site is 2.7 feet. These analyses indicate that the 2-year tide elevation only varies about 0.4 feet across most of the Chesapeake Bay. Therefore, interpolation between tide stations is a reasonable approach for determining the 2-year tide elevation at SHA bridge sites.

Final Comment: The recommended procedures in this chapter do not consider future sea level rise. The T-year storm surge elevations are obtained from FEMA studies or the NOAA web site that do not consider future sea level rise.

There are several sources of future projections of sea level rise for Maryland. The most recent report for Maryland is entitled “Sea Level Rise: Projections for Maryland 2018” (Boesch and others, 2018). The most likely range (66 percent chance) of sea level rise as defined by Boesch and others (2018) is:

- 0.8 to 1.6 feet by 2050, and
- 2.0 to 4.2 feet by 2100 if gas emissions continue to increase.

Boesch and others (2018) also indicate there is about a 5-percent chance that sea level rise will exceed 2.0 feet by 2050 and exceed 5.2 feet by 2100. However, elevation of coastal roads and structures to accommodate sea level rise is not currently part of MDOT SHA operational plans because of prohibitive costs.

6.8 ATTACHMENT 6-1. FREQUENCY AND TIMING OF RIVERINE AND STORM SURGE HYDROGRAPHS

A major issue associated with estimation of peak discharges in tidal reaches is the timing of the riverine and storm surge hydrographs. The timing and frequency of flooding were examined at streamflow and tidal stations. The objective was to find a gaging station on a stream that was draining into an estuary or near an estuary where there was a long-term tide gage.

The following tables are organized by long-term tide gages in the Chesapeake Bay: Cambridge, Solomon Islands, Baltimore and Annapolis.

6.8.1 Cambridge Tide Station

The Cambridge tide station 8571892 is on the Choptank River at Cambridge Maryland and is located just downstream of US 50 (Figure 6-6). Of the four long-term tide stations in the Chesapeake Bay, the Cambridge station has the shortest record from 1943 to 1950 and 1971 to 2015. Several major storm surge events in the 1950s were not observed at the Cambridge station.

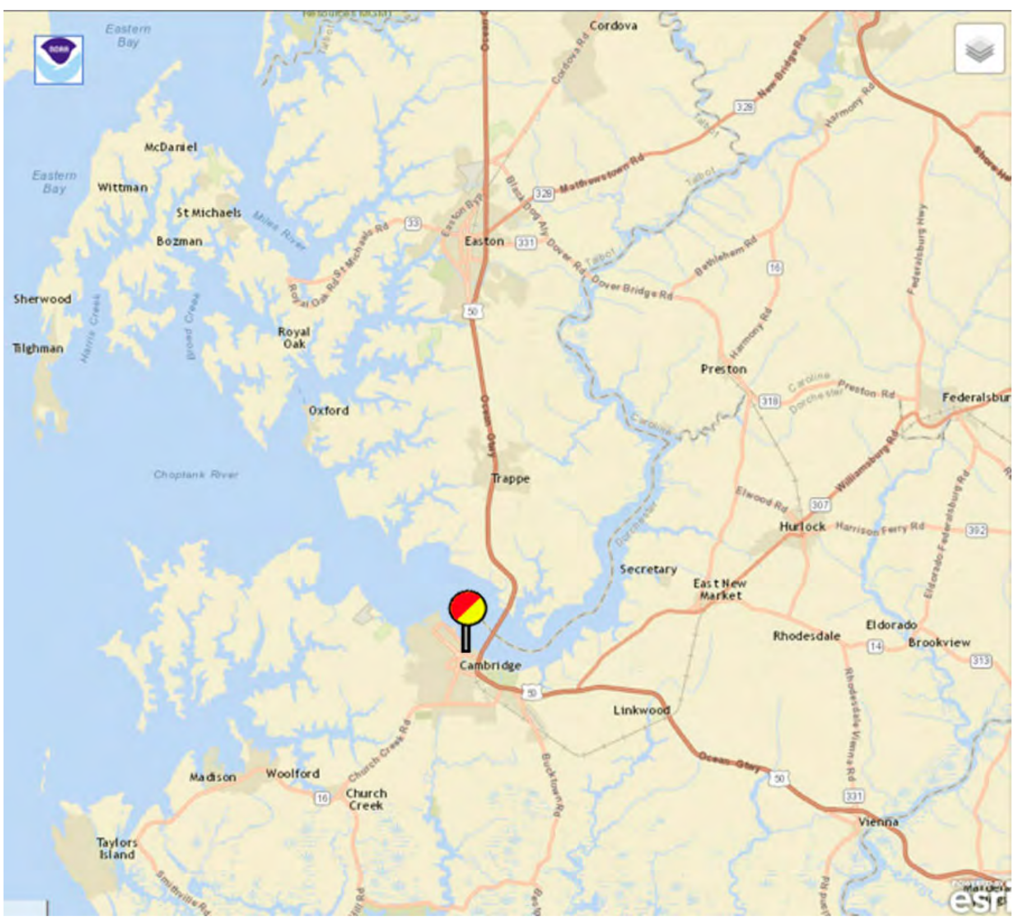

Figure 6-6: Location of the Cambridge tide station

The streamflow gaging station on the Choptank River near Greensboro, MD (station 01491000) with a drainage area of 113.7 square miles is about 50 miles upstream of the tide station. The record at this station is continuous from 1948 to 2015. The time of concentration is 36.9 hours at the gaging station and the travel time from the streamflow gaging station to tidal station is more than 3 days. This means one should add three days to the riverine times given in Table 1 to estimate the timing of the riverine peak discharge at the tidal station.

Table 6-1: Summary of surge and riverine events for the Choptank River (113.7 square miles) and Cambridge tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
September 1979 (Tropical Storm David)	September 6 at 2350 hours 20-year event	Time and frequency of riverine peak unknown*
November 1985	November 5 at 0000 hours 15-year event	Date and time of peak unknown < 3-year event
September 1996	September 7 at 0006 hours 25-year event	September 7 at 0345 hours < 1.25 year event
September 1999 (Hurricane Floyd)	September 16 at 2350 hours < 2-year event	September 17 at 0445 hours 20-year event
September 2003 (Hurricane Isabel)	September 19 at 1100 hours ~100-year event	September 19 at 2100 hours < 2-year event
August 2011 (Hurricane Irene)	August 28 at 1600 hours < 2-year event	August 28 at 1000 hours 50-year event
October 2012 (Hurricane Sandy)	October 29 at 1736 hours 15-year event	October 30 at 1415 hours (8-year event)

* The 1979 annual peak discharge occurred on February 26, 1979 (about a 20-year event) – no data available for the peak discharge on September 6, 1979

The data in Table 6-1indicate that high storm surge and large rainfall-runoff events do NOT tend to occur on the same flooding event on the Choptank River. This might imply that the strong wind events do not have high rainfall. The riverine peak discharges generally occur later at the gaging station than the peak storm surges at the tidal station 50 miles downstream.

6.8.2 Solomons Island Tide Station

The Solomons Island tide station 8577330 is on the Patuxent River at Solomons near the mouth of the river. The tide station is downstream of MD 4 near the mouth of the Patuxent River at Solomons, MD (see Figure 6-7).

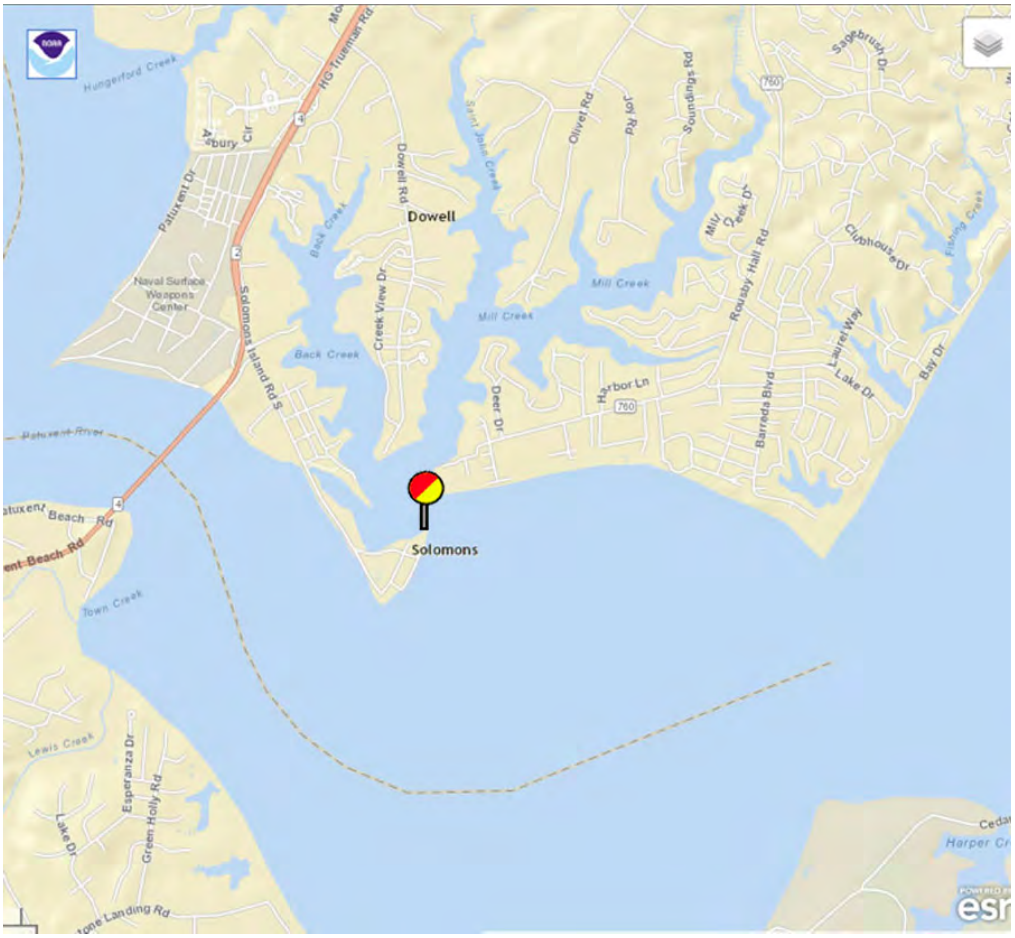

Figure 6-7: Location of the Solomons Island tide station

The streamflow gaging station Patuxent River near Bowie, MD (station 01594440) with a drainage area of 348.9 square miles is 60 miles upstream from the tide station. The record at station 01594440 is from 1978 to 2015 with a historical peak in June 1972 (Hurricane Agnes). The time of concentration at the gaging station is about 21.5 hours and the travel time from the streamflow station to the tide station is about 3 days.

Table 6-2: Summary of surge and riverine events for the Patuxent River (348.9 square miles) and Solomons Island tide station

Date of Flood Event	Time of surge peak/frequency	Time of riverine peak/frequency
June 1972 (Tropical Storm Agnes)	No major storm surge	June 22 > 100-year event
November 1985	November 4 at 2000 hours 15-year event	Date and time of peak unknown < 1.1-year event
September 1996 (Tropical Storm Fran)	September 6 at 2206 hours 20-year event	September 7 at 2100 hours ~1.1 year event
September 2003 (Hurricane Isabel)	September 19 at 1000 hours ~100-year event	September 19 at 2030 hours 2-year event
September 2006	September 1 at 1924 hours 30-year event	September 2 at 2145 hours ~1.1-year event
May 2008	May 11 at 2312 hours May 12 at 0836 hours **Two 10-year events**	May 13 at 0100 hours 3-year event
September 2011 (Tropical Storm Lee)	September 7 at 2342 hours < 2-year event	September 8 at 1200 hours 25-year event

The data in Table 6-2 indicate that high storm surge and large rainfall-runoff events do NOT tend to occur on the same flooding event on the Patuxent River. Most of these events are the same as for the Choptank River so a similar conclusion is that the large storm surge events do not have the accompanying high rainfall. For the large storm surge events (10-year event or larger), the riverine frequency was a 3-year event or less. The riverine peak discharges occur later than at the tide station 60 miles downstream.

6.8.3 Baltimore Tide Station

The Baltimore tide station 8574680 is in the Patapsco River estuary near Fort McHenry and just north of I-95 (see Figure 6-8).

The Baltimore tide station is in the Patapsco River estuary; there is a gaging station 28 miles upstream of the mouth of the river at Hollofield, MD where the drainage area is 284.7 square miles. The available record for the Patapsco River station 01589000 is from 1945 to 2015 with a historic peak in August 1933. A few other smaller drainage areas also drain into the Patapsco River or nearby Back River estuary from within Baltimore City; those stations were also investigated for timing of the flood events.

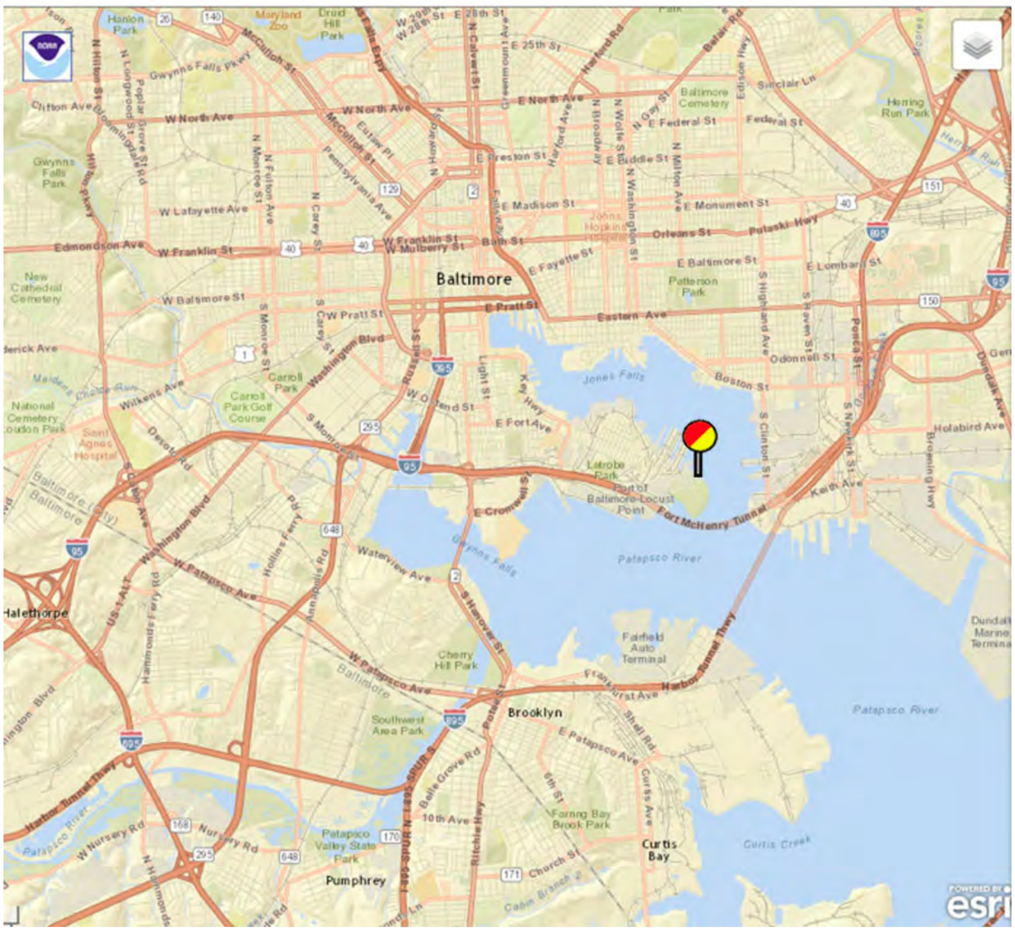

Figure 6-8: Location of the Baltimore tide station

Table 6-3: Summary of surge and riverine events for the Patapsco River (284.7 square miles) and the Baltimore tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
August 1933	August 23 at 2300 hours 80-year event	August 1933 (day unknown) 25-year event
November 1950	November 25 at 2100 hours 10-year event	Date and time of peak unknown <1.25-year event
October 1954 (Hurricane Hazel)	October 15 at 1900 hours 10-year event	Date and time of peak unknown <2-year event
August 1955 (Tropical Storm Connie)	August 13 at 0800 hours 40-year event	August 13 (time unknown) 2-year event
June 1972 (Tropical Storm Agnes)	June 23 at 0600 hours <2-year event	June 22 >200-year event
September 1975 (Hurricane Eloise)	September 26 at 2300 hours <2-year event	September 26 50-year event
September 1979 (Tropical Storm David)	September 6 at 0700 hours 15-year event	September 6 (time unknown) 2-year event
November 1985	November 4 at 2200 hours 15-year event	Date and time of peak unknown <1.25-year event
September 2003 (Hurricane Isabel)	September 19 at 0806 hours 100-year event	September 19 at 0345 hours <1.25-year event
September 2011 (Tropical Storm Lee)	September 6 at 0206 hours 2-year event	September 7 at 1322 hours 5-year event

The data in Table 6-3 indicate that high storm surge and large rainfall-runoff events do NOT tend to occur on the same flooding event on the Patapsco River. The only exception is the August 1933 flood event where the storm surge was an 80-year event and the riverine peak discharge a 25-year event. For the data before 1990, only the annual maximum peak flows are readily available with just the date (day) of the flood known. In many cases, the annual maximum peak discharge and the annual maximum storm surge do not occur on the same event. The frequency of the riverine peak discharge for the given storm surge event was estimated as being less than a given return period by using return period of the annual maximum peak discharge.

As shown in Table 6-3 (highlighted entries), the riverine flooding events on August 13, 1955 and September 6, 1979 occurred on the same day as the storm surge peak although the riverine events were relatively minor (2-year events). This may be related to the fact that the Patapsco River watershed is in the Piedmont region and the time of concentration may be less than Coastal Plain streams of the same watershed size. Or the rainfall may have preceded the storm surge for these two storm events.

There is a streamflow gaging station on Gwynns Falls at Washington Blvd at Baltimore, MD (01589352) where the drainage area is 63.6 square miles. The gaging station is 1.6 miles upstream of the estuary. The record for the Gwynns Falls streamflow station is from 1999 to 2015.

Table 6-4: Summary of surge and riverine events for Gwynns Falls (63.6 square miles) and Baltimore tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
August 1999	No major storm surge	August 26 25-year event
September 2003 (Hurricane Isabel)	September 19 at 0806 hours 100-year event	September 18 at 2335 hours < 1.25-year event
September 2011 (Tropical Storm Lee)	September 6 at 0206 hours 2-year event	September 7 at 1405 hours 8-year event

The peak of record at the Gwynns Falls station occurred in August 1999 (25-year event) but there was no significant storm surge event. Other large floods occurred in April, June and July when there were no major storm surge events. Due to the short record at the Gwynns Falls gaging station, there are not many surge events to evaluate.

There is a streamflow gaging station on Moores Run at Radecke Ave at Baltimore, MD where the drainage area is 3.52 square miles. The record available for Moores Run is from 1997 to 2015. The gaging station is 2 miles upstream of the mouth and actually drains into the Back River estuary, which should peak about the same time as the Patapsco River estuary.

Table 6-5: Summary of surge and riverine events for Moores Run (3.52 square miles) and the Baltimore tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
September 1996	September 6 at 2118 hours 10-year event	September 6 at 2155 hours 1.25-year event
September 2003 (Hurricane Isabel)	September 19 at 0806 hours 100-year event	September 19 at 0009 hours < 1.25-year event
September 2011 (Tropical Storm Lee)	September 6 at 0206 hours 2-year event	September 7 at 2353 hours 4-year event

The timing of the Moore's Run peak riverine and surge events are closer than for the larger riverine stations (except for Tropical Storm Lee). As with the other sites, the major storm surge events did not produce large runoff events. Due to the short record at the Moores Run gaging station, there are not many surge events to evaluate.

6.8.4 Annapolis Tide Station

The Annapolis tide station 8575512 is located at the Naval Academy in Annapolis, MD as shown in Figure 6-9. This is in the Severn River estuary. There are not many gaged streams that drain into the Severn River estuary. One small streamflow gaging station that does drain into the Severn River estuary is South Fork Jabez Branch at Millersville, MD where the drainage area is 1 square mile. The record length is from 1997 to 2015.

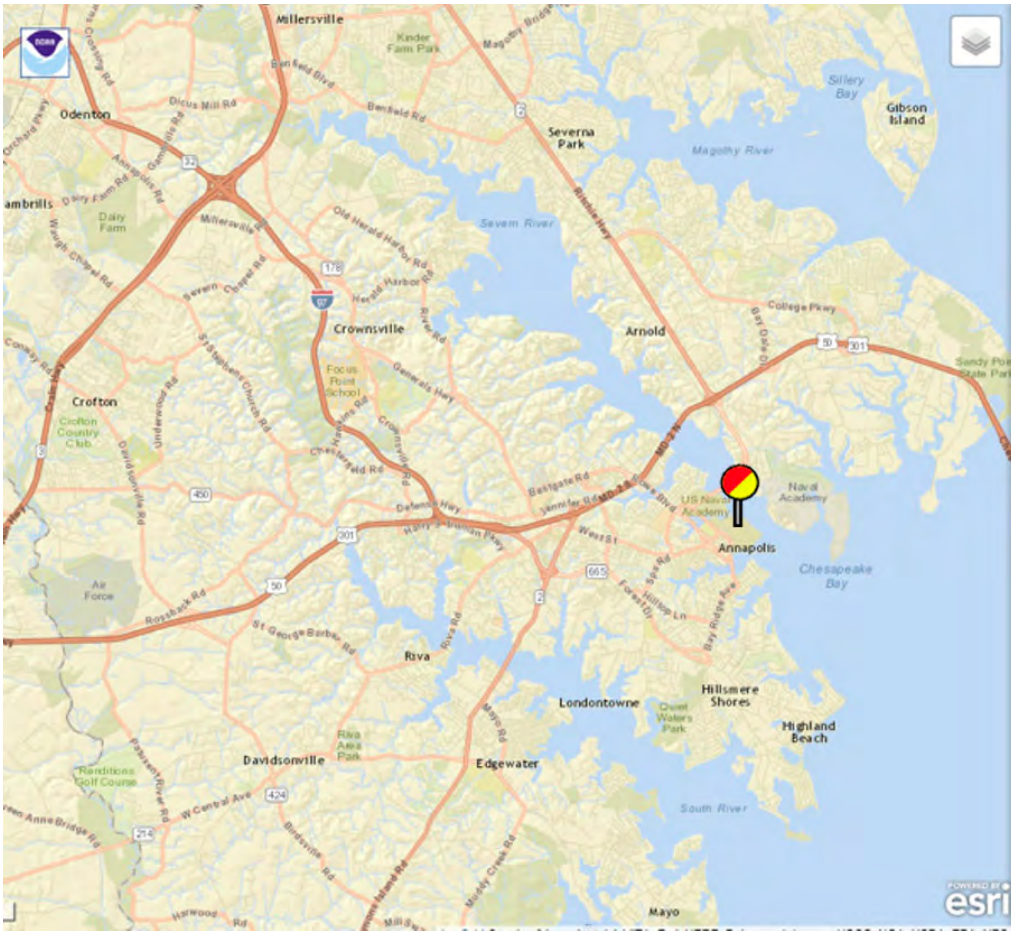

Figure 6-9: Location of the Annapolis tide station

Table 6-6: Summary of surge and riverine events for SF Jabez Branch (1.0 square miles) and the Annapolis tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
September 2003 (Hurricane Isabel)	September 19 at 0754 hours 100-year event	September 18 at 2040 hours 20-year event
September 2011 (Tropical Storm Lee)	September 8 at 0218 hours <2-year event	September 7 at 2310 hours 100-year event

These are the same events as used at other stations but the results are a little different. The riverine event for Hurricane Isabel was a 20-year event and actually peaked before the peak surge event. Tropical Storm Lee did not produce a major storm surge but did produce a major runoff event. Due to the short record for the SF Jabez Branch gaging station, there are not many surge events to evaluate.

Two discontinued gaging stations just west of Annapolis drain into the South River estuary south of Annapolis:

- North River near Annapolis (station 01590000) – drainage area = 8.93 square miles, period of record 1932-1973, and
- Bacon Ridge Branch at Chesterfield (station 01590500 – drainage area = 6.97 square miles, period of record 1944-52, 1965-90.

Data for surge and riverine events are summarized in Tables 6-7 and 6-8 for these two stations to supplement the limited data in Table 6-6 and to provide data for storm surge events before 1990.

Table 6-7: Summary of surge and riverine events for North River (8.93 square miles) and the Annapolis tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
August 1933	August 23 at 2100 hours 50-year event	August 23 (time unknown) 1.5-year event
November 1950	November 25 at 1700 hours 5-year event	Date and time unknown 1.5-year event
August 1955 (Hurricane Diane)	August 13 at 0700 hours 12-year event	August 13 (time unknown) 20-year event

Table 6-8: Summary of surge and riverine events for Bacon Ridge Branch (6.97 square miles) and the Annapolis tide station

Date of flood event	Time of surge peak/frequency	Time of riverine peak/frequency
November 1950	November 25 at 1700 hours 5-year event	Date and time unknown 3-year event
September 1979	September 6 at 0500 hours 12-year event	September 6 (time unknown) 15-year event
November 1985	November 4 at 2200 hours 12-year event	Date and time unknown < 1.25-year event

The highlighted data in Tables 6-7 and 6-8 indicate that the riverine flood for August 1955 was a 20-year event for the North River and the riverine flood for September 6, 1979 was a 15-year event for Bacon Ridge Branch while the tidal events were both 12-year events. This is the first evidence we have that major storm surges and riverine peak discharges can occur approximately at the same time. The tidal and riverine peaks occurred on the same day although the timing of the riverine flood peaks are unknown (only the day [date] of the annual peak discharge is known). Both riverine stations have drainage areas less than 10 square miles.

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CHAPTER SEVEN

7 Recommendations for Future Research

7.1 INTRODUCTION

In spite of the volume of research reported in professional literature, knowledge of many aspects of applied engineering hydrology is lacking. In this section, some aspects of design hydrology that require additional research are identified along with the potential benefits that could result from better knowledge about these topics. Research on the topics below would possibly enable better decisions to be made with respect to the use of hydrologic methods in hydrologic design.

7.2 CLIMATE CHANGE

Climate change is an emerging issue that has significant potential impacts on highway infrastructure planning and design. Climate change is anticipated to result in rising baseline water levels in tidal-influenced areas of Maryland, with commensurate shifts in the tidal range in this zone. Additionally, precipitation intensity-duration-frequency (IDF) relationships, currently quantified by the NOAA Atlas 14 dataset, are expected to shift, and most likely increase, as climate change becomes more pronounced.

The need for this research is further reinforced by the 2020 proposed change by the Maryland Coast Smart Council, which indicates all new state structures in coastal areas need to be built three feet above the 100-year base flood elevation. Existing roadways are excluded but all new roadways must be constructed three feet above the 100-year base flood elevation. In addition, MDOT SHA needs to assess the impacts on existing structures and evacuation routes.

In the tidally-influenced zone, significant areas will simply be lost to inundation while other areas may be protected from tidal encroachment. Research on strategies to plan for and manage infrastructure in both situations is needed. In many cases, the costs of elevating MDOT SHA roads as instructed would result in prohibitive costs, and may also have unintended negative consequences on flows and sediment transport experienced during normal and amplified tidal conditions.

In riverine systems, MDOT SHA sponsored research suggests varying degrees of change in precipitation IDF for the mid-21st century (Brubaker and others, 2017). This research used simulated 3-hour timestep precipitation forecasts from the North American Regional Climate Change Assessment Program as input to precipitation frequency analysis software. Output from the frequency analysis is used to create operational IDF estimates covering the GISHydro domain. Research is needed to determine the best approach for employing these estimates and their associated uncertainty into future designs. As many

MDOT SHA structures have planned operational lifetimes of 50 years and more, there exists a great need for similar estimates of precipitation IDF corresponding to the end of the 21st century. Although most of the CMIP3 and CMIP5 research efforts (CMIP, 2016) produce *daily* precipitation depths for the end of the 21st century, these simulated data may not be sufficient to create IDF estimates for shorter durations (e.g. 3, 6, and 12 hours). The arrival of such higher temporal resolution precipitation forecasts in the near future is likely. Uncertainties associated with these longer-range climate forecasts will be greater than for mid-century forecasts. The most appropriate planning and design strategy to accommodate these uncertainties also merits further research.

Research by the Transportation Research Board (TRB) National Cooperative Highway Research Program (NCHRP) Project 15-61 produced a Design Practices report on "Applying Climate Change Information to Hydrologic and Coastal Design of Transportation Infrastructure," which is available from the TRB web site at https://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=4046 (Kilgore and others, 2019). This research provides guidance in incorporating climate change into riverine hydrology and coastal analyses with respect to selecting:

- climate gas emission scenarios,
- high-resolution climate projections and data sets,
- global climate models, and
- projections of sea level rise.

In 2021 a consortium including Rand Corporation, Carnegie Mellon University and the Northeastern Regional Climate Center at Cornell University developed a web tool for estimating future precipitation for the Chesapeake Bay Watershed (https://midatlantic-idf.rcc-acis.org/). This work was funded by NOAA through their Mid-Atlantic Regional Integrated Sciences and Assessments (MARISA) program. The development of this web site and the data used in the analysis are described in a Rand Corporation report, which is available at (https://www.rand.org/content/dam/rand/pubs/tools/TLA1300/TLA1365-1/RAND_TLA1365-1.pdf).

The MARISA web tool is readily available and provides change ratios for all counties in the Chesapeake Bay watershed (including all counties in Maryland). These ratios are used to increase precipitation depths from NOAA Atlas 14, Volume 2. Specifically, the MARISA web site provides ratios for:

- The 2-, 5-, 10-, 25-, 50- and 100-year precipitation depths,
- Emission scenarios for RCP 4.5 (low) and RCP 8.5 (high),
- Two time periods: 2020-2070 and 2050-2100, andRatios for the 10th, 25th, median, 75th, and 90th percentiles (exhibiting the uncertainty in the ratios).

As described in Chapter 1, the Hydrology Panel recommended that MDOT-SHA use of the median ratio for the high emission scenario RCP 8.5 for the time period 2050-2100 to estimate the final discharges for selected projects in Maryland. The median ratios were incorporated into GISHydro for easy use for project design in Maryland. The ratios are assumed applicable for all storm durations. The use of the MARISA data is an interim approach until more detailed data are available.

Climate change research is rapidly evolving with time and improved GCMs are being developed that will produce more accurate estimates of future precipitation. As these improved models and data sets are published, the guidance for use of future precipitation in Maryland will change.

7.3 TIME OF CONCENTRATION

The time of concentration is a principal input to most hydrologic design methods. The velocity method generally uses Manning's equation to compute the velocity. The NRCS WinTR-55 kinematic wave equation is frequently applied for computing travel time for shallow sheet flow.

When the velocity is computed using Manning's equation for channel flow, estimates of the roughness coefficient, the hydraulic radius, and the slope are required. Each of these inputs is important, and error or uncertainty in the inputs reduces the accuracy of estimates of the time of concentration. Roughness varies considerably with river stage. Since the river stage for a design discharge is related to the return period of the flow, the roughness used to compute a velocity should depend on the cross section that reflects the discharge rate for the design return period. Research on the effects of depth dependent Manning roughness coefficients on time of concentration is needed. If only the roughness of bankfull flow is used when the design return period would suggest out-of bank flow conditions, the estimated velocity and, therefore, the computed T_c could be significantly different than the most appropriate value.

An estimated velocity is sensitive to the hydraulic radius. The hydraulic radius is a function of the stage of flow, which as indicated above depends on the return period. The hydraulic radius also depends on the shape of the cross section, which can vary considerably along a channel. Therefore, it is necessary to understand the sensitivity of computed velocities when using a single supposedly representative hydraulic radius for a stream in which the cross section changes noticeably over the channel length. Research on the effects of variation in both the return period and cross-section characteristics as they relate to the hydraulic radius could improve the estimation of T_c.

If a representative cross section is difficult to select because of excessive variation in cross section characteristics throughout a channel reach, the Fish and Wildlife Service (FWS) (2002) equations can be used to compute the cross-section characteristics. While preliminary analyses suggest that these equations provide reasonable estimates in Maryland, more analyses of these equations using data from Maryland are needed.

The slope of a channel section is computed using the elevation drop and the reach length. Generally, the variation in reach length for different scale maps is considerably greater than variation in the elevation drop. Where the reach length is estimated from a map or digital data, the accuracy of the length estimate will influence the accuracy of the computed slope. If a large map scale or digital data are used and the scale of the data prevents accurate depiction of the meanders, then the overall length could be underestimated, which leads to an overestimate of the slope and velocity and an underestimate of the T_c. The significance of the error in length needs investigation.

Empirical models are possible alternatives to the velocity method. While a number of studies indicate that some empirical models provide reasonable estimates of T_c, the accuracy of empirical models for use in Maryland has not been evaluated. Useful research could result from using times of concentration obtained from rainfall-runoff data to assess the accuracy of empirical equations. As additional research, T_c values estimated from rainfall-runoff data could be used with measured physiographic data to calibrate empirical equations for different regions of Maryland and develop a synthetic hydrograph in conjunction with these times.

Another alternative to the velocity method is to define the time of concentration from observed rainfall hyetographs and discharge hydrographs. Using this approach, the time of concentration is defined as the time from the ending of rainfall excess to the first inflection point on the recession of the discharge hydrograph. The limitation to this approach is finding suitable observed rainfall hyetographs that are consistent with and representative of discharge hydrographs at gaged watersheds. Regression analysis can be used to relate the computed time of concentration to watershed and climatic characteristics for the gaged watershed. Estimates of the time of concentration can be made at ungaged locations by simply determining the watershed and climatic characteristics and applying the regression equation.

An alternative procedure to determine T_c from rainfall-runoff data is first to determine the event runoff curve number based on rainfall and runoff volumes. The next step is to set up a WinTR-20 data set with the watershed drainage area, curve number, and event rainfall table and try different T_c's until the simulated hydrographs are close as possible to the actual hydrographs. The dimensionless unit hydrograph may also be adjusted, if needed, to provide a better match of simulated and actual hydrographs.

A regression equation for estimating time of concentration for Maryland streams is described in Appendix 6 (Thomas and others, 2000). The regression approach is easy to use and provides reproducible estimates, but the time of concentration is often in excess of that determined by the velocity method. The computed times of concentration and the resultant regression equation given in Appendix 6 were generally based on runoff events less than the 2-year flood. Research is needed to determine if the time of concentration varies significantly with the magnitude and frequency of peak discharge.

A recent research project at the University of Maryland (UMD) used NEXRAD precipitation data and observed flood hydrographs to estimate T_c for multiple storm events at gaging stations in Maryland (Brubaker and others, 2021). The use of NEXRAD data overcomes the issue with lack of suitable observed hyetographs but the length of record for archived NEXRAD data is less than 10 years reducing the number of large runoff events available for analysis. The UMD project estimated the time of concentration for 54 gaged watersheds across Maryland using NEXRAD precipitation and streamflow at USGS gaging stations for a total of 108 storm events. However, this project did not develop more accurate procedures for estimating time of concentration for ungaged watersheds than the regression equation given in Appendix 6 of this report (Thomas and others, 2000). Hence, the regression equation in Appendix 6 will continue to be used to evaluate time of concentration estimates based on the NRCS travel time method.

7.4 UNIT HYDROGRAPH PEAK RATE FACTORS

While some research on the peak rate factor for the NRCS unit hydrograph has been completed, additional work is still needed. Most importantly, peak rate factors need to be estimated from hydrograph data, not just peak discharge data. It is important to estimate the peak rate factor from unit hydrographs computed from measured hyetographs and hydrographs. This research could show the geographic variation of peak rate factors, as well as the extent of their uncertainty. The Hydrology Panel performed research in 2018 on estimating peak rate factors using observed rainfall and runoff data for gaged streams in the Appalachian Plateau Region, small Agricultural Research Service (ARS) watersheds near College Park and Hagerstown in the Piedmont-Blue Ridge Region, and Eastern Coastal Plain Region. The results are summarized as:

- Appalachian Plateau Region: five watersheds with a total of 37 events, the average peak rate factor (PRF) for each watershed ranged from 171 to 381,
- Small ARS watersheds: five watersheds with a total of 13 events, the range of PRF for individual events was 150 to 600 with an average of 300, and
- Eastern Coastal Plain Region: five watersheds with a total of 12 events, the range of PRF for individual events was from 100 to 450 with an average of 240.

There was a large range in peak rate factors across storm events at a given gaging station and among gaging stations. For this reason, no attempt was made to relate the peak rate factors to watershed characteristics and the average value for the PRF was not considered reliable enough to revise existing guidance. One major reason for the large variation in PRFs was that the available rainfall data were generally several miles from the gaged watershed. The data indicated that the 484 PRF used in the Appalachian Plateau and Piedmont-Blue Ridge Regions may be too large. On the other hand, the 284 PRF used in the Eastern Coastal Plain Region was reasonably close to the 240 average value based on 12 storm events at five gaging stations and no change was recommended.

The results of the Hydrology Panel investigations were not definitive in estimating the peak rate factor by region or based on watershed characteristics. No recommendations on PRFs values could be made based on analyzing 62 rainfall-runoff events at 15 gaged watersheds across Maryland.

The Brubaker and others (2021) study discussed above developed PRFs for 54 gaged watersheds across Maryland using NEXRAD precipitation data and streamflow at USGS gaging stations for a total of 108 storm events. The PRFs varied a lot across different runoff events for a given station and among gaging stations. No procedure was developed for estimating PRFs for a given region or based on watershed characteristics.

Additionally, peak rate factors computed from unit hydrographs obtained from rainfall-runoff data could be compared to the peak rate factors computed using geomorphic unit hydrographs derived from time-area curves. This would enable geomorphic unit hydrographs to be combined with hyetograph – hydrograph generated unit hydrographs in selecting regional peak rate factors. Improving estimates of the peak rate factor for the NRCS unit hydrograph for use in Maryland watersheds will improve design accuracy.

7.5 PEAK DISCHARGE TRANSPOSITION

While various forms of peak discharge transposition from gaged to ungaged locations are widely used, surprisingly little understanding of their accuracy exists. The results provided by McCuen and Levy (1999) for Pennsylvania, Virginia, and Maryland appear to be the only empirical assessment of the transposition procedure. The PA/VA/MD data base is sparse; therefore, these results need to be verified for other data sets. The USGS method of transposing peak discharges to ungaged locations within 50 percent of the drainage area of the gaging station is based primarily on engineering judgment. The variation of the weighting functions, both of the area-ratio and USGS methods, needs to be assessed over a broader range of data. The structures of the weighting functions need to be specifically evaluated.

Research on alternative transposition methods should be performed to assess the accuracy of the methods. The results would increase the confidence that could be placed in their use. Without this additional research, transposition methods should be used with caution.

7.6 TRANSFORMATION OF ZONING-MAP INFORMATION INTO HYDROLOGIC MODEL INPUT

Most designs in Maryland require assessment for ultimate-development watershed conditions. The input to hydrologic models for ultimate-development conditions often requires obtaining information from zoning maps. Zoning maps delineate areas assigned to different land use categories. However, these categories are not consistent across political boundaries and, more importantly, a systematic method for transforming the land use categories into inputs for hydrologic models is lacking. For example, different jurisdictions use different notations for the various densities of residential development, and measures of the corresponding impervious area, which is important input to hydrologic design methods, are not provided or are ambiguously assessed.

While it would be useful to have standard zoning classifications for all jurisdictions in Maryland, this is unlikely to happen. Even this would not eliminate the need for a procedure for transforming zoning map classifications into input parameters for hydrologic design methods. Research could provide a procedure for estimating model inputs such as impervious areas and curve numbers from zoning classifications. This would improve the reproducibility of designs.

7.7 ADJUSTING WINTR-20 USING REGRESSION EQUATION ESTIMATES

When applying the WinTR-20 adjustment procedure using the prediction limit on the regression equation, the regression estimate plus one standard error of prediction window is recommended herein. This value is based on the judgment and hydrologic experience of the Panel members.

Research needs to be undertaken on the most accurate and appropriate confidence level, which will probably vary with geographic region, return period, drainage area and project. Evaluation of plus and minus prediction limits should also be evaluated. A systematic research effort should provide confidence levels that can make WinTR-20 adjustments more accurate.

7.8 THE DESIGN STORM

Before NOAA Atlas 14 was published, the traditional approach followed in Maryland was to use the NRCS Type II 24-hour duration storm as the input to the WinTR-20. The depth of precipitation was selected from the appropriate precipitation depth frequency maps. The access of precipitation data and use of the data to develop site-specific rainfall distributions has changed with the release of WinTR-20 version 3.10. NOAA Atlas 14 precipitation data may be downloaded and saved as a text file from the NOAA web site for a location selected by the user. This text file may then be imported to WinTR-20. Rainfall distributions are developed for each return period based on the ratio of rainfall at durations of 5 minutes to 12 hours to the 24-hour rainfall. If WINTR-20 is implemented within GISHydro, the NOAA Atlas 14 precipitation depth and distribution data are readily available.

After application of WinTR-20, if the WinTR-20 over-predicts peak discharge, a major portion of the problem may originate from the severity of this design storm input. Twenty-four hours may be too long and the storm distribution may not be appropriate for all parts of Maryland. The 24-hour duration coupled with the NRCS storm distribution may be especially inappropriate for Western Maryland where gaged discharges tend to be much lower than those estimated by the WinTR-20 model. More research is needed to finalize a synthetic storm structure and duration to be used for specific frequencies and locations.

A flood hydrograph study for the State of Maryland by the U.S. Geological Survey (Dillow, 1998) identified 278 rainfall-runoff events at 81 gaging stations throughout Maryland. These rainfall-runoff events were used to develop dimensionless hydrographs for three hydrologic regions in Maryland and to estimate the average basin lag time for each of the 81 gaging stations.

These rainfall-runoff data were used to investigate the duration of rainfall to provide insight into whether the 24-hour duration storm used with the WinTR-20 model was reasonable. Rainfall events were analyzed for 10 gaging stations where one of the runoff events exceeded a 10-year event. The time from the beginning of rainfall to the ending of rainfall, including intermittent periods of rainfall, was tabulated. The longest duration storms tended to be tropical depressions such as the November 1985 Hurricane Juan that caused severe flooding in Western Maryland or the June 1972 Hurricane Agnes that caused extensive flooding across central Maryland and Delaware. The duration of these tropical depressions ranged from 14 to 24 hours.

Spring and summer rainfall events were generally less than 10 hours in duration. A few spring or summer rainfall events in Western Maryland exceeded 10 hours in duration but the rainfall was intermittent with long periods of no rainfall. Based on a limited sample of events, it appears that rainfall events in Western Maryland are less intense than in Central and Eastern Maryland and this may contribute to the lower peak discharges per square mile that have been observed in this region.

Additional research is needed to determine the most appropriate storm duration and structure for use with WinTR-20.

7.9 GEOMORPHIC UNIT HYDROGRAPHS

Standard unit hydrograph shapes are used in hydrologic design. For Maryland, the NRCS 484-UHG and 284-UHG are used. Research suggests that the most appropriate unit hydrograph for a watershed is one that is based on the geomorphic characteristics of the watershed. Recent research in the professional literature suggests that time-area based unit hydrographs accurately regenerate observed storm runoffs. With the capability of GIS to generate watershed boundaries and internal drainage structures from digital terrain data, it is feasible to use GIS to develop a unit hydrograph that is unique to a watershed, thus improving the accuracy of design hydrographs.

A study of Maryland watersheds should be undertaken to evaluate the accuracy of geomorphic unit hydrographs. Predictions of storm runoff based on these should be compared with predictions based on the 484-UHG and 284-UHG. Both the NRCS and geomorphic unit hydrographs could be compared with measured runoff events in Maryland to assess their accuracy.

7.10 STATISTICAL ALTERNATIVES

The Maryland Department of Planning (MDP) data were used to estimate land use conditions such as impervious area. The MDP approach is to assign a percentage of impervious area to various land use categories. For example, Institutional Lands are assigned an impervious area of 50 percent but there is considerable variation in impervious area for this land use category. Impervious area as estimated from the MDP data was statistically significant in estimating flood discharges for urban watersheds in the Western Coastal Plains and the Piedmont-Blue Ridge Regions but this variable did not explain as much variability as anticipated. The impervious area near the midpoint of the gaging station record was used to estimate impervious area for developing the regression equations. For some gaging stations, a more homogeneous period of record was used when the impervious area was not changing so dramatically and there was not a significant upward trend in annual peak flows. A time varying mean approach in the frequency analysis was used for selected gaging stations to account for changing land use conditions (Kilgore and others, 2016; Kilgore and others, 2019). For future analyses, a more detailed approach should be developed for determining a homogeneous period for frequency analysis or for adjusting the annual peak data to existing conditions. Improved measures of urbanization would likely provide more accurate regression equations in the future.

Many of the gaging stations on small rural watersheds (less than 10 square miles) were discontinued in the late 1970s resulting in short periods of record with large floods for the small watersheds in Maryland. For the 2020 revisions to the Piedmont-Blue Ridge Region equations, flood frequency estimates were adjusted for 13 small rural gaging stations to account for bias using the procedure developed by Carpenter (1980) and described in Appendix 3. Graphical record extension techniques were used at selected short-term stations in developing the regression equations. There were many short-record stations in Maryland for which no adjustment was made. For future analyses, a more detailed or systematic approach should be used for record extension techniques to obtain improved estimates of flood discharges for short-record stations in Maryland. Improving the data base of small watershed data would provide more accurate regression equations in the future.

Finally, only stations primarily in Maryland were used in developing the Fixed Region regression equations in Appendix 3 because the required land use data were not available in neighboring states. The exception was the inclusion of 17 gaging stations in Delaware where consistent land use data were available for 2002. For future analyses, comparable land use data should be investigated for nearby states in order to increase the number of gaging stations used in the regression analysis.

7.11 DEVELOPMENT OF A MODEL FOR USE ON MIXED URBAN-RURAL WATERSHEDS

An increasing number of watersheds of concern to the SHA are going to have some portions that are highly urbanized and other areas that are in agricultural or forest land cover. The WinTR-20 model can adjust the structure of the runoff flow paths to reflect man-made drainage, and urban curve number categories can define the land covers. However, the WinTR-20 was not designed for this type of watershed. The dimensionless UHG, as one example, was derived from rural watershed data.

The SHA needs a deterministic model that can handle a rational partitioning of the watershed into urban and rural segments. Such a model would not have to be a totally original system. It could be a combination of two models, one of which would be implemented on the urbanized portions and the other on the rural portions. The urban component might draw on the EPA Storm Water Management Model as a base and the rural component could be a revision of the WinTR-20. The mechanics of this approach could be done today. However, a significant level of research would have to be conducted to put the components into a package that would give consistent results and would be relatively easy to run.

7.12 MUSKINGUM-CUNGE CHANNEL ROUTING PROCEDURE

A research project similar to that of Ragan and Pfefferkorn (1992) is needed to indicate the changes in the routed hydrograph caused by different decisions on the input parameters to the Muskingum-Cunge method. The project will need to provide more guidance to the user on the selection of the input parameters than is currently available. The project should be based on actual stream gage data. More research is needed in selecting a representative cross section location and developing a representative cross section based on a number of cross sections within a routing reach.

7.13 RELATIONSHIP OF PERCENT IMPERVIOUS AND LAND USE

The current guidelines used by SHA for percent impervious and land use are taken from WinTR-55. There are many other sources for this relation and many are related to the technique used to determine the land use. Aerial photograph analysis has provided additional sources for this relationship. A research effort is needed to provide additional guidelines for determining percent impervious for various land uses. This would provide the SHA a better idea of the curve number that should be used with the range of normal land use categories.

7.14 CONCURRENT RIVERINE AND COASTAL FLOODING

The recommendations for combining storm surges and riverine discharges in Chapter 6 of this report were based on analyzing concurrent storm events at four tidal stations and the associated riverine gaging stations plus engineering judgement and experience. More detailed research is needed to develop improved guidelines for selecting concurrent return periods for storm surges and riverine peak discharges.

7.15 RECOMMENDATIONS FOR UPDATING THE HYDROLOGY PANEL REPORT

The recommendations provided in this report are based on a combination of hydrologic judgment, existing reports and methodologies, and limited testing and evaluations of new concepts. The centerpiece of the recommendations is to quasi-calibrate the WinTR-20 deterministic watershed model using the regional regression equations where these equations are applicable. This approach appears to be a logical approach for improving estimates of flood discharges for Maryland streams and for combining the strengths of WinTR-20 modeling and regional regression equations. As more experience is gained with this approach and as technology changes, this approach may need to be revised. Similarly, as new research is completed, new technology should be incorporated into this report.

This report should be considered a dynamic document with updates as needed. SHA and MDE should jointly pursue the recommended research to improve the estimation of flood discharges for Maryland streams. To date five editions of the Hydrology Panel report have been developed in 2001, 2006, 2010, 2016 and 2020 to incorporate new data and research.

7.16 SUMMARY OF THE MAJOR RESEARCH ITEMS

In summary, there are many areas of hydrology that require additional research if we are to improve our confidence in the modeling process. It is imperative that a continuing, well-conceived and adequately funded research program be implemented to address a number of problems, especially:

Improving the structure and duration of the design storms,

Developing improved methods for estimating T_c for rural and urban watersheds and determining if the T_c varies significantly with the magnitude and frequency of peak discharge,

Using the time-area curve available from the digital terrain data to generate geomorphic unit hydrographs that are unique for the watershed being modeled,

Continuing research on the regionalized peak rate factors to be used with the NRCS dimensionless unit hydrograph,

Refining the transposition procedures of peak discharges from a gaging station to an ungaged location, Providing improved statistical alternatives to develop estimates of the 2- to 500-year peak discharges for rural and urban streams in Maryland,

Defining guidelines for the application of the Muskingum-Cunge routing module in the NRCS WinTR-20,

Developing guidelines for estimating NRCS runoff curve numbers from information on planning and zoning maps,

Investigating the effects of depth-dependent Manning roughness coefficients on the time of concentration,

Investigation of the procedure for estimating the model inputs such as impervious area and curve number from zoning classifications,

Developing an improved approach for determining a homogeneous period for frequency analysis for urban watersheds or for adjusting the annual peak data to existing land use conditions.

Developing improved guidelines for selecting concurrent return periods for storm surge and riverine peak discharges, and

Planning for climate change in terms of both tidal-influenced systems affected by sea level rise and in riverine systems where precipitation intensity-duration-frequency is anticipated to change.

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APPENDIX 1
WATERSHED PROPERTIES
FOR USGS STREAM GAGES
IN MARYLAND AND DELAWARE

Watershed Properties for USGS Stream Gages in Maryland and Delaware

This appendix tabulates the values used in developing the current Fixed Region Regression Equations (FRRE) for estimating T-year peak discharges (as detailed in Chapter 2 and Appendix 3 of this Report). For the western part of the state (Piedmont, Blue Ridge and Appalachian Plateau Regions), regression equations were developed in 2016 with some minor revisions in 2020. For the Eastern and Western Coastal Plain regions, regression equations were developed in 2022. The FRRE for different regions use different properties as predictor variables, and no set of equations uses all the properties.

Sixteen properties are tabulated for many of the 196 stations listed in this Appendix. Each station's values appear as a table row crossing facing pages (left and right); table columns 1 through 9 on the left-hand page, and 10 through 16 on the right-hand page.

Property Name and Description	**Table Column Number**
Station Number: the station identification number as reported by the USGS. The leading zero of each gage is omitted.	-
Station Name: the station name as reported by the USGS.	-
Years of Record: the number of years of gage record, excluding those years of regulated gage record (range: 9 – 89 years).	1
Area: probably the single most important watershed characteristic for hydrologic design. It reflects the volume of water that can be generated from rainfall. GIS calculated variable equal to the number of pixels composing the watershed times the pixel's area or cell size (mi^2) (range: 0.027 – 816.45 mi^2).	2
Land Slope: the average basin slope is the average of all neighborhood slopes determined along the steepest direction of flow. These are the local slopes determined from the upstream to downstream pixel for each pixel within the watershed (ft/ft). This quantity is represented by the symbol "LSLOPE" in the Fixed Region Method text (range: 0.00463 – 0.25265 ft/ft).	3
Lime: the percentage of limestone within the watershed (%). (range: 0 – 100 percent).	4
Hyd. A: the percentage of the basin defined as hydrologic soil A, computed as the number of pixels of hydrologic soil A divided by the number of pixels in the basin (%). This is computed from SSURGO soils data (range: 0 – 86.2 %).	5

Property Name and Description	Table Column Number
Hyd. B: the percentage of the basin defined as hydrologic soil B, computed as the number of pixels of hydrologic soil B divided by the number of pixels in the basin (%). This is computed from SSURGO soils data (range: 0 – 100 %).	6
Hyd. C: the percentage of the basin defined as hydrologic soil C, computed as the number of pixels of hydrologic soil C divided by the number of pixels in the basin (%). This is computed from SSURGO soils data (range: 0 – 100 %).	7
Hyd. D: the percentage of the basin defined as hydrologic soil D, computed as the number of pixels of hydrologic soil D divided by the number of pixels in the basin (%). This is computed from SSURGO soils data (range: 0 – 89.2 %).	8
Province: the physiographic province in which the watershed is located (A = Appalachian, B = Blue Ridge, E = Eastern Coastal Plain, P = Piedmont, W = Western Coastal Plain).	9
IA70: the percentage of the basin defined as impervious area by the USGS 1970's land use (%). Impervious area includes the following land use classifications: residential, commercial, industrial, transportation, industrial/commercial complexes, mixed urban or built-up land, dry salt flats, and bare exposed rock (range: 0 – 49.3 %).	10
IA85: the percentage of the basin defined as impervious area by the Ragan 1985 land use (%). Impervious area includes the following land use classifications: low density residential, medium density residential, high density residential, commercial, industrial, institutional, extractive, open urban land, bare exposed rock, and bare ground (range: 0 – 41.1 %).	11
IA90: the percentage of the basin defined as impervious area by the MOP 1990 land use (%). Impervious area includes the following land use classifications: low density residential, medium density residential, high density residential, commercial, industrial, institutional, extractive, open urban land, bare exposed rock, and bare ground (range: 0 – 43.8 %).	12
IA97: the percentage of the basin defined as impervious area by the MOP 1997 land use (%) (range: 0 – 45.4 %).	13
IA00: the percentage of the basin defined as impervious area by the MOP 2000 land use (%) (range: 0 – 50.7 %).	14
IA02: the percentage of the basin defined as impervious area by the MOP 2002 land use (%) (range: 0 – 51.1 %).	15
IA10: the percentage of the basin defined as impervious area by the MOP 2010 land use (%) (range: 0 – 53.5 %).	16

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1483155*	Silver Lake Tributary at Middleton, DE	16	2.03	0.02045	0	2.3	94.5	1.4	1.6	E
1483200	Blackbird Creek at Blackbird, DE	54	4.06	0.01898	-	34.1	33.2	12.6	20	E
1483290	Paw Paw Branch Tributary near Clayton, DE	10	0.91	0.01053	0	6.5	29.4	22.6	41.4	E
1483500	Leipsic River near Cheswold, DE	34	9.21	0.0161	0	14.1	47.6	11.7	26.6	E
1483720	Puncheon Branch at Dover, DE	10	2.41	0.01334	0	46.4	21.7	24.3	7.6	E
1484000	Murderkill River near Felton, DE	35	12.64	0.00949	0	26.1	12	8.8	51.2	E
1484002*	Murderkill River Tributary near Felton, DE	10	0.96	0.01201	0	86.2	6.8	0	7	E
1484050	Pratt Branch near Felton, DE	10	3.1	0.01292	0	11.6	64.8	9.2	14.5	E
1484100	Beaverdam Branch at Houston, DE	60	3.31	0.0073	0	29.6	5	0	65.3	E
1484270	Beaverdam Creek near Milton, DE	19	6.21	0.01195	0	73.5	6.4	3.8	16.3	E
1484300	Sowbridge Branch near Milton, DE	22	7.45	0.01045	0	82.7	0.6	1.2	15.4	E
1484500	Stockley Branch at Stockley, DE	62	4.8	0.00805	0	26.2	17.2	2.6	52.7	E
1484550	Pepper Creek at Dagsboro, DE	16	8.31	0.00463	0	3.6	7.1	0.1	89.2	E
1484695	Beaverdam Ditch near Millville, DE	19	2.71	0.0062	0	6.6	8.4	2.4	82.5	E
148471320*	Birch Branch at Sowell, MD	18	6.38	0.00619	0	36.2	5.6	10.7	47.6	E
1484719	Bassett Creek near Ironshire, MD	10	1.39	0.01248	0	2	9.1	70.3	18.6	E
1485000	Pocomoke River near Willards, MD	66	51.61	0.00667	0	20	3.1	0.2	76.7	E
1485500	Nassawango Creek near Snow Hill, MD	68	45.47	0.00841	0	26.3	3.5	0.9	69.2	E
1486000	Manokin Branch near Princess Anne, MD	64	3.98	0.00544	0	28.5	10.9	11.1	49.5	E
1486100*	Andrews Branch near Delmar, MD	10	4.54	0.01044	0	26.4	0.2	0	73.4	E
1486980*	Toms Dam Branch near Greenwood, DE	10	5.97	0.00593	0	14.6	6.2	27.9	51.3	E
1487000	Nanticoke River near Bridgeville, DE	75	71.99	0.00768	0	18.2	22.5	17.2	42	E

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1483155*	Silver Lake Tributary at Middleton, DE	-	-	-	-	-	4.4	-
1483200	Blackbird Creek at Blackbird, DE	-	-	-	-	-	4.3	-
1483290	Paw Paw Branch Tributary near Clayton, DE	-	-	-	-	-	1	-
1483500	Leipsic River near Cheswold, DE	-	-	-	-	-	1	-
1483720	Puncheon Branch at Dover, DE	-	-	-	-	-	1	-
1484000	Murderkill River near Felton, DE	-	-	-	-	-	4.3	-
1484002*	Murderkill River Tributary near Felton, DE	-	-	-	-	-	1.8	-
1484050	Pratt Branch near Felton, DE	-	-	-	-	-	1	-
1484100	Beaverdam Branch at Houston, DE	-	-	-	-	-	2	-
1484270	Beaverdam Creek near Milton, DE	-	-	-	-	-	5.8	-
1484300	Sowbridge Branch near Milton, DE	-	-	-	-	-	2.6	-
1484500	Stockley Branch at Stockley, DE	-	-	-	-	-	5	-
1484550	Pepper Creek at Dagsboro, DE	-	-	-	-	-	1.9	-
1484695	Beaverdam Ditch near Millville, DE	-	-	-	-	-	4.8	-
148471320*	Birch Branch at Sowell, MD	-	-	-	-	-	0.9	-
1484719	Bassett Creek near Ironshire, MD	-	-	-	-	-	-	0.9
1485000	Pocomoke River near Willards, MD	0.2	1.5	0.5	0.7	-	1.2	-
1485500	Nassawango Creek near Snow Hill, MD	0.8	1.7	1.3	2	-	-	2.3
1486000	Manokin Branch near Princess Anne, MD	0	1.5	0.6	0.8	-	-	2.2
1486100*	Andrews Branch near Delmar, MD	-	1	-	-	-	-	2.5
1486980*	Toms Dam Branch near Greenwood, DE	-	-	-	-	-	1	-
1487000	Nanticoke River near Bridgeville, DE	-	-	-	-	-	4.2	-

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1487900	Meadow Branch near Delmar, DE	9	2.73	0.00575	0	19.4	6.5	0.8	73.4	E
1488500	Marshyhope Creek near Adamsville, DE	45	46.47	0.00636	0	6.3	6.7	9.4	77.7	E
1489000	Faulkner Branch near Federalsburg, MD	42	8.06	0.00805	0	24.6	37.6	19.6	18.2	E
1490000	Chicamacomico River near Salem, MD	46	16.96	0.00757	0	44.8	19.9	3.8	31.3	E
1490600	Meredith Branch Near Sandtown, DE	10	8.76	0.00643	-	3.5	8	13	75.6	E
1490800	Oldtown Branch at Goldsboro, MD	10	4.45	0.00951	0	9.2	31	19	40.7	E
1491000	Choptank River near Greensboro, MD	71	113.8	0.00922	0	11.5	15	12.8	60.6	E
1491010	Sangston Prong near Whiteleysburg, DE	10	1.94	0.00699	0	5	19.4	21.6	54	E
1491050	Spring Branch near Greensboro, MD	10	3.76	0.01008	0	14.7	41.8	22	21.5	E
1491500	Tuckahoe Creek near Ruthsburg, MD	22	87.67	0.01189	-	21.1	22.3	30.2	26.3	E
1492000	Beaverdam Branch at Matthews, MD	34	6.05	0.01794	0	6.8	36.4	39.1	17.7	E
1492050	Gravel Run at Beulah, MD	11	8.53	0.01385	0	72.1	14.7	4.2	8.4	E
1492500	Sallie Harris Creek near Carmicheal, MD	47	8	0.01948	0	11.7	13	56.9	18.3	E
1492550	Mill Creek near Skipton, MD	11	4.24	0.01814	0	10.5	54.8	26.8	8	E
1493000	Unicorn Branch near Millington, MD	69	20.67	0.0127	0	39.2	21.4	14.7	24.4	E
1493112	Chesterville Branch near Crumpton, MD	13	6.14	0.01857	0	0.5	28.6	65.8	4.9	E
1493500	Morgan Creek near Kennedyville, MD	66	12.73	0.02445	0	1.8	21.2	72.6	3.9	E
1494000	Southeast Creek at Church Hill, MD	14	12.6	0.01893	0	39.9	14.8	25.7	19.6	E
1494150	Three Bridges Branch at Centerville, MD	11	8.24	0.022	0	21.9	17.8	40.8	19.3	E
1495000	Big Elk Creek at Elk Mills, MD	80	53.36	0.073	0	0	76	10.3	13.4	P
1495500	Little Elk Creek at Childs, MD	12	26.46	0.06752	0	0	65.2	22	12.6	P
1496000	Northeast River at Leslie, MD	37	24.87	0.04863	0	0	54.6	27.5	17.6	P

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1487900	Meadow Branch near Delmar, DE	-	-	-	-	-	1	-
1488500	Marshyhope Creek near Adamsville, DE	-	-	-	-	-	1.9	-
1489000	Faulkner Branch near Federalsburg, MD	0.6	3	1.4	2.1	-	-	1.8
1490000	Chicamacomico River near Salem, MD	0.2	0.2	0.4	0.8	-	-	0.9
1490600	Meredith Branch Near Sandtown, DE	-	-	-	-	-	2.2	-
1490800	Oldtown Branch at Goldsboro, MD	0.4	0.7	2	2.9	-	-	2
1491000	Choptank River near Greensboro, MD	1.1	0.8	1.6	2.2	-	3.9	-
1491010	Sangston Prong near Whiteleysburg, DE	-	-	-	-	-	0.3	-
1491050	Spring Branch near Greensboro, MD	0.7	0	0.3	0.6	-	0.7	-
1491500	Tuckahoe Creek near Ruthsburg, MD	-	-	-	-	-	-	1.3-
1492000	Beaverdam Branch at Matthews, MD	0.4	0.6	0.4	1	-	-	2.2
1492050	Gravel Run at Beulah, MD	0.4	1.3	0.5	2	-	-	2
1492500	Sallie Harris Creek near Carmicheal, MD	1.8	0.1	0.3	0.6	-	-	1.3
1492550	Mill Creek near Skipton, MD	0	0	0	0.1	-	-	1.4
1493000	Unicorn Branch near Millington, MD	-	-	-	-	-	-	1.3
1493112	Chesterville Branch near Crumpton, MD	-	-	-	-	-	-	0.4
1493500	Morgan Creek near Kennedyville, MD	0.4	0.6	0.6	0.8	-	-	1
1494000	Southeast Creek at Church Hill, MD	0.2	0.6	0.7	0.9	-	-	1.6
1494150	Three Bridges Branch at Centerville, MD	-	-	-	-	-	-	5.1
1495000	Big Elk Creek at Elk Mills, MD	3.7	6.4	2.1	2.5	2.7	3.1	0.8
1495500	Little Elk Creek at Childs, MD	4	2.5	4.2	6.3	-	-	-
1496000	Northeast River at Leslie, MD	3	1.9	2.5	3.2	-	-	-

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1496080	Northeast River Tributary near Charlestown, MD	10	1.75	0.073	0	0	31	60.6	8.4	P
1496200	Principio Creek near Principio Furnace, MD	27	9	0.06388	0	0	68.6	19.7	11.6	P
1577940	Broad Creek tributary at Whiteford, MD	16	0.67	0.0743	0	4.7	85	10.3	0	P
1578500	Octoraro Creek near Rising Sun, MD	19	191.7	0.08256	0	0	60.4	28.2	10.4	P
1578800	Basin Run at West Nottingham, MD	10	1.25	0.05	0	0	71.5	15.5	12.9	P
1579000	Basin Run at Liberty Grove, MD	22	5.08	0.06	0	0	71.9	15.1	13	P
1580000	Deer Creek at Rocks, MD	86	94.31	0.103	0	2.7	82.1	12.4	2.5	P
1580200	Deer Creek at Kalmia, MD	11	127.2	0.09671	0	2.4	78.9	15.2	3.2	P
1581500	Bynum Run at Bel Air, MD	38	8.79	0.048	0	0.2	39.9	35.6	24.2	P
1581700	Winter Run near Benson, MD	45	34.64	0.07	0	0.7	76.8	16.5	5.8	P
1581752	Plumtree Creek near Bel Air, MD	11	2.47	0.048	0	1.7	66.4	18.4	13.5	P
1581810	Gunpowder Falls at Hoffmanville, MD	12	27.46	0.112	2	31.7	54.7	7.7	5.9	P
1581830*	Grave Run near Beckleysville, MD	13	7.56	0.097	0	27.5	59.9	5.3	7.2	P
1581870	Georges Run near Beckleysville, MD	13	15.76	0.075	0	14.3	69.1	8.7	7.7	P
1581940	Mingo Branch near Hereford, MD	10	0.77	0.105	0	23.2	72.1	4.7	0	P
1581960	Beetree Run at Bentley Springs, MD	13	9.66	0.098	0	3.4	78.8	16.8	1	P
1582000	Little Falls at Blue Mount, MD	69	53.7	0.103	0	2.6	83.4	11.2	2.6	P
1582510	Piney Creek near Hereford, MD	14	1.39	0.07866	0	0.2	81.2	16.8	1.8	P
1583000*	Slade Run near Glyndon, MD	36	2.05	0.088	0	0	87.9	10.9	1.2	P
1583100	Piney Run at Dover, MD	23	12.45	0.083	0	2.4	85.1	9.4	3.1	P
1583495	Western Run tributary at Western Run, MD	10	0.23	0.08274	0	0	77.4	18.2	4	P
1583500	Western Run at Western Run, MD	68	60.31	0.082	0	1.6	83.6	10.1	4.6	P
1583570*	Pond Branch at Oregon Ridge, MD	17	0.131	0.101	0	2.4	59.2	38.5	0	P

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1496080	Northeast River Tributary near Charlestown, MD	-	1.5	-	-	-	-	-
1496200	Principio Creek near Principio Furnace, MD	0	1	1.2	2.8	-	-	-
1577940	Broad Creek tributary at Whiteford, MD	1.3	1.6	3	3.8	-	-	-
1578500	Octoraro Creek near Rising Sun, MD	1.3	1.9	3.5	5.5	-	-	-
1578800	Basin Run at West Nottingham, MD	-	2.5	-	-	-	-	-
1579000	Basin Run at Liberty Grove, MD	-	2.9	-	-	-	-	-
1580000	Deer Creek at Rocks, MD	0.7	1	2.4	2.7	2.8	3.2	3.9
1580200	Deer Creek at Kalmia, MD	0.6	1.2	2.6	3.1	-	-	-
1581500	Bynum Run at Bel Air, MD	10.8	12.9	19.6	23.6	26.6	27.6	33.4
1581700	Winter Run near Benson, MD	2.8	4.6	6.4	8.1	8.7	9.5	13
1581752	Plumtree Creek near Bel Air, MD	-	-	-	29.1	31.8	31.5	42.9
1581810	Gunpowder Falls at Hoffmanville, MD	-	-	-	3.9	4.6	4.7	4.9
1581830*	Grave Run near Beckleysville, MD	-	-	-	2.9	3.2	3.5	5.4
1581870	Georges Run near Beckleysville, MD	-	-	-	5.3	5.9	6.3	7.8
1581940	Mingo Branch near Hereford, MD	-	-	-	2	2	2.5	3.9
1581960	Beetree Run at Bentley Springs, MD	-	-	-	3.9	4	4.3	4.8
1582000	Little Falls at Blue Mount, MD	1	1.3	2.6	3.3	3.4	4	5.3
1582510	Piney Creek near Hereford, MD	0.6	2.4	3.3	3.4	-	-	-
1583000*	Slade Run near Glyndon, MD	2.1	1.2	2.5	2.5	2.9	3.3	4.3
1583100	Piney Run at Dover, MD	0.5	1.9	1.9	3.4	3.4	3.8	4.7
1583495	Western Run tributary at Western Run, MD	0	0	0	2.7	-	-	-
1583500	Western Run at Western Run, MD	0.4	1.5	2.1	2.7	3	3.4	4.4
1583570*	Pond Branch at Oregon Ridge, MD	-	-	-	0	0	0	0

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1583580	Baisman Run at Broadmoor, MD	26	1.49	0.108	0	4.9	73.6	20.4	1.1	P
1583600*	Beaverdam Run at Cockeysville, MD	29	20.88	0.076	0	4.1	69.3	18.5	7.8	P
158397967	Minebank Run near Glen Arm, MD	11	2.1	0.091	0	2	79.8	7.7	10.5	P
1584050	Long Green Creek at Glen Arm, MD	37	9.31	0.065	0	0.9	76.3	17	5.6	P
1584500	Little Gunpowder Falls at Laurel Brook, MD	72	36.04	0.071	0	1.1	79.7	14.4	4.8	P
1585090	Whitemarsh Run near Fullerton, MD	18	2.58	0.06888	0	8.7	59.1	23.1	9	P
1585095	North Fork Whitemarsh Run near White Marsh, MD	17	1.36	0.049	0	2.2	28.9	55.6	13.1	P
1585100	White Marsh Run at White Marsh, MD	40	7.56	0.061	0	8.8	44.1	30.1	16.6	P
1585104	Honeygo Run near White Marsh, MD	13	2.44	0.054	0	3	47.4	32.2	17.2	P
1585200	West Branch Herring Run at Idlewylde, MD	46	2.31	0.059	0	9.5	68.1	12.3	10.1	P
1585225	Moores Run tributary near Todd Ave at Baltimore, MD	16	0.14	0.051	0	0	66.9	8	25.1	P
1585230	Moores Run at Radecke Ave at Baltimore, MD	16	3.5	0.045	0	1	50.1	30.6	18.4	P
1585300	Stemmers Run at Rossville, MD	29	4.54	0.062	0	9.5	8.8	53.8	27.8	W/P
1585400	Brien Run at Stemmers Run, MD	29	1.96	0.035	0	5.3	26	27.5	53.8	W/P
1585500	Cranberry Branch near Westminster, MD	64	3.26	0.081	0	29.1	54.6	10.5	4.8	P
1586000	North Branch Patapsco River at Cedarhurst, MD	67	55.48	0.081	3.1	20.3	65.7	7.5	6.2	P
1586210*	Beaver Run near Finksburg, MD	30	14.11	0.079	0	30.4	57.1	5.7	6.7	P
1586610	Morgan Run near Louisville, MD	30	28.01	0.089	0.1	49	38.5	6.8	5.5	P
1587000	North Branch Patapsco River near Marriottsville, MD	24	164.2	0.09138	1.74	22.4	61.7	8.2	4.9	P

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	Station Name	IA70 (%)	IA85 (%)	IA90 (%)	IA97 (%)	IA00 (%)	IA02 (%)	IA10 (%)
1583580	Baisman Run at Broadmoor, MD	-	4.5	-	8.4	8.4	9	10.4
1583600*	Beaverdam Run at Cockeysville, MD	14.5	18	18.9	22	23.3	24.5	27.5
158397967	Minebank Run near Glen Arm, MD	-	-	-	33.6	36.6	37.4	40.2
1584050	Long Green Creek at Glen Arm, MD	2.9	5.6	5.8	5.7	5.3	6.2	7
1584500	Little Gunpowder Falls at Laurel Brook, MD	1.1	3.5	4.3	5	5.1	5.2	6.8
1585090	Whitemarsh Run near Fullerton, MD	-	-	-	43.2	43.2	44	47.2
1585095	North Fork Whitemarsh Run near White Marsh, MD	-	-	-	38.3	40.3	42.9	42.3
1585100	White Marsh Run at White Marsh, MD	18.9	21.6	25.8	37.7	38.9	40.9	42.6
1585104	Honeygo Run near White Marsh, MD	-	-	-	14.2	14.7	15.1	22.5
1585200	West Branch Herring Run at Idlewylde, MD	41.4	37.5	37.8	42.1	42.1	43.7	43.2
1585225	Moores Run tributary near Todd Ave at Baltimore, MD	-	-	-	42	39.2	40.2	41.1
1585230	Moores Run at Radecke Ave at Baltimore, MD	-	-	-	42.5	44.1	44	45.4
1585300	Stemmers Run at Rossville, MD	30.4	25.3	25.4	29.3	-	-	37.1
1585400	Brien Run at Stemmers Run, MD	31.1	36.8	39.4	45	-	-	52.1
1585500	Cranberry Branch near Westminster, MD	0.9	4.2	5.5	5.5	7.3	7.2	7.5
1586000	North Branch Patapsco River at Cedarhurst, MD	2.6	5.4	6.6	8.5	9.1	9.8	12.1
1586210*	Beaver Run near Finksburg, MD	3.1	6	7	10.1	11.9	12.3	14.5
1586610	Morgan Run near Louisville, MD	0.7	3	4	4.9	4.9	5	6.7
1587000	North Branch Patapsco River near Marriottsville, MD	2	4.6	5.5	7.2	-	-	-

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1587050	Hay Meadow Branch tributary at Poplar Springs, MD	11	0.49	0.08716	0	0	84.5	8.2	7.3	P
1587500	South Branch Patapsco River at Henryton, MD	32	64.26	0.09709	0	24.9	59.2	8.5	7.2	P
1588000	Piney Run near Sykesville, MD	43	11.4	0.07545	0	18.5	65	9.5	6.9	P
1589000	Patapsco River at Hollofield, MD	23	284.7	0.09301	0	20.9	62.4	9.2	5.7	P
1589100	East Branch Herbert Run at Arbutus, MD	47	2.47	0.054	0	4	76.6	9.8	9.4	P
1589180	Gwynns Falls at Glyndon, MD	14	0.31	0.026	0	0	75.6	14.4	10.1	P
1589197	Gwynns Falls near Delight, MD	14	4.09	0.049	0	0.1	61.1	31.3	7.5	P
1589200	Gwynns Falls near Owings Mills, MD	17	4.89	0.05587	0	0	86.4	5.9	7.7	P
1589238*	Gwynns Falls tributary at McDonough, MD	13	0.027	0.056	0	0	100	0	0	P
1589240	Gwynns Falls at McDonough, MD	12	19.27	0.06318	0	0.4	67.5	26.4	5.6	P
1589300	Gwynns Falls at Villa Nova, MD	34	32.59	0.056	0	0.5	65.6	25.8	8	P
1589330	Dead Run at Franklintown, MD	31	5.52	0.047	0	0	41.3	35.3	23.3	P
1589352	Gwynns Falls at Washington Blvd at Baltimore, MD	14	63.57	0.057	0	0.4	61.6	22.6	15.3	P
1589440	Jones Fall at Sorrento, MD	47	25.21	0.078	0	4	73.5	13.7	8.7	P
1589464	Stony Run at Ridgemede Road at Baltimore, MD	9	2.26	0.05	0	0.6	85.5	9.3	4.4	P
1589500	Sawmill Creek at Glen Burnie, MD	34	5.04	0.036	0	67.2	0.5	17.4	14.8	W
1589795	South Fork Jabez Branch at Millersville, MD	22	0.96	0.048	0	15.1	46.2	14.4	24.3	W
1590000	North River near Annapolis, MD	43	8.63	0.101	0	20.4	31.9	36.7	11	W
1590500	Bacon Ridge Branch at Chesterfield, MD	35	6.97	0.114	0	30.2	33.3	25.2	11.1	W
1591000	Patuxent River near Unity, MD	68	34.95	0.092	0	0	68.5	13.3	18.1	P
1591400	Cattail Creek near Glenwood, MD	46	22.86	0.08	0	0	76	13.6	9.9	P

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1587050	Hay Meadow Branch tributary at Poplar Springs, MD	0.6	10	10.3	10.9	-	-	-
1587500	South Branch Patapsco River at Henryton, MD	2.2	4	4.7	7.1	-	-	-
1588000	Piney Run near Sykesville, MD	1.5	4.6	4.7	6.9	-	-	-
1589000	Patapsco River at Hollofield, MD	2.4	4.7	5.6	7.4	-	-	-
1589100	East Branch Herbert Run at Arbutus, MD	49.3	33.8	39	44.6	44.7	44.7	47.9
1589180	Gwynns Falls at Glyndon, MD	-	-	-	37.6	37.8	39.5	42
1589197	Gwynns Falls near Delight, MD	-	-	-	33.5	34.7	36.6	37.7
1589200	Gwynns Falls near Owings Mills, MD	15.5	14.6	17.5	26.7	-	-	-
1589238*	Gwynns Falls tributary at McDonough, MD	-	-	-	0	0	0	0
1589240	Gwynns Falls at McDonough, MD	14.2	16.6	19.3	27.4	-	-	-
1589300	Gwynns Falls at Villa Nova, MD	19.7	19.5	21.6	30	31	32.7	35.7
1589330	Dead Run at Franklintown, MD	43.1	41.1	43.8	45.4	46	49.3	51.9
1589352	Gwynns Falls at Washington Blvd at Baltimore, MD	-	-	-	37.6	38.3	39.3	41.3
1589440	Jones Fall at Sorrento, MD	12.1	11.4	13.7	14.9	15.6	16.7	18.9
1589464	Stony Run at Ridgemede Road at Baltimore, MD	-	-	-	41	41	40.6	41.7
1589500	Sawmill Creek at Glen Burnie, MD	26	11.5	23.5	28.7	-	29.7	33.5
1589795	South Fork Jabez Branch at Millersville, MD	-	8.2	-	-	-	16.8	20
1590000	North River near Annapolis, MD	2	2.7	3	5.2	-	8.1	9.3
1590500	Bacon Ridge Branch at Chesterfield, MD	4.5	1.5	3.7	4.6	-	5.4	9.7
1591000	Patuxent River near Unity, MD	0.4	1.4	2.1	2.1	2.1	2.6	3.9
1591400	Cattail Creek near Glenwood, MD	2	2.9	3	4.3	4.6	5.8	8.3

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1591700	Hawlings River near Sandy Spring, MD	34	27.31	0.056	0	0	76.6	8.3	14.8	P
1592000*	Patuxent River near Burtonsville, MD	32	127	0.09	0	0	72.8	11.9	13.8	P
1593350	Little Patuxent River tributary at Guilford Downs, MD	11	1.06	0.05	0	0	68.4	11.7	19.9	P
1593500*	Little Patuxent River at Guilford, MD	80	38.1	0.053	0	0	65.1	11.3	22.9	P
1594000	Little Patuxent River at Savage, MD	59	98.25	0.059	0	0.1	68.5	13.9	17.1	P
1594400*	Dorsey Run near Jessup, MD	20	11.91	0.051	0	10.5	10.9	34.8	43.7	W
1594440	Patuxent River near Bowie, MD	41	350.2	0.064	0	16.2	41.7	23.8	17.3	W
1594445	Mill Branch near Mitchellville, MD	11	1.25	0.033	0	10.7	14	60.1	14.2	W
1594500*	Western Branch near Largo, MD	25	30.04	0.047	0	21.9	30.5	13.2	33.7	W
1594526	Western Branch at Upper Marlboro, MD	29	89.38	0.055	0	16.1	31.2	30.3	21.9	W
1594600	Cocktown Creek near Huntington, MD	19	3.9	0.094	0	45	10.2	1.3	43.5	W
1594670	Hunting Creek near Huntingtown, MD	10	9.33	0.098	0	57.1	4.6	3.4	34.9	W
1594710*	Killpeck Creek at Huntersville, MD	12	3.46	0.08	0	16.8	51.4	24.5	7.2	W
1594800	St. Leonard Creek near St. Leonard, MD	14	7.23	0.099	0	85.2	0.2	8.3	6.2	W
1594930	Laurel Run at Dobbin Road near Wilson, MD	26	8.23	0.15523	0	0	3	84.6	12.2	A
1594936	North Fork Sand Run near Wilson, MD	28	1.91	0.1516	0	0	2.6	84.8	12.5	A
1594950	McMillan Fork near Fort Pendleton, MD	25	2.36	0.13418	0	0	0.4	94.1	5.6	A
1596005	Savage River near Frostburg, MD	14	1.43	0.09585	0	1.2	26.5	45.9	26.4	A
1596500	Savage River near Barton, MD	54	48.53	0.22802	0	0.1	29.4	63.7	6.6	A
1597000	Crabtree Creek near Swanton, MD	33	16.75	0.21771	0	0	52.5	46.5	1.1	A
1598000	Savage River at Bloomington, MD	24	115.9	0.25265	0	0	27.9	67.2	4.3	A

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1591700	Hawlings River near Sandy Spring, MD	2.8	3.8	8.9	8.9	10.2	10.1	11.5
1592000*	Patuxent River near Burtonsville, MD	1.9	3.1	5.1	5.6	-	-	-
1593350	Little Patuxent River tributary at Guilford Downs, MD	36.2	34.8	32.5	32.9	-	-	-
1593500*	Little Patuxent River at Guilford, MD	16.9	18.5	21.7	27.5	27.5	28.4	31.2
1594000	Little Patuxent River at Savage, MD	9	11	13.3	17.6	18	19.1	21.5
1594400*	Dorsey Run near Jessup, MD	22	16.7	19.6	29.3	-	32.1	40.4
1594440	Patuxent River near Bowie, MD	9.6	8.6	10.7	12.9	-	14.9	17.6
1594445	Mill Branch near Mitchellville, MD	2.7	4.5	8	17.6	-	37.7	38.1
1594500*	Western Branch near Largo, MD	15.1	11.4	13.8	19	-	25.2	27.9
1594526	Western Branch at Upper Marlboro, MD	13.5	9.5	11.8	17.5	-	21.4	24.6
1594600	Cocktown Creek near Huntington, MD	21.6	8.7	9	14.6	-	15.3	16.8
1594670	Hunting Creek near Huntingtown, MD	7.4	1.5	2.4	5.6	-	5.9	8.5
1594710*	Killpeck Creek at Huntersville, MD	19.2	4.1	7.8	10.8	-	13.8	15.8
1594800	St. Leonard Creek near St. Leonard, MD	3.3	0.3	1.7	4.5	-	4.9	8.4
1594930	Laurel Run at Dobbin Road near Wilson, MD	0	1.3	1.4	1.1	2.6	2.7	1.9
1594936	North Fork Sand Run near Wilson, MD	0	0.9	0.9	0.5	0.5	0.5	0.9
1594950	McMillan Fork near Fort Pendleton, MD	0	0.6	0.4	1.2	1.2	1.2	1.6
1596005	Savage River near Frostburg, MD	1.2	1	0.8	3.7	-	-	-
1596500	Savage River near Barton, MD	0.1	0.3	0.3	0.6	0.8	0.8	1.3
1597000	Crabtree Creek near Swanton, MD	0.2	0.5	0.4	0.5	-	-	-
1598000	Savage River at Bloomington, MD	0.1	0.3	0.4	0.6	-	-	-

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1599000	Georges Creek at Franklin, MD	82	72.74	0.17098	0	4.6	13.5	77.5	4.5	A
1601500	Wills Creek near Cumberland, MD	83	247	0.21402	0	3.6	35.6	44.5	16.2	A
1609000	Town Creek near Oldtown, MD	33	149.5	0.20585	0	3.9	12.2	73.5	10.3	A
1609500	Sawpit Run near Oldtown, MD	25	5	0.17155	0	0	10.8	89.1	0.1	A
1610105	Pratt Hollow Tributary at Pratt, MD	15	0.65	0.19342	-	0	0	100	0	A
1610150	Bear Creek at Forest Park, MD	18	10.27	0.15619	0	0.6	8.5	89.4	1.4	A
1610155	Sideling Hill Creek near Bellegrove, MD	24	102.7	0.18753	0	0.4	7.5	87	5.1	A
1612500	Little Tonoloway Creek near Hancock, MD	17	17.28	0.16977	0	0.3	13.4	64.9	21.2	A
1613150	Ditch Run near Hancock, MD	22	4.6	0.12376	0	0	3.6	93.7	2.2	A
1613160	Potomac River tributary near Hancock, MD	12	1.24	0.14781	0	-	-	-	-	A
1614500	Conococheague Creek at Fairview, MD	85	502.4	0.1	41.5	0.4	33.6	53.7	11.8	B
1617800*	Marsh Run at Grimes, MD	48	18.34	0.035	100	1.2	71.7	3.3	23.7	B
1619000	Antietam Creek near Waynesboro, PA	27	93.9	0.103	64.6	0.3	43	49	7.6	B
1619475	Dog Creek tributary near Locust Grove, MD	11	0.11	0.0805	81.7	11	76.7	11.6	0.3	B
1619500	Antietam Creek near Sharpsburg, MD	85	280.9	0.081	75.6	0.6	55.2	29.7	14.1	B
1637000	Little Catoctin Creek at Harmony, MD	30	8.76	0.15203	0	0	51.7	45.1	3.2	B
1637500	Catoctin Creek near Middletown, MD	65	67.33	0.124	0	0	47.2	47.1	5.3	B
1637600	Hollow Road Creek near Middletown, MD	11	2.32	0.13042	0	0	56.9	41	1.9	B
1639000	Monocacy River at Bridgeport, MD	72	172.7	0.052	1.3	0	25.6	60.6	13.1	B
1639095*	Piney Creek tributary at Taneytown, MD	10	0.61	0.0338	0	0	6.2	93.8	0	B
1639140	Piney Creek near Taneytown, MD	12	31.07	0.042	2.4	16.9	22.2	57	3.9	B

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	Station Name	IA70 (%)	IA85 (%)	IA90 (%)	IA97 (%)	IA00 (%)	IA02 (%)	IA10 (%)
1599000	Georges Creek at Franklin, MD	2.2	3.7	3.4	3.9	3.9	3.8	4.2
1601500	Wills Creek near Cumberland, MD	1	4.2	4.4	5.8	5.9	5.9	1.5
1609000	Town Creek near Oldtown, MD	0.2	0.3	0.1	0.5	0.5	0.5	0.5
1609500	Sawpit Run near Oldtown, MD	0	0	0	0.4	-	-	-
1610105	Pratt Hollow Tributary at Pratt, MD	-	0	-	-	-	-	-
1610150	Bear Creek at Forest Park, MD	1.4	0	3.2	3.3	-	-	-
1610155	Sideling Hill Creek near Bellegrove, MD	0.4	0	0.5	1.1	0.8	0.8	0.5
1612500	Little Tonoloway Creek near Hancock, MD	1.7	0	1.4	2	-	-	-
1613150	Ditch Run near Hancock, MD	0.3	0	0.8	1.9	-	-	-
1613160	Potomac River tributary near Hancock, MD	-	2	-	-	-	-	-
1614500	Conococheague Creek at Fairview, MD	2.4	1.6	7.1	8.7	0	0	0.1
1617800*	Marsh Run at Grimes, MD	3.2	3.4	5.1	5.9	7.9	7	9.2
1619000	Antietam Creek near Waynesboro, PA	2.6	3.9	5.9	7.8	11.5	8	0.7
1619475	Dog Creek tributary near Locust Grove, MD	0	0	0	0	-	-	-
1619500	Antietam Creek near Sharpsburg, MD	3.6	4.8	5.4	7.6	9.3	8.9	6.7
1637000	Little Catoctin Creek at Harmony, MD	0.6	0.8	2.5	2.8	-	-	-
1637500	Catoctin Creek near Middletown, MD	1.2	0.8	1.5	2.6	2.9	3.4	6.1
1637600	Hollow Road Creek near Middletown, MD	6	1.5	1.8	3.6	-	-	-
1639000	Monocacy River at Bridgeport, MD	1.2	0.8	0.9	0.9	1	1.1	0.1
1639095*	Piney Creek tributary at Taneytown, MD	6.2	11.4	10.9	19.7	-	-	-
1639140	Piney Creek near Taneytown, MD	-	-	-	3.7	3.9	4	4

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1639500	Big Pipe Creek at Bruceville, MD	65	103	0.081	1.1	49.9	19.5	26.4	4.1	B
1640000	Little Pipe Creek at Bruceville, MD	30	8.11	0.09645	76.5	67.4	22.6	9.4	0.6	P
1640500	Owens Creek at Lantz, MD	53	6.1	0.12628	0	0	42.7	49.9	7.4	B
1640700	Owens Creek tributary near Rocky Ridge, MD	11	1.12	0.04022	0	0	1.7	85.1	12.8	B
1640965	Hunting Creek near Foxville, MD	13	2.19	0.14899	0	0	30.2	64.6	5.2	B
1640970	Hunting Creek tributary near Foxville, MD	10	3.91	0.11883	0	0	34.4	55.9	9.7	B
1641000	Hunting Creek at Jimtown, MD	43	18.69	0.13256	16.23	3.4	38.9	51.1	6.1	B
1641500	Fishing Creek near Lewistown, MD	39	7.3	0.141	0	0	78.8	17.8	3	B
1642000	Monocacy River near Frederick, MD	35	665.1	0.08206	14.14	12.7	31.2	46.8	8.8	P
1642400	Dollyhyde Creek at Libertytown, MD	10	2.67	0.07329	0	6.2	45.9	30.9	16.9	P
1642500	Lingamore Creek near Frederick, MD	49	82.37	0.09365	0	20.4	50.7	18.9	9.3	P
1643000	Monocacy River at Jug Bridge near Frederick, MD	84	816.5	0.076	12.3	13.4	36	41.3	8.8	P/B
1643395	Soper Branch at Hyattstown, MD	9	1.18	0.1	0	0	5.6	8.1	86.3	B
1643500	Bennett Creek at Park Mills, MD	62	62.94	0.103	0	5	37.8	23.1	34.1	B
1644371	Little Seneca Creek tributary near Clarksburg, MD	9	0.42	0.068	0	0	59.4	19.4	21.3	P
1644375	Little Seneca Creek tributary near Germantown, MD	9	1.29	0.043	0	0	84.5	4.9	9.9	P
1644380	Cabin Branch near Boyd, MD	9	0.81	0.091	0	0	41.8	23.2	35	P
1644420	Bucklodge Branch tributary near Barnesville, MD	10	0.28	0.07449	0	0	9.2	32.3	58.5	B
1644600	Great Seneca Creek near Quince Orchard, MD	12	53.89	0.073	0	0	58.4	13.7	27.3	P
1645000	Seneca Creek near Dawsonville, MD	48	102.2	0.073	0	0	49	20.1	29.9	B
1645200	Watts Branch at Rockville, MD	30	3.7	0.05605	0	0	82.5	3.9	13.6	P
1646550	Little Falls Branch near Bethesda, MD	40	4.09	0.05174	0	0	84.1	0.7	15.2	P
1647720	North Branch Rock Creek near Norbeck, MD	11	9.68	0.05331	0	0	79.1	4.7	16	P

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
1639500	Big Pipe Creek at Bruceville, MD	0.2	1.8	2.5	3.1	3.4	3.4	5
1640000	Little Pipe Creek at Bruceville, MD	7.6	6.9	11.1	15.4	-	-	-
1640500	Owens Creek at Lantz, MD	0.2	0.5	0.4	1.1	-	-	-
1640700	Owens Creek tributary near Rocky Ridge, MD	0	0	0	0.2	-	-	-
1640965	Hunting Creek near Foxville, MD	0	0	0.8	0.4	-	-	-
1640970	Hunting Creek tributary near Foxville, MD	1.1	1.2	1.1	1.2	-	-	-
1641000	Hunting Creek at Jimtown, MD	2.1	1.8	2.3	4.3	-	-	-
1641500	Fishing Creek near Lewistown, MD	0	0	0.2	0.2	0.2	0.5	0.7
1642000	Monocacy River near Frederick, MD	0.9	1.7	2.2	2.8	-	-	-
1642400	Dollyhyde Creek at Libertytown, MD	0	0.1	0.5	1.4	-	-	-
1642500	Lingamore Creek near Frederick, MD	0.4	1.3	2.6	3.9	-	-	-
1643000	Monocacy River at Jug Bridge near Frederick, MD	1.4	2.4	3.1	4.2	4.9	4.8	4.7
1643395	Soper Branch at Hyattstown, MD	-	-	-	1.9	1.9	1.9	1.5
1643500	Bennett Creek at Park Mills, MD	1.7	2	2.6	4	4.7	4.8	6.4
1644371	Little Seneca Creek tributary near Clarksburg, MD	-	-	-	3.5	3.5	3.8	28
1644375	Little Seneca Creek tributary near Germantown, MD	-	-	-	33.4	50.7	51.1	53.5
1644380	Cabin Branch near Boyd, MD	-	-	-	1.1	1.1	1.2	1.5
1644420	Bucklodge Branch tributary near Barnesville, MD	0	0	0	0	-	-	-
1644600	Great Seneca Creek near Quince Orchard, MD	-	-	-	22.4	23.1	21.4	25.5
1645000	Seneca Creek near Dawsonville, MD	4.4	8.3	11.6	15	16.3	15.4	18.8
1645200	Watts Branch at Rockville, MD	27.2	26.2	30.4	31.6	-	-	-
1646550	Little Falls Branch near Bethesda, MD	46.3	32.4	33.6	35.3	-	-	-
1647720	North Branch Rock Creek near Norbeck, MD	6.6	9.9	14.3	15.9	-	-	-

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
1649500	North East Branch Anacostia River at Riverdale, MD	78	73.2	0.055	0	9.3	26.5	24.7	39	W
1650050	Northwest Branch Anacostia River at Norwood, MD	10	2.51	0.05	0	0	78.1	9.1	12.6	P
1650085	Nursery Run at Cloverly, MD	10	0.35	0.08	0	0	80.8	18.2	1	P
1650190	Batchellors Run at Oakdale, MD	10	0.49	0.06	0	0	82.7	10.4	5.7	P
1650500	Northwest Branch Anacostia River near Colesville, MD	62	21.23	0.062	0	0	80.1	6.1	13.6	P
1651000	Northwest Branch Anacostia River near Hyattsville, MD	46	49.33	0.065	0	1.7	68	10.3	19.8	W/P
1653500	Henson Creek at Oxon Hill, MD	30	17.19	0.059	0	11.6	12.4	56	19.8	W
1653600	Piscataway Creek at Piscataway, MD	51	39.43	0.057	0	12.8	10.6	60.9	15.5	W
1658000	Mattawoman Creek near Pomonkey, MD	54	55.57	0.034	0	12	2.4	51.5	33.8	W
1660900	Wolf Den Branch near Cedarville, MD	14	2.31	0.029	0	15.8	3.7	57.3	22.8	W
1660920	Zekiah Swamp Run near Newtown, MD	33	81.61	0.044	0	33.3	2	44	20.6	W
1660930	Clark Run near Bel Alton, MD	11	11.27	0.049	0	24.9	0.8	63.7	10.5	W
1661000	Chaptico Creek at Chaptico, MD	25	10.23	0.069	0	5.8	27.9	55.6	10.7	W
1661050	St. Clements Creek near Clements, MD	48	18.18	0.059	0	6.7	31.2	48.6	13.5	W
1661430*	Glebe Branch at Valley Lee, MD	11	0.24	0.032	0	2	21.2	72.2	4.7	W
1661500	St. Marys River at Great Mills, MD	70	25.29	0.041	0	4.6	13	68.3	12.5	W
3075450	Little Youghiogheny River tributary at Deer Park, MD	12	0.55	0.064	0	0	12.5	55.8	31.7	A
3075500	Youghiogheny River near Oakland, MD	72	134.2	0.12635	0	0	14.7	69.3	15.3	A
3075600	Toliver Run tributary near Hoyes Run, MD	22	0.52	0.0798	0	0	22.4	74.7	3	A
3076500	Youghiogheny River at Friendsville, MD	89	294.1	0.11602	0	0	18.7	66.5	12.1	A

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	Station Name	IA70 (%)	IA85 (%)	IA90 (%)	IA97 (%)	IA00 (%)	IA02 (%)	IA10 (%)
1649500	North East Branch Anacostia River at Riverdale, MD	28.6	18.9	21.4	24.8	-	-	28.3
1650050	Northwest Branch Anacostia River at Norwood, MD	-	5.1	-	-	-	-	-
1650085	Nursery Run at Cloverly, MD	-	3.8	-	-	-	-	-
1650190	Batchellors Run at Oakdale, MD	0	5.4	14.6	6.7	-	-	-
1650500	Northwest Branch Anacostia River near Colesville, MD	10.9	11.6	16.9	21.7	22.1	22.2	23.8
1651000	Northwest Branch Anacostia River near Hyattsville, MD	27.2	22.3	25.1	30.7	30.5	28.4	30.3
1653500	Henson Creek at Oxon Hill, MD	40.9	26.5	28.2	34.8	-	35.9	37.2
1653600	Piscataway Creek at Piscataway, MD	17.5	7.7	9.9	11.6	-	14.3	17
1658000	Mattawoman Creek near Pomonkey, MD	7.8	5	7.1	10	-	11.7	15.3
1660900	Wolf Den Branch near Cedarville, MD	9.2	0	4.6	6.2	-	4.9	5.6
1660920	Zekiah Swamp Run near Newtown, MD	6.9	4	5.3	6.7	-	7.2	9.2
1660930	Clark Run near Bel Alton, MD	-	6.4	-	-	-	8.5	10.3
1661000	Chaptico Creek at Chaptico, MD	8.2	1.9	3.3	4.6	-	5	6
1661050	St. Clements Creek near Clements, MD	6	1.8	2.3	3.4	-	4.2	6
1661430*	Glebe Branch at Valley Lee, MD	31.4	2.1	8.4	9.2	-	2.7	5.4
1661500	St. Marys River at Great Mills, MD	5.4	4	6.1	9.4	-	11.9	14.9
3075450	Little Youghiogheny River tributary at Deer Park, MD	0	0.1	0.7	1.8	-	-	-
3075500	Youghiogheny River near Oakland, MD	1.3	1.6	2.5	3.7	4.5	4.5	3
3075600	Toliver Run tributary near Hoyes Run, MD	0	0	0	0	-	-	-
3076500	Youghiogheny River at Friendsville, MD	0.9	1.3	2.1	3.2	4.1	3.7	3.1

* Gaging station not used in regression analysis

	Column number:	1	2	3	4	5	6	7	8	9
Station Number	**Station Name**	**Years of Record**	**Area (mi^2)**	**Land Slope (ft/ft)**	**Lime (%)**	**Hyd. A (%)**	**Hyd. B (%)**	**Hyd. C (%)**	**Hyd. D (%)**	**Province**
3076505*	Youghiogheny River Tributary near Friendsville, MD	11	0.21	0.2	-	0	14	86	0	A
3076600	Bear Creek at Friendsville, MD	48	49.07	0.17541	0	0	34.5	62.7	2.7	A
3077700	North Branch Casselman River tributary at Foxtown, MD	11	1.07	0.05438	0	0	58.8	29	12.2	A
3078000	Casselman River at Grantsville, MD	65	63.77	0.10292	0	0	12.5	73.5	13.7	A

* Gaging station not used in regression analysis

	Column number:	10	11	12	13	14	15	16
Station Number	**Station Name**	**IA70 (%)**	**IA85 (%)**	**IA90 (%)**	**IA97 (%)**	**IA00 (%)**	**IA02 (%)**	**IA10 (%)**
3076505*	Youghiogheny River Tributary near Friendsville, MD	-	0	-	-	-	-	-
3076600	Bear Creek at Friendsville, MD	0.5	0.3	0.9	1.3	2.2	2.4	2.9
3077700	North Branch Casselman River tributary at Foxtown, MD	0	0	0	0.2	-	-	-
3078000	Casselman River at Grantsville, MD	0.2	0.8	0.8	1.4	1.6	1.7	2.2

* Gaging station not used in regression analysis

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APPENDIX 2
FLOOD FREQUENCY RESULTS
FOR USGS GAGES
IN MARYLAND AND DELAWARE

Station Number	Station Name	Years of Record	Discharge in ft³/s									
			Return Period (yr)									
			1.25	1.5	2	5	10	25	50	100	200	500
1483155◇*	Silver Lake Tributary at Middletown, DE	16	58	74	96	169	232	333	424	532	658	857
1483200	Blackbird Creek at Blackbird, DE	65	90	120	160	240	350	680	760	840	900	980
1483290◇	Paw Paw Branch Tributary near Clayton, DE	10	93	116	149	255	346	489	617	766	940	1,210
1483500	Leipsic River near Cheswold, DE	34	120	156	211	412	612	964	1,320	1,770	2,340	3,340
1483720	Puncheon Branch at Dover, DE	10	77	103	141	268	381	562	727	922	1,150	1,510
1484000	Murderkill River near Felton, DE	35	137	191	271	552	810	1,230	1,620	2,070	2,610	3,470
1484002*	Murderkill River Tributary near Felton, DE	10	11	15	22	51	81	137	196	273	372	550
1484050	Pratt Branch near Felton, DE	10	35	49	70	153	241	404	573	796	1,090	1,600
1484100	Beaverdam Branch at Houston, DE	60	34	43	56	95	128	179	224	276	335	428
1484270◇	Beaverdam Creek near Milton, DE	19	25	31	39	65	86	118	146	179	216	274
1484300	Sowbridge Branch near Milton, DE	22	25	29	36	56	72	96	118	143	171	215
1484500	Stockley Branch at Stockly, DE	62	76	88	111	171	219	290	352	420	497	859
1484550◇	Pepper Creek at Dagsboro, DE	16	166	204	254	400	511	669	800	942	1,100	1,320
1484695◇	Beaverdam Ditch near Millville, DE	19	57	72	94	160	215	295	365	442	529	660
148471320◇*	Birch Branch at Sowell, MD	18	418	495	598	887	1,110	1,420	1,680	1,960	2,270	2,720
1484719◇	Bassett Creek near Ironshire, MD	10	66	92	133	293	460	765	1,080	1,490	2,020	2,960
1485000	Pocomoke River near Willards, MD	66	494	589	717	1,100	1,410	1,860	2,260	2,700	3,200	3,960
1485500	Nassawango Creek near Snow Hill, MD	68	362	459	600	1,070	1,490	2,170	2,800	3,560	4,470	5,940
1486000	Manokin Branch near Princess Anne, MD	64	75	100	145	270	340	425	480	540	590	660
1486100*	Andrews Branch near Delmar, MD	10	64	78	95	143	179	230	271	316	363	432
1486980*	Toms Dam Branch near Greenwood DE	10	25	31	38	59	75	98	116	136	157	188
1487000	Nanticoke River near Bridgeville, DE	75	391	506	670	1,200	1,650	2,360	2,990	3,720	4,570	5,880
1487900	Meadow Branch near Delmar, DE	9	52	61	72	99	118	143	162	182	202	229
1488500	Marshyhope Creek near Adamsville, DE	45	970	1,300	1,650	2,400	2,900	3,400	3,800	4,200	4,500	5,000
1489000	Faulkner Branch near Federalsburg, MD	42	108	159	238	532	814	1,290	1,730	2,270	2,910	3,940
1490000	Chicamacomico River near Salem, MD	46	123	162	219	407	573	834	1,080	1,360	1,700	2,220

◇ New gaging station added since 2016 analysis.
* Gaging station not used in regression analysis.

Station Number	Station Name	Years of Record	Discharge in ft³/s Return Period (yr) 1.25	1.5	2	5	10	25	50	100	200	500
1490600	Meredith Branch Near Sandtown, DE	10	136	178	241	464	674	1,030	1,380	1,800	2,330	3,210
1490800	Oldtown Branch at Goldsboro, MD	10	111	139	178	301	403	558	695	851	1,030	1,300
1491000	Choptank River near Greensboro, MD	71	1,090	1,460	1,990	3,590	4,860	6,680	8,190	9,820	11,600	14,100
1491010	Sangston Prong near Whiteleysburg, DE	10	34	48	70	157	248	418	594	824	1,120	1,650
1491050	Spring Branch near Greensboro, MD	10	38	53	77	169	265	441	622	856	1,160	1,690
1491500◇	Tuckahoe Creek near Ruthsburg, MD	22	1,100	1,370	1,740	2,890	3,850	5,320	6,620	8,100	9,800	12,400
1492000	Beaverdam Branch at Matthews, MD	34	156	209	289	577	857	1,340	1,810	2,410	3,140	4,390
1492050	Gravel Run at Beulah, MD	11	56	73	98	186	267	404	535	695	889	1,210
1492500	Sallie Harris Creek near Carmicheal, MD	47	133	188	274	602	932	1,510	2,090	2,820	3,730	5,280
1492550	Mill Creek near Skipton, MD	11	70	94	132	273	412	657	901	1,210	1,600	2,260
1493000	Unicorn Branch near Millington, MD	69	198	264	360	668	929	1,330	1,680	2,080	2,530	3,220
1493112	Chesterville Branch near Crumpton, MD	13	99	151	241	640	1,110	2,040	3,080	4,510	6,450	10,100
1493500	Morgan Creek near Kennedyville, MD	66	192	272	405	976	1,640	2,980	4,490	6,600	9,540	15,200
1494000	Southeast Creek at Church Hill, DE	14	400	500	640	1,110	1,420	1,850	2,250	2,700	3,100	3,800
1494150◇	Three Bridges Branch at Centerville, MD	11	100	155	250	640	1,050	2,000	3,000	4,300	5,800	8,800
1495000	Big Elk Creek at Elk Mills, MD	80	1,790	2,250	2,890	4,850	6,450	8,860	10,950	13,300	16,000	20,000
1495500	Little Elk Creek at Childs, MD	12	1,320	1,440	1,650	2,440	3,230	4,650	6,100	7,990	10,400	14,800
1496000	Northeast River at Leslie, MD	37	1,010	1,220	1,540	2,530	3,400	4,760	6,010	7,480	9,220	12,000
1496080	Northeast River tributary near Charlestown, MD	10	120	199	258	408	514	654	760	872	986	1,140
1496200	Principio Creek near Principio Furnace, MD	27	617	808	1,090	2,120	3,110	4,820	6,490	8,590	11,200	15,600
1577940	Broad Creek tributary at Whiteford, MD	16	92	118	156	293	427	663	899	1,200	1,580	2,240
1578500	Octoraro Creek near Rising Sun, MD	19	2,490	3,280	4,480	8,920	13,400	21,200	29,200	39,500	52,500	75,300
1578800	Basin Run at West Nottingham, MD	10	261	320	397	581	700	847	949	1,060	1,160	1,280
1579000	Basin Run at Liberty Grove, MD	22	441	591	816	1,600	2,330	3,540	4,700	6,090	7,780	10,500
1580000	Deer Creek at Rocks, MD	86	2,430	2,950	3,660	5,700	7,290	9,580	11,500	13,600	15,900	19,400
1580200	Deer Creek at Kalmia, MD	11	2,890	3,580	4,550	7,610	10,200	14,200	17,800	21,900	26,700	34,200
1581500	Bynum Run at Bel Air, MD	38	756	962	1,250	2,140	2,870	3,960	4,910	5,980	7,180	9,000

◇ New gaging station added since 2016 analysis.

Station Number	Station Name	Years of Record	Discharge in ft³/s									
			Return Period (yr)									
			1.25	1.5	2	5	10	25	50	100	200	500
1581700	Winter Run near Benson, MD	45	1,270	1,830	2,600	4,800	6,360	8,340	9,790	11,200	12,600	14,300
1581752♦	Plumtree Creek near Bel Air, MD	11	276	365	502	1,010	1,530	2,440	3,370	4,560	6,090	8,740
1581810♦	Gunpowder Falls at Hoffmanville, MD	12	686	897	1,220	2,360	3,470	5,410	7,320	9,720	12,700	17,900
1581830♦*	Grave Run near Beckleysville, MD	13	168	226	313	618	905	1,380	1,840	2,400	3,070	4,180
1581870♦	Georges Run near Beckleysville, MD	13	531	690	930	1,810	2,670	4,190	5,700	7,630	10,100	14,300
1581940♦	Mingo Branch near Hereford, MD	10	36	53	83	225	495	730	1,140	1,700	2,500	4,100
1581960♦	Beetree Run at Bentley Springs, MD	13	491	618	794	1,320	1,740	2,370	2,900	3,490	4,150	5,140
1582000	Little Falls at Blue Mount, MD	69	1,500	1,820	2,270	3,570	4,600	6,100	7,380	8,790	10,400	12,700
1582510	Piney Creek near Hereford, MD	14	106	158	247	581	909	1,480	2,030	2,710	3,520	4,840
1583000*	Slade Run near Glyndon, MD	36	97	119	150	244	320	434	534	646	774	968
1583100	Piney Run at Dover, MD	23	524	636	796	1,310	1,760	2,460	3,110	3,880	4,780	6,230
1583495	Western Run tributary at Western Run, MD	10	49	69	99	203	295	441	571	726	906	1,180
1583500	Western Run at Western Run, MD	68	1,240	1,630	2,210	4,330	6,420	10,100	13,800	18,500	24,400	34,800
1583570*	Pond Branch at Oregon Ridge, MD	17	2.4	3.3	4.6	9.5	14	22	31	41	53	74
1583580	Baisman Run at Broadmoor, MD	26	45	68	107	268	443	768	1,110	1,550	2,110	3,100
1583600*	Beaverdam Run at Cockeysville, MD	29	787	938	1,140	1,700	2,130	2,730	3,220	3,760	4,340	5,190
158397967♦	Minebank Run near Glen Arm, MD	11	500	625	789	1,260	1,610	2,110	2,520	2,960	3,430	4,100
1584050	Long Green Creek at Glen Arm, MD	37	310	441	645	1,400	2,150	3,440	4,690	6,240	8,140	11,300
1584500	Little Gunpowder Falls at Laurel Brook, MD	72	1,460	1,930	2,610	4,790	6,630	9,440	11,900	14,700	17,900	22,700
1585090	Whitemarsh Run near Fullerton, MD	18	704	844	1,020	1,480	1,790	2,200	2,520	2,840	3,170	3,620
1585095♦	Nork Fork Whitemarsh Run near White Marsh, MD	17	320	340	405	680	980	1,500	2,050	2,700	3,600	5,000
1585100	White Marsh Run at White Marsh, MD	40	1,140	1,370	1,690	2,670	3,490	4,740	5,840	7,100	8,550	10,800
1585104♦	Honeygo Run near White Marsh, MD	13	337	415	521	838	1,090	1,470	1,790	2,140	2,540	3,140
1585200	West Branch Herring Run at Idlewylde, MD	46	421	559	749	1,300	1,720	2,310	2,770	3,270	3,780	4,520
1585225♦	Moores Run Tributary near Todd Ave at Baltimore, MD	16	134	142	156	210	260	333	400	475	560	680

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Station Number	Station Name	Years of Record	Discharge in ft^3/s Return Period (yr) 1.25	1.5	2	5	10	25	50	100	200	500
1585230♦	Moores Run at Radecke Ave at Baltimore, MD	16	1,400	1,680	2,040	3,030	3,760	4,760	5,560	6,410	7,320	8,630
1585300	Stemmers Run at Rossville, MD	29	790	991	1,260	2,080	2,720	3,660	4,450	5,330	6,290	7,720
1585400	Brien Run at Stemmers Run, MD	29	188	237	316	633	984	1,680	2,450	3,530	5,030	7,930
1585500	Cranberry Branch near Westminster, MD	64	117	166	243	538	836	1,370	1,900	2,570	3,410	4,860
1586000	North Branch Patapsco River at Cedarhurst, MD	67	1,520	1,850	2,360	4,100	5,750	8,560	11,300	14,700	19,000	26,300
1586210*	Beaver Run near Finksburg, MD	30	451	572	739	1,240	1,650	2,250	2,760	3,320	3,960	4,900
1586610	Morgan Run near Louisville, MD	30	711	928	1,240	2,230	3,070	4,360	5,510	6,810	8,310	10,600
1587000	North Branch Patapsco River near Marriottsville, MD	24	2,270	2,840	3,660	6,360	8,770	12,600	16,300	20,600	25,700	34,000
1587050	Hay Meadow Branch tributary at Poplar Springs, MD	11	65	86	116	212	296	430	518	702	872	1,140
1587500	South Branch Patapsco River at Henryton, MD	32	1,520	1,990	2,720	5,510	8,380	13,600	19,100	26,200	35,500	52,100
1588000	Piney Run near Sykesville, MD	43	332	463	674	1,520	2,440	4,160	6,000	8,440	11,700	17,500
1589000	Patapsco River at Hollofield, MD	23	6,120	7,920	10,500	18,800	26,000	37,300	47,400	59,100	72,700	93,900
1589100	East Branch Herbert Run at Arbutus, MD	47	465	540	645	986	1,280	1,760	2,190	2,710	3,320	4,320
1589180♦	Gwynns Falls at Glyndon, MD	14	58	66	85	175	265	430	630	880	1,200	1,800
1589197♦	Gwynns Falls near Delight, MD	14	495	548	636	995	1,380	2,110	2,900	3,980	5,440	8,230
1589200	Gwynns Falls near Owings Mills, MD	17	147	190	262	596	1,020	1,970	3,160	5,010	7,850	14,000
1589238*	Gwynns Falls Tributary at McDonough, MD	13	1.5	2.8	5.7	27	66	184	371	717	1,340	2,940
1589240	Gwynns Falls at McDonough, MD	12	599	787	1,080	2,210	3,400	5,600	7,910	11,000	15,000	22,300
1589300	Gwynns Falls at Villa Nova, MD	34	1,310	1,580	2,000	3,640	5,360	8,610	12,100	16,800	23,200	35,100
1589330	Dead Run at Franklintown, MD	31	1,260	1,490	1,830	2,980	4,040	5,820	7,540	9,670	12,300	16,700
1589352♦	Gwynns Falls at Washington Blvd at Baltimore, MD	14	4,730	5,920	7,580	12,900	17,400	24,400	30,700	38,000	46,500	59,700
1589440	Jones Fall at Sorrento, MD	47	636	830	1,150	2,500	4,080	7,320	11,100	16,500	24,200	39,700
1589464	Stony Run at Ridgemede Road at Baltimore, MD	9	420	538	703	1,200	1,610	2,210	2,730	3,310	3,950	4,910
1589500	Sawmill Creek at Glen Burnie, MD	34	69	81	101	184	280	475	703	1,030	1,510	2,480

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Station Number	Station Name	Years of Record	Discharge in ft³/s									
			Return Period (yr)									
			1.25	1.5	2	5	10	25	50	100	200	500
1589795	South Fork Jabez Branch at Millersville, MD	22	51	78	122	300	486	823	1,160	1,590	2,140	3,050
1590000	North River near Annapolis, MD	43	92	102	122	231	385	767	1,300	2,220	3,800	7,760
1590500	Bacon Ridge Branch at Chesterfield, MD	35	112	149	204	396	576	879	1,170	1,520	1,950	2,660
1591000	Patuxent River near Unity, MD	68	768	1,050	1,510	3,310	5,230	8,840	12,700	17,700	24,400	36,500
1591400	Cattail Creek near Glenwood, MD	46	669	846	1,100	1,960	2,720	3,960	5,110	6,490	8,140	10,800
1591700	Hawlings River near Sandy Springs, MD	34	652	895	1,260	2,530	3,700	5,610	7,390	9,510	12,000	16,000
1592000*	Patuxent River near Burtonsville, MD	32	1,770	2,100	2,560	3,950	5,080	6,780	8,260	9,950	11,900	14,800
1593350	Little Patuxent River tributary at Guilford Downs, MD	11	94	130	185	382	572	896	1,210	1,600	2,070	2,850
1593500*	Little Patuxent River at Guilford, MD	80	892	1,100	1,440	2,620	3,780	5,820	7,880	10,500	13,900	19,700
1594000	Little Patuxent River at Savage, MD	59	2,090	2,660	3,500	6,370	9,000	13,400	17,500	22,500	28,600	38,600
1594400*	Dorsey Run near Jessup, MD	20	326	379	459	750	1,030	1,530	2,040	2,690	3,520	5,000
1594440	Patuxent River near Bowie, MD	41	3,880	4,900	6,370	10,800	14,400	18,300	22,700	29,400	35,100	43,800
1594445	Mill Branch near Mitchellville, MD	11	73	99	137	270	394	598	790	1,020	1,300	1,750
1594500*	Western Branch near Largo, MD	25	601	724	880	1,300	1,590	1,980	2,280	2,600	2,920	3,370
1594526	Western Branch at Upper Marlboro, MD	29	2,480	3,000	3,810	6,265	8,350	11,600	14,500	17,900	21,800	28,000
1594600	Cocktown Creek near Huntington, MD	19	71	99	145	331	534	923	1,340	1,910	2,660	4,040
1594670	Hunting Creek near Huntingtown, MD	10	149	193	255	450	613	860	1,080	1,320	1,600	2,020
1594710*	Killpeck Creek at Huntersville, MD	12	123	139	159	209	243	287	321	356	392	441
1594800	St. Leonard Creek near St. Leonard, MD	14	62	77	98	159	208	282	345	416	496	616
1594930	Laurel Run at Dobbin Road near Wilson, MD	26	253	308	381	581	729	934	1,100	1,270	1,460	1,730
1594936	North Fork Sand Run near Wilson, MD	28	69	92	127	258	389	624	861	1,170	1,550	2,230
1594950	McMillan Fork near Fort Pendleton, MD	25	76	98	130	242	345	517	681	880	1,120	1,530
1596005	Savage River near Frostburg, MD	14	20	39	50	82	107	144	177	212	252	312
1596500	Savage River near Barton, MD	64	1,060	1,250	1,520	2,380	3,100	4,220	5,240	6,420	7,800	10,000
1597000	Crabtree Creek near Swanton, MD	33	305	378	485	844	1,170	1,720	2,230	2,860	3,630	4,900
1598000	Savage River at Bloomington, MD	24	2,180	2,710	3,450	5,880	8,030	11,500	14,600	18,400	22,900	30,100
1599000	Georges Creek at Franklin, MD	82	1,270	1,520	1,880	2,970	3,890	5,310	6,580	8,040	9,730	12,400

* Gaging station not used in regression analysis.

Station Number	Station Name	Years of Record	Discharge in ft³/s Return Period (yr) 1.25	1.5	2	5	10	25	50	100	200	500
1601500	Wills Creek near Cumberland, MD	83	4,140	4,900	6,040	10,100	14,000	20,900	27,800	36,600	47,800	67,500
1609000	Town Creek near Oldtown, MD	33	2,490	3,120	3,970	6,510	8,540	11,500	14,000	16,800	19,900	24,500
1609500	Sawpit Run near Oldtown, MD	25	190	223	267	398	502	657	789	938	1,110	1,360
1610105	Pratt Hollow Tributary at Pratt, MD	15	41	46	54	74	88	107	121	137	153	176
1610150	Bear Creek at Forest Park, MD	18	219	283	375	666	912	1,290	1,620	2,000	2,440	3,110
1610155	Sideling Hill Creek near Bellegrove, MD	24	2,180	2,880	3,860	6,930	9,460	13,200	16,400	20,000	24,100	30,000
1612500	Little Tonoloway Creek near Hancock, MD	17	315	399	518	896	1,220	1,720	2,160	2,680	3,280	4,210
1613150	Ditch Run near Hancock, MD	22	155	189	236	376	489	656	800	960	1,140	1,410
1613160	Potomac River Tributary near Hancock, MD	12	60	74	94	151	197	263	319	380	448	550
1614500	Conococheague Creek at Fairview, MD	85	5,380	6,340	7,620	11,400	14,400	18,700	22,400	26,600	31,200	38,200
1617800*	Marsh Run at Grimes, MD	48	58	76	102	187	262	382	490	618	768	1,000
1619000	Antietam Creek near Waynesboro, PA	27	986	1,210	1,540	2,570	3,460	4,880	6,160	7,670	9,450	12,300
1619475	Dog Creek tributary near Locust Grove, MD	11	11	15	21	44	68	109	152	207	276	398
1619500	Antietam Creek near Sharpsburg, MD	85	1,580	2,020	2,620	4,520	6,100	8,520	10,600	13,100	15,800	20,100
1637000	Little Catoctin Creek at Harmony, MD	30	268	387	584	1,410	2,320	4,080	5,980	8,530	11,900	18,100
1637500	Catoctin Creek near Middletown, MD	65	1,470	1,900	2,510	4,540	6,320	9,140	11,700	14,700	18,300	23,900
1637600	Hollow Road Creek near Middletown, MD	11	141	192	274	600	949	1,610	2,310	3,240	4,480	6,730
1639000	Monocacy River at Bridgeport, MD	72	6,690	7,580	8,740	12,000	14,400	17,800	20,600	23,600	26,900	31,700
1639095*	Piney Creek tributary at Taneytown, MD	10	76	97	126	218	296	416	522	644	784	1,000
1639140♦	Piney Creek near Taneytown, MD	12	1,310	1,550	1,890	2,940	3,820	5,180	6,390	7,800	9,430	12,000
1639500	Big Pipe Creek at Bruceville, MD	65	2,250	2,700	3,360	5,560	7,550	10,900	14,000	17,800	22,400	30,100
1640000	Little Pipe Creek at Bruceville, MD	31	228	306	424	857	1,280	2,020	2,760	3,680	4,830	6,800
1640500	Owens Creek at Lantz, MD	53	179	255	378	876	1,410	2,420	3,490	4,910	6,760	10,100
1640700	Owens Creek tributary near Rocky Ridge, MD	11	98	128	175	329	469	700	916	1,190	1,520	2,040
1640965	Hunting Creek near Foxville, MD	13	59	80	115	251	392	653	925	1,280	1,740	2,570

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Station Number	Station Name	Years of Record	Discharge in ft³/s Return Period (yr) 1.25	1.5	2	5	10	25	50	100	200	500
1640970	Hunting Creek tributary near Foxville, MD	10	146	213	325	796	1,320	2,340	3,440	4,920	6,900	10,500
1641000	Hunting Creek at Jimtown, MD	43	482	624	821	1,400	1,860	2,520	3,060	3,650	4,300	5,230
1641500	Fishing Creek near Lewistown, MD	39	71	100	148	346	568	1,000	1,480	2,130	3,020	4,670
1642000	Monocacy River near Frederick, MD	35	13,000	14,800	16,900	22,600	26,500	31,700	35,700	39,900	44,300	50,400
1642400	Dollyhyde Creek at Libertytown, MD	10	223	295	405	740	1,020	1,450	1,810	2,240	2,710	3,420
1642500	Lingamore Creek near Frederick, MD	49	1,600	1,960	2,480	4,150	5,600	7,920	10,000	12,600	15,500	20,300
1643000	Monocacy River at Jug Bridge near Frederick, MD	84	13,900	15,900	18,600	26,600	33,000	42,400	50,400	59,300	69,300	84,500
1643395♦	Soper Branch at Hyattstown, MD	9	46	68	105	266	449	811	1,210	1,750	2,490	3,850
1643500	Bennett Creek at Park Mills, MD	62	1,500	1,900	2,520	4,780	7,060	11,200	15,400	20,900	28,000	40,600
1644371♦	Little Seneca Creek Tributary near Clarksburg, MD	9	87	106	134	241	347	538	734	990	1,320	1,920
1644375♦	Little Seneca Creek Tributary near Germantown, MD	9	93	128	184	411	660	1,140	1,660	2,360	3,310	5,060
1644380♦	Cabin Branch near Boyd, MD	9	45	88	175	530	830	1,320	1,800	2,300	2,900	3,850
1644420	Bucklodge Branch tributary near Barnesville, MD	10	51	66	89	157	212	293	362	442	529	655
1644600♦	Great Seneca Creek near Quince Orchard, MD	12	1,720	2,100	2,600	4,400	5,900	8,400	10,800	13,600	17,400	23,100
1645000	Seneca Creek near Dawsonville, MD	48	2,340	3,010	4,050	7,980	12,000	19,400	27,100	37,300	50,500	74,400
1645200	Watts Branch at Rockville, MD	30	341	454	622	1,210	1,760	2,680	3,560	4,620	5,910	8,040
1646550	Little Falls Branch near Bethesda, MD	40	492	657	887	1,570	2,110	2,860	3,480	4,140	4,850	5,860
1647720	North Branch Rock Creek near Norbeck, MD	11	434	589	842	1,770	2,700	4,380	6,060	8,300	11,100	15,900
1649500	North East Branch Anacostia River at Riverdale, MD	78	5,090	6,000	7,350	10,300	12,200	14,500	16,100	17,700	19,200	21,100
1650050	Northwest Branch Anacostia River at Norwood, MD	10	280	391	568	1,200	1,790	2,810	3,780	5,000	6,500	8,890
1650085	Nursery Run at Cloverly, MD	10	43	65	101	247	404	699	1,000	1,410	1,930	2,840
1650190	Batchellors Run at Oakdale, MD	10	90	120	167	315	448	658	852	1,090	1,360	1,790
1650500	Northwest Branch Anacostia River near Colesville, MD	75	829	1,000	1,280	2,320	3,400	5,400	7,530	10,400	14,200	21,200
1651000	Northwest Branch Anacostia River near Hyattsville, MD	46	2,760	3,460	4,450	7,570	10,200	14,250	17,800	22,000	26,800	34,200

♦ New gaging station added between 2010 and 2016 analysis.

Station Number	Station Name	Years of Record	Discharge in ft³/s — Return Period (yr) 1.25	1.5	2	5	10	25	50	100	200	500
1653500	Henson Creek at Oxon Hill, MD	30	756	952	1,220	2,010	2,630	3,520	4,270	5,090	5,990	7,310
1653600	Piscataways Creek at Piscataway, MD	51	650	840	990	2,200	5,300	7,400	8,700	10,000	11,000	12,500
1658000	Mattawoman Creek near Pomonkey, MD	54	630	877	1,260	2,650	3,990	6,280	8,500	11,200	14,500	20,000
1660900	Wolf Den Branch near Cedarville, MD	14	70	92	128	258	388	617	847	1,140	1,510	2,160
1660920	Zekiah Swamp Run near Newtown, MD	33	782	1,040	1,440	2,880	4,310	6,820	9,320	12,500	16,500	23,300
1660930	Clark Run near Bel Alton, MD	11	240	312	430	954	1,560	2,810	4,280	6,470	9,650	16,100
1661000	Chaptico Creek at Chaptico, MD	25	195	260	362	763	1,190	1,980	2,830	3,950	5,440	8,160
1661050	St. Clements Creek near Clements, MD	48	325	466	697	1,650	2,700	4,700	6,840	9,720	13,500	20,500
1661430*	Glebe Branch at Valley Lee, MD	11	16	20	26	46	64	92	117	148	184	241
1661500	St. Marys River at Great Mills, MD	70	481	653	923	1,960	3,020	4,970	6,970	9,570	12,900	18,900
3075450	Little Youghiogheny River tributary at Deer Park, MD	12	20	23	28	41	51	68	74	91	105	140
3075500	Youghiogheny River near Oakland, MD	72	2,910	3,490	4,280	6,660	8,580	11,400	13,900	16,700	19,800	24,600
3075600	Toliver Run tributary near Hoyes Run, MD	22	18	23	30	54	75	111	144	184	232	310
3076500	Youghiogheny River at Friendsville, MD	89	4,570	5,360	6,350	8,920	10,700	13,100	14,900	16,800	18,700	21,400
3076505*	Youghiogheny River Tributary near Friendsville, MD	12	9.4	10	12	16	18	21	23	26	28	31
3076600	Bear Creek at Friendsville, MD	48	1,150	1,370	1,640	2,040	2,340	3,600	4,800	5,400	5,800	6,400
3077700	North Branch Casselman River tributary at Foxtown, MD	12	18	25	36	78	145	220	320	450	640	1,000
3078000	Casselman River at Grantsville, MD	65	1,500	1,730	2,040	3,000	3,690	4,780	5,710	6,750	7,930	9,720

* Gaging station not used in regression analysis.

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APPENDIX 3
FIXED REGION REGRESSION EQUATIONS
FOR MARYLAND

The regression equations described in this appendix are based on flood discharges documented in the following reports:

- Regression Equations for Estimating Flood Discharges for the Piedmont, Blue Ridge and Appalachian Plateau Regions in Western Maryland (Thomas and Moglen, 2016) that used annual peak flow data through the 2012 water year,
- Regression Equations for Estimating Flood Discharges for the Eastern Coastal Plain Region of Maryland (Thomas and Sanchez-Claros, 2019a) that used annual peak data through the 2017 water year,
- Regression Equations for Estimating Flood Discharges for the Western Coastal Plain Region of Maryland (Thomas and Sanchez-Claros, 2019b) that used annual peak data through the 2017 water year, and
- Revisions to the equations in Thomas and Sanchez-Claros (2019a; 2019b) as described herein.

Minor revisions were made to the regression equations developed by Thomas and Moglen (2016), as described herein. Therefore, regression equations were updated for all hydrologic regions — Appalachian Plateau, Piedmont-Blue Ridge, Western Coastal Plain and Eastern Coastal Plain — since the publication of the regression equations in the July 2016 Hydrology Panel report. Revisions were made to the Eastern and Western Coastal Plain Regions since publication of the July 2020 Hydrology Panel report, as described herein. The regression equations documented in this report supersede previous regression equations published by the Maryland Hydrology Panel.

The regression equations documented in Thomas and Sanchez-Claros (2019a; 2019b) for the Eastern Coastal Plain (ECP) and Western Coastal Plain (WCP) Regions, respectively, and included in the July 2020 version of the Hydrology Panel report were based on SSURGO soils that used the Dominant Component approach for aggregating the hydrologic soils data. The explanatory variable in the 2020 regression equations that represents soil conditions is the percent of Hydrologic Soil Group A (highest infiltration soils). On August 4, 2020, extreme flooding from heavy rainfall from Tropical Storm Isaias washed out the bridge on MD 6 over Persimmon Creek in St. Mary's County in the Western Coastal Plain Region. The Maryland Department of Transportation State Highway Administration (MDOT SHA) performed hydrologic analyses on Persimmon Creek as part of designing a new bridge for MD 6. During this analysis, it was determined that there was a significant difference in the percent A and B soils based on the May 2018 SSURGO soils data in GISHydroNXT (based on Dominant Component) and the current (2020) default data in the NRCS Soil Survey web site (based on Dominant Condition). Apparently between 2018 and 2020, NRCS changed the default option for aggregating the soils data on the Soil Survey web site. The definition of each aggregation approach from the NRCS Soil Survey website is as follows.

Table A-4. Aggregation Methods [These methods determine the attribute value for thematic maps of soil properties and interpretative ratings in WSS.]	
Method	**Description**
Dominant Condition	Groups components in a map unit based on like-values for the attribute. For each group, percent composition becomes the sum of the percent composition of all components in the group. These groups therefore represent conditions rather than components. If more than one group shares the highest percent composition, a corresponding tie-breaker rule determines which value is returned.
Dominant Component	Returns the attribute value associated with the component that has the highest percent composition in the map unit. If more than one component shares the highest percent composition, a corresponding tie-breaker rule determines which value is returned.

MDOT SHA contacted Mr. Philip King, who is the NRCS soil scientist for Maryland and Delaware, and asked his opinion about what aggregation method is best for engineering studies. Mr. King responded that the Dominant Condition approach was best for engineering studies (written communication, August 29, 2020). For Persimmon Creek in St. Mary's County, the percent A soil was 61.4 percent for the 2018 soils data in GISHydroNXT (Dominant Component) and 3.6 percent for the 2020 NRCS Soil Survey data (Dominant Condition). The runoff curve numbers were 49.2 and 61.4 for the Dominant Component and Dominant Condition, respectively. The resultant T-year flood discharges were more reasonable for Persimmon Creek using the Dominant Condition soils data and an evaluation was undertaken to determine what SSURGO soils data were best for the ECP and WCP Regions.

New SSURGO soils data were downloaded in October 2021 for the WCP and ECP Regions using the Dominant Condition approach and the 2020 regression equations were revised using a new value of percent A soils. All other data remained the same from the analyses documented by Thomas and Sanchez-Claros (2019a; 2019b). The standard errors of the WCP equations decreased significantly using the Dominant Condition soils data with the most improvement for watersheds in St. Mary's County (consistent with the MDOT SHA findings for Persimmon Creek). The standard errors increased slightly for the ECP Region when using the Dominant Condition data. Because of the significant improvement in the WCP Region, regression equations were revised for both regions. Details on this analysis are provided later in this section.

A summary of the regression equations is provided for all four hydrologic regions for easy access and reference and then a more detailed description of the development of the equations.

Fixed Region Regression Equations for Rural Watersheds in the Eastern Coastal Plain Region

The following equations are based on 35 rural gaging stations, 19 in Maryland and 16 in Delaware, with drainage area (DA) ranging from 0.91 to 113.8 square miles, percent A soils (Acond) ranging from 0.5 to 82.7 percent based on the October 2021 SSURGO soils data, and land slope (LSLOPE) ranging from 0.00463 to 0.0220 ft/ft or 0.463 to 2.2 percent. The regression equations were developed using LSLOPE in percent **so percent must be used in application of the equations.** The land slope data are based on the May 2018 DEM data, the most recent data at the time of the 2020 regression analysis. DA and LSLOPE were transformed to logarithms in the regression analysis but Acond was more significant when not transforming to logarithms and therefore Acond appears in the equations to the power 10. All variables are statistically significant at the 5-percent level of significance except that LSLOPE is only statistically significant for the 5-year flood and larger. The 5-percent level of significance means there is only a 5 percent chance of erroneously including a variable in the equation when it is not actually statistically significant. The equations, standard error of estimate in percent, and equivalent years of record are as follows:

Eastern Coastal Plain Region Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 35.3\ DA^{0.763}\ LSLOPE^{0.120}\ 10^{-0.00815 * Acond}$	46.0	2.8
$Q_{1.5} = 47.7\ DA^{0.762}\ LSLOPE^{0.195}\ 10^{-0.009 * Acond}$	44.1	3.0
$Q_{2} = 67.1\ DA^{0.754}\ LSLOPE^{0.276}\ 10^{-0.00921 * Acond}$	42.3	3.2
$Q_{5} = 135.6\ DA^{0.738}\ LSLOPE^{0.470}\ 10^{-0.01032 * Acond}$	40.0	6.8
$Q_{10} = 202.4\ DA^{0.726}\ LSLOPE^{0.603}\ 10^{-0.01098 * Acond}$	39.2	10
$Q_{25} = 321.4\ DA^{0.705}\ LSLOPE^{0.795}\ 10^{-0.01161 * Acond}$	39.6	19
$Q_{50} = 432.1\ DA^{0.696}\ LSLOPE^{0.898}\ 10^{-0.01214 * Acond}$	40.4	19
$Q_{100} = 569.0\ DA^{0.687}\ LSLOPE^{0.996}\ 10^{-0.01263 * Acond}$	42.1	21
$Q_{200} = 736.0\ DA^{0.679}\ LSLOPE^{1.082}\ 10^{-0.01312 * Acond}$	44.4	23
$Q_{500} = 1033.8\ DA^{0.663}\ LSLOPE^{1.185}\ 10^{-0.01365 * Acond}$	47.9	24

Fixed Region Regression Equations for Rural and Urban Watersheds in the Western Coastal Plain Region

The following equations are based on 23 gaging stations (13 rural and 10 urban) in the Western Coastal Plain region of Maryland with drainage area (DA) ranging from 0.96 to 350.21 square miles, impervious area (IA) ranging from 0.0 to 36.8 percent, and the percent A soils (Acond) ranging from 1.7 to 85.2 percent based on the October 2021 SSURGO soils data.

All explanatory variables in the following equations are statistically significant at the 5-percent level of significance for all recurrence intervals. This implies there is only a 5 percent chance of erroneously including a variable in the equation when it is not actually statistically significant. DA and IA were transformed to logarithms in the regression analysis but Acond was more significant when not transforming to logarithms and therefore Acond appears in the equations to the power 10. The equations, standard error of estimate in percent, and equivalent years of record are as follows:

Western Coastal Plain Region Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 33.0\ DA^{0.709}\ (IA+1)^{0.389}\ 10^{-0.00734 * Acond}$	50.8	2.3
$Q_{1.5} = 46.7\ DA^{0.696}\ (IA+1)^{0.374}\ 10^{-0.00778 * Acond}$	49.2	2.4
$Q_{2} = 69.0\ DA^{0.680}\ (IA+1)^{0.357}\ 10^{-0.00819 * Acond}$	48.4	2.5
$Q_{5} = 164.1\ DA^{0.645}\ (IA+1)^{0.321}\ 10^{-0.00908 * Acond}$	40.8	6.4
$Q_{10} = 272.0\ DA^{0.630}\ (IA+1)^{0.303}\ 10^{-0.00960 * Acond}$	34.7	13
$Q_{25} = 493.2\ DA^{0.592}\ (IA+1)^{0.279}\ 10^{-0.00994 * Acond}$	29.2	28
$Q_{50} = 736.9\ DA^{0.567}\ (IA+1)^{0.262}\ 10^{-0.01010 * Acond}$	27.0	43
$Q_{100} = 1065.3\ DA^{0.547}\ (IA+1)^{0.248}\ 10^{-0.01022 * Acond}$	28.0	50
$Q_{200} = 1529.3\ DA^{0.521}\ (IA+1)^{0.234}\ 10^{-0.01030 * Acond}$	32.6	45
$Q_{500} = 2418.7\ DA^{0.489}\ (IA+1)^{0.215}\ 10^{-0.01041 * Acond}$	42.7	34

Fixed Region Regression Equations for Rural and Urban Watersheds in the Piedmont-Blue Ridge Region

The following equations are based on 64 rural and 32 urban stations in the combined Piedmont and Blue Ridge Regions with drainage area (DA) ranging from 0.11 to 816.4 square miles, percentage of carbonate/limestone rock (LIME) ranging from 0.0 to 81.7 percent, and percentage of impervious area (IA) ranging 0.0 to 53.5 percent. An impervious area of greater than 10 percent was used to classify watersheds as urban. The impervious area near the mid-point of the gaging station record was used in the development of the regression. For estimation at ungaged locations, the most recent impervious area should be used. There were 10 stations identified as outliers and not used in developing the regression equations. Both rural and urban watersheds were included in the same analysis to avoid any discontinuities in estimates in transitioning from rural to urban watersheds. The equations, the standard error of estimate in percent, and the equivalent years of record are as follows:

Piedmont-Blue Ridge Region Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 63.0\ DA^{0.685}\ (LIME+1)^{-0.090}\ (IA+1)^{0.284}$	53.1	2.0
$Q_{1.50} = 89.8\ DA^{0.669}\ (LIME+1)^{-0.100}\ (IA+1)^{0.253}$	48.3	2.4
$Q_{2} = 131.7\ DA^{0.653}\ (LIME+1)^{-0.112}\ (IA+1)^{0.225}$	43.6	2.8
$Q_{5} = 283.7\ DA^{0.625}\ (LIME+1)^{-0.136}\ (IA+1)^{0.184}$	35.2	8.3
$Q_{10} = 434.7\ DA^{0.610}\ (LIME+1)^{-0.148}\ (IA+1)^{0.166}$	31.6	14
$Q_{25} = 683.3\ DA^{0.599}\ (LIME+1)^{-0.164}\ (IA+1)^{0.153}$	30.0	24
$Q_{50} = 929.3\ DA^{0.591}\ (LIME+1)^{-0.174}\ (IA+1)^{0.145}$	30.8	29
$Q_{100} = 1{,}240.1\ DA^{0.584}\ (LIME+1)^{-0.184}\ (IA+1)^{0.139}$	33.0	32
$Q_{200} = 1{,}616.8\ DA^{0.578}\ (LIME+1)^{-0.193}\ (IA+1)^{0.134}$	36.6	31
$Q_{500} = 2{,}252.2\ DA^{0.571}\ (LIME+1)^{-0.205}\ (IA+1)^{0.129}$	42.9	29

Fixed Region Regression Equations for Rural Watersheds in the Appalachian Plateau Region

The regression equations for the Appalachian Plateau Region are based on 24 rural gaging stations in Maryland with drainage area (DA) ranging from 0.52 to 294.14 square miles and land slope (LSLOPE) ranging from 0.06400 to 0.25265 ft/ft. The land slope data are based on the May 2018 DEM data, the most recent data being used to estimate watershed characteristics in 2020. One station, 03076505, was an outlier and eliminated from the regression analysis. LSLOPE is only statistically significant at the 5-percent level up to the 5-year flood but was retained in the equations for the larger floods for consistency. LSOPE does reduce the standard error somewhat even though not always statistically significant at the 5-percent level. The equations, standard error of estimate in percent, and equivalent years of record are as follows:

Appalachian Plateau Region Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 79.4\ DA^{0.840}\ LSLOPE^{0.397}$	29.2	1.3
$Q_{1.50} = 92.4\ DA^{0.831}\ LSLOPE^{0.348}$	21.8	4.4
$Q_{2} = 115.2\ DA^{0.825}\ LSLOPE^{0.333}$	19.9	7.5
$Q_{5} = 183.4\ DA^{0.813}\ LSLOPE^{0.306}$	20.7	11
$Q_{10} = 221.2\ DA^{0.808}\ LSLOPE^{0.248}$	24.9	12
$Q_{25} = 317.6\ DA^{0.803}\ LSLOPE^{0.261}$	28.7	13
$Q_{50} = 397.6\ DA^{0.803}\ LSLOPE^{0.263}$	33.6	13
$Q_{100} = 474.5\ DA^{0.799}\ LSLOPE^{0.244}$	38.3	12
$Q_{200} = 559.4\ DA^{0.795}\ LSLOPE^{0.227}$	44.0	11
$Q_{500} = 664.0\ DA^{0.790}\ LSLOPE^{0.183}$	51.3	10

Regional Regression Equations for Maryland Streams as of 2022

Overview of the Current Regression Equations

Chapter 2 **Statistical Methods for Estimating Flood Discharges** provides a detailed history of the development of regional regression equations in Maryland from 1980 to 2022. Regional regression equations have been updated in Maryland by the MDOT SHA as the need and funding become available. Since 2006, the regression equations have been updated approximately every four years.

The current regression equations described in this report are based on the following reports:

- Regression Equations for Estimating Flood Discharges for the Piedmont, Blue Ridge and Appalachian Plateau Regions in Western Maryland (Thomas and Moglen, 2016) that used annual peak flow data through the 2012 water year,
- Regression Equations for Estimating Flood Discharges for the Eastern Coastal Plain Region of Maryland (Thomas and Sanchez-Claros, 2019a) that used annual peak data through the 2017 water year,
- Regression Equations for Estimating Flood Discharges for the Western Coastal Plain Region of Maryland (Thomas and Sanchez-Claros, 2019b) that used annual peak data through the 2017 water year, and
- Revisions to the equations in Thomas and Sanchez-Claros (2019a; 2019b) as described herein.

The regression equations in Thomas and Moglen (2016) for the Appalachian Plateau Region were based on land slope from the legacy DEM data in GISHydro2000 prior to the 2016 study. The current default DEM data in GISHydro are based on DEM data dated May 2018. Therefore, the Appalachian Plateau Region equations were updated in 2020 using the new land slopes based on the May 2018 DEM data to simplify the application of the equations and the revised equations are described herein. Flood frequency estimates based on annual peak data through the 2012 water year and the drainage areas documented in Thomas and Moglen (2016) were used for the revised regression analysis with the only change the updated land slope data.

The regression equations in Thomas and Moglen (2016) for the Piedmont-Blue Ridge Region were based on drainage area, percent impervious area, percent limestone and percent forest cover. Based on experience, these regression equations tended to give conservatively high flood discharges for small rural watersheds (less than 10 square miles and less than 10 percent impervious area). The reason was that most small rural gaging stations had flood data during the period 1965 to 1977, when several large floods occurred. To correct this bias, flood discharges for 13 small stream rural gaging stations were adjusted downward and the regression equations were recomputed in 2020. All other data given in Thomas and Moglen (2016) were used in the 2020 regression analysis. The revised regression equations for the Piedmont-Blue Ridge Region were based on drainage area, percent impervious area and percent limestone. The percent forest cover was omitted from the revised 2020 equations because the high correlation with impervious area resulted in irrational regression coefficients for impervious area and forest cover. Details on developing the revised regression equations are provided later.

As discussed earlier in this Appendix, the 2020 regression equations for the ECP and WCP Regions (Thomas and Sanchez-Claros, 2019a; 2019b) were revised in 2022 to use a new estimate of the percent A soils based on the Dominant Condition approach for aggregating the soils data. Details on developing the revised 2022 equations are provided later. The only changes in the regression equations from the July 2020 (Fifth Edition) Hydrology Panel report to the equations reported in this report are for the ECP and WCP Regions.

Background

A brief history of the development of regional regression equations in Maryland since 1980 is as follows:

- Statewide study described in USGS Open-File Report 80-1016 (Carpenter, 1980) based on annual peak flow data through the 1977 water year for rural watersheds.
- Statewide study described in USGS Water-Resources Investigations Report 95-4154 (Dillow, 1996) based on annual peak flow data through the 1990 water year for rural watersheds.
- Statewide study by Moglen and others (2006) that investigated and compared regional L-moments, Region of Influence and Fixed Region regression equations for regional analysis using annual peak flow data through the 1999 water year. The Fixed Region equations from this analysis were considered most reasonable and documented in the 2006 Hydrology Panel report. Rural and urban watersheds were combined into a single analysis for the Western Coastal Plain Region but separate equations were provided for rural and urban watersheds in the Piedmont Region.
- A 2010 update of regression equations included using SSURGO soils (in lieu of STATSGO) for the Eastern and Western Coastal Plain Regions with annual peak data through 2006 for the Eastern Coastal Plain and through 2008 for the Western Coastal Plain. The Piedmont Region **urban equations** were not updated (same as 2006 Hydrology Panel report) but the Piedmont and Blue Ridge Regions were combined in a single region for **rural watersheds** using annual peak data through the 1999 water year. The Hydrology Panel developed a new map that defined limestone areas in both the Piedmont and Blue Ridge Regions. These equations were documented in the 2010 Hydrology Panel report.
- The 2016 update of the regression equations included revised equations for the Piedmont-Blue Ridge and Appalachian Plateau Regions using annual peak data through the 2012 water year (Thomas and Moglen, 2016). These regression equations were documented in the 2016 Hydrology Panel report.

- The 2020 update of the regression equations included revised equations for the Eastern and Western Coastal Plain Regions using annual peak data through the 2017 water year (Thomas and Sanchez, 2019a; 2019b). The percent A soils used in these equations were based on the Dominant Component approach for aggregating the soils data.
- This current update in 2022 revised the regression equations for the Eastern and Western Coastal Plain Regions to use the percent A soils based on the Dominant Condition soils data. All of the other data remained the same as documented in Thomas and Sanchez-Claros (2019a; 2019b) including annual peak data through the 2017 water year.

As noted above, the Piedmont and Blue Ridge Regions were combined into a single region in the 2010 update of the regression equations for rural watersheds because the flood characteristics of the two regions were very similar. In addition, the Hydrology Panel determined that areas of limestone are also prevalent in the Piedmont Region so combining the two regions was reasonable. The limestone map developed by the Hydrology Panel in 2010 has been used in all subsequent regional regression analyses.

In the Thomas and Moglen (2016) analysis, the rural and urban watersheds were combined into a single analysis for the Piedmont-Blue Ridge Region so one set of regression equations is appropriate for rural and urban watersheds and this combined region includes all limestone areas in Maryland. The four current hydrologic regions in Maryland are shown in Figure A3-1.

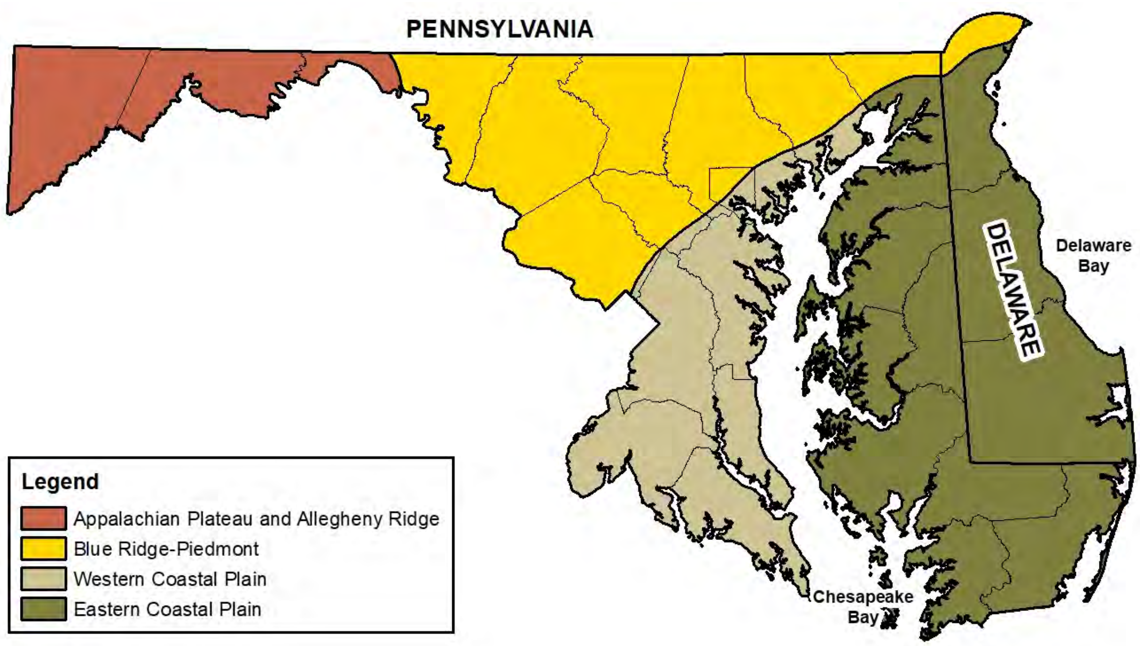


Figure A3-1: Hydrologic regions for Maryland as used by Thomas and Moglen (2016), Thomas and Sanchez-Claros (2019a) and Thomas and Sanchez-Claros (2019b)

Regional Skew Analyses

The recommended approach in Bulletin 17B (Interagency Advisory Committee on Water Data (IACWD), 1982) and Bulletin 17C (England and others, 2019) is to estimate flood discharges based on a weighted skew that is computed by weighting station and generalized (regional) skew inversely proportional to their respective mean square errors. For all regional studies since 2006, a regional skew has been determined by averaging skews at rural gaging stations with approximately 20 years or more of data and estimating the standard error as the standard deviation of those values. Additional details on the regional skew studies are given later in this appendix. The regional or generalized skew and standard error for rural watersheds in the four hydrologic regions are:

- Appalachian Plateau: regional or generalized skew is 0.43 with a standard error of 0.42,
- Piedmont-Blue Ridge: regional or generalized skew is 0.43 with a standard error of 0.42,
- Western Coastal Plain: regional or generalized skew is 0.0.38 with a standard error of 0.0.38, and
- Eastern Coastal Plain: regional or generalized skew is 0.0.38 with a standard error of 0.0.38.

These regional skew values were used in estimating a weighted skew for the final frequency analyses for all rural gaging stations. Station skew was used for the final frequency curves for all urban gaging stations since the skew changes with urbanization.

Measures of Accuracy of the Regional Regression Equations

The accuracy of regression equations can be described by several measures. For this report, two measures of accuracy are provided: the standard error of estimate in percent and the equivalent years of record that are used to weight gaging station and regression estimates.

The standard error of estimate is a measure of how well the gaging station estimates of flood discharges agree with the computed regression equation. This value is estimated as the standard deviation of the residuals about the computed equation where the residuals are the difference between gaging station and regression estimates.

The equivalent years of record is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of estimate of the regional regression equation. The equivalent years of record (Nr) is computed as follows (Hardison, 1971):

$$Nr = (S/SE)^2 \, R^2$$

where S is an estimate of the standard deviation of the logarithms of the annual peak discharges at the ungaged site, SE is the standard error of estimate of the Fixed Region regression estimates in logarithmic units, and R^2 is a function of recurrence interval and skewness and is computed as (Stedinger and others, 1993):

$$R^2 = 1 + G*Kx + 0.5 *(1 + 0.75 * G^2) * Kx^2$$

where G is an estimate of the average skewness for a given hydrologic region, and Kx is the Pearson Type III frequency factor for recurrence interval x and skewness G. In addition to estimate the equivalent years of record at an ungaged site, the standard deviation of the logarithms of the annual peak discharges (S in the Nr equation above) must be estimated. These values for each region are as follows:

- Appalachian Plateau: S = 0.2353 log units, G = 0.39,
- Piedmont-Blue Ridge: S = 0.3070 log units, G = 0.48,
- Western Coastal Plain: S = 0.3196 log units, G = 0.541, and
- Eastern Coastal Plain: S = 0.3104 log units, G = 0.330.

Values of S and G are used to estimate the equivalent years of record for the regression equations in order to weight the gaging station and regression estimates.

Regression Equations for Rural Watersheds in the Eastern Coastal Plain Region

Introduction

Fixed region regression equations are used to estimate flood discharges for bridge and culvert design and floodplain mapping in Maryland by several state and local agencies. These empirical equations are developed based on relations between flood discharges at gaging stations and watershed characteristics that can be estimated from available digital data layers. For ungaged locations, the watershed characteristics are used in the regression equations to predict the flood discharges. The MDOT SHA uses the regression equations to primarily evaluate the reasonableness of flood discharges estimated using the TR-20 watershed model (Maryland Hydrology Panel, 2020). The objective of the current analysis is to update the Fixed Region regression equations for the Eastern Coastal Plain Region for estimating the 1.25-, 1.5-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year flood discharges using the following data:

- Annual peak flow data through the 2017 water year, if available,
- Flood frequency analyses using Bulletin 17C (England and others, 2019),
- Watershed characteristics computed using GISHydro, land use data for various time periods, 30-meter Digital Elevation Model (DEM) data dated May 2018, 1:100,000 National Hydrography Data (NHD) dated May 2018, and legacy SSURGO data in GISHydro, and
- SSURGO data downloaded from the Natural Resources Conservation Service (NRCS) web site in October 2021.

Documentation for GISHydro is located at (http://www.gishydro.eng.umd.edu/document.htm). Both sets of SSURGO data were evaluated as explanatory variables to see which data set was most appropriate for estimating flood discharges using regression equations.

Previous Studies in the Eastern Coastal Plain Region

Several studies have been completed since 1980 that developed regional regression equations for Maryland. Following is a brief description of the data used in previous regression equations for the Eastern Coastal Plain Region:

- U.S. Geological Survey (USGS) Open-File Report 80-1016 (Carpenter, 1980) – used drainage area, channel slope, percent storage, percent forest cover and percent A and D soils based on **STATSGO** soils and annual peak flow data through the 1977 water year,
- USGS Water-Resources Investigations Report 95-4154 (Dillow, 1996) – used drainage area, runoff curve number (**STATSGO** data), basin relief, percent forest, percent storage and annual peak flow data through the 1990 water year,

- Maryland Hydrology Panel report (2006) and Moglen and others (2006) – used drainage area, basin relief and percent A soils based on **STATSGO** soils data and annual peak flow data through the 1999 water year,
- USGS Scientific Investigations Report 2006-5146 (Ries and Dillow, 2006) (report for Delaware streams) – used drainage area, land slope and percent A soils based on **STATSGO** soils data and annual peak flow data through the 2004 water year,
- Maryland Hydrology Panel report (2010) – used drainage area, land slope, percent A soils based on **SSURGO** soils data and annual peak flow data through the 2006 water year, and
- Maryland Hydrology Panel report (2020) - used drainage area, land slope, percent A soils based on **SSURGO** soils data and annual peak flow data through the 2017 water year.

A water year is from October 1 to September 30 with the ending month determining the water year. For example, the 2017 water year is from October 1, 2016 to September 30, 2017. The 2022 update of the Eastern Coastal Plain (ECP) regression equations just involved the update of the SSURGO soils data. The flood frequency data and other explanatory variables remained the same as used in the 2020 analysis.

Flood Frequency Analyses at Gaging Stations

Flood frequency estimates were updated through 2017 if data were available using the USGS PeakFQ program (https://water.usgs.gov/software/PeakFQ/) that implements Bulletin 17C (England and others, 2019). Flood data were compiled and analyzed for 41 gaging stations in the ECP: 22 stations in Maryland and 19 stations in Delaware. The location of the gaging stations is shown in Figure A3-2 that defines the four major hydrologic regions in Maryland: Appalachian Plateau and Allegheny Ridge, Blue Ridge-Piedmont, Western and Eastern Coastal Plains. The gaging stations are numbered in terms of their USGS downstream station number with stream names and numbers listed in Attachment ECP-1.

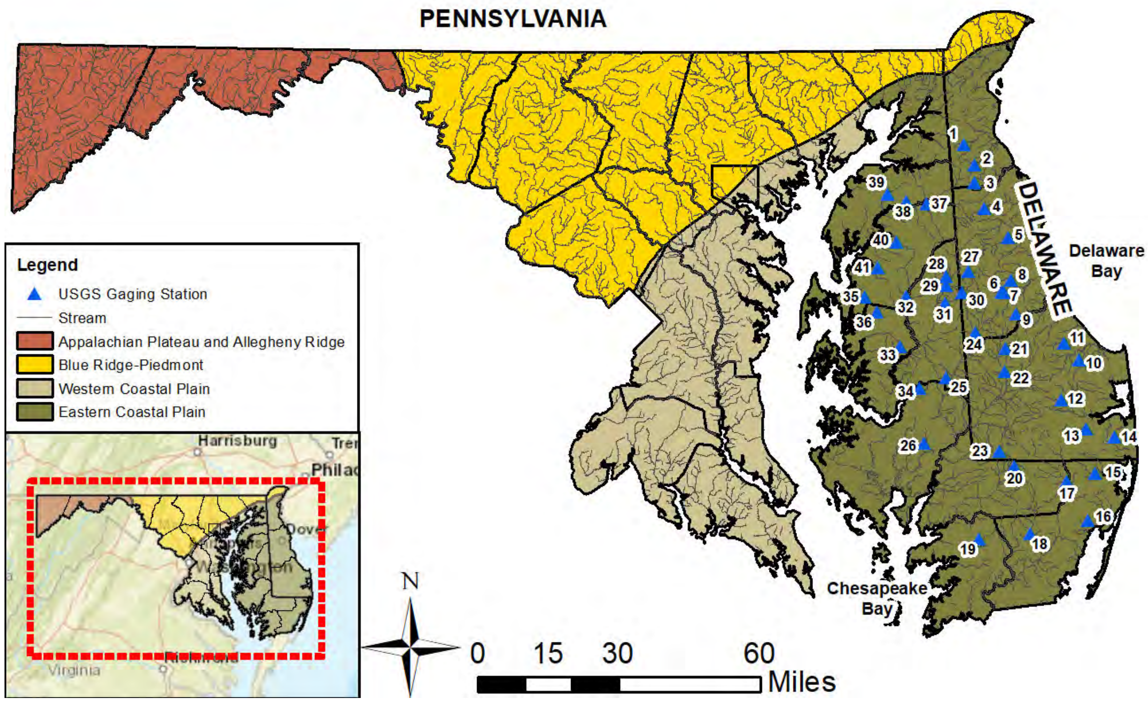


Figure A3-2: Location of gaging stations in the Eastern Coastal Plain Region of Maryland

For the 41 stations shown in Figure A3-2, only 16 stations were still active in 2017. Record lengths ranged from 9 to 75 years with 19 stations having 20 or more years of record. As noted earlier, there are 22 stations in Maryland and 19 stations in Delaware shown in Figure A3-2. The 2010 analysis (latest since the 2020-2022 updates) for the ECP Region evaluated 16 stations in Maryland and 15 stations in Delaware. Therefore, six new stations in Maryland and four new stations in Delaware were added to the current analysis.

Rural stations with 19 or more years of record were analyzed to obtain station skew to estimate a new regional skew. Using 15 stations from the ECP Region and eight stations from the Western Coastal Plain Region (WCP), a regional skew of 0.38 with a standard error of 0.38 was determined. The 2010 regional skew analysis for the ECP resulted in a regional skew of 0.45 with a standard error of 0.41 so the change in regional skew is minimal.

For three stations, the frequency curves were S-shaped (likely due to floodplain storage) and the plotting positions for the logarithms of the data did not fit a Pearson Type III distribution very well. A graphical frequency analysis was performed for: Blackbird Creek at Blackbird, DE (01483200), Manokin Branch near Princess Anne, MD (19486000) and Marshyhope Creek near Adamsville, DE (10488500). Records were extended for two short record stations using a graphical analysis with a nearby long-record station: Southeast Creek at Church Hill, MD (01494000) extended with records from Beaverdam Branch at Matthews, MD (01492000) and Three Bridges Branch at Centerville, MD (01494150) extended with records from Sallie Harris Creek near Carmichael, MD (01492500).

Thirteen of the 41 stations had flood data for the period 1965 to 1976 when the USGS small streams program was active (gaging stations less than 10 square miles). This was an active flood period with major floods in 1967 and 1975. Most of the small stream sites experienced their maximum flood in August 1967 that was generally known to be the highest flood since 1935. A historical period of 40 years (highest flood in the period 1936 to 1976) was used in the frequency analysis for those stations experiencing a major flood in 1967 to obtain more reasonable estimates of the flood discharges. Historical information was also available for some stations for the 1975 flood. There were 10 small stream stations where Carpenter (1980) extended the records using a rainfall-runoff model and nearby long-term climatic data. The frequency estimates from Carpenter (1980) were evaluated and it was determined that the frequency estimates based on observed and historical data as described above were more reasonable.

There were nine stations that had 62 to 74 years of record and seven of these stations had statistically significant upward trends due to large floods near the end of the record in 1989, 1999, 2011 and 2016. The impervious area of the watersheds was less than 10 percent of the drainage area so the upward trends were not related to urbanization. The upward trends were assumed to be climatic persistence or variability (not a permanent change in climate) and the entire period of record was used in the frequency analysis. The only exception was Marshyhope Creek near Adamsville, DE (01488500) where extensive channelization occurred during 1969-71. The period 1972 to 2017 was considered a homogeneous period and used for the frequency analysis that was based on a graphical analysis.

The final flood frequency estimates were based on a weighted skew (combining station and regional skew) for all stations because there are no urban stations in the ECP Region. The flood discharges are provided in Attachment ECP-1. The period of record and years of record at the gaging stations are given in Attachment ECP-2.

Watershed Characteristics Evaluated for the Regression Analysis

The watershed characteristics evaluated for the regression analysis included those that were statistically significant in previous regression analyses and were estimated using the digital data in GISHydroNXT (http://www.gishydro.eng.umd.edu/document.htm) as of May 2018 with the addition of SSURGO soils data added in October 2021. The watershed characteristics included:

- Drainage area (DA), in square miles, computed as the number of pixels covering the watershed area times the pixel's area or cell size,

- Channel slope (CSL), in feet per mile, computed as the difference in elevation between two points located 10 and 85 percent of the distance along the main channel from the outlet divided by the distance between the two points,
- Land slope (LSLOPE), in feet per feet, sometimes referred to as watershed slope, computed as the average of all neighborhood slopes determined along the steepest direction of flow for all the pixels in the watershed (used in the regression analysis as a percent),
- Basin Relief (BR), in feet, computed as the average elevation of all pixels within the watershed minus the elevation at the outlet of the watershed,
- Forest cover (FOR), in percent of the drainage area, for 2002 and 2010 land use conditions,
- Percent A, B, C and D SSURGO soils based on the legacy data in GISHydro prior to 2018, and
- Percent A, B, C and D SSURGO soils based on soils data downloaded from the NRCS Soil Survey web site in October 2021.

The legacy SSURGO soils data in GISHydro prior to 2018 are shown in Figure A3-3 for the four Hydrologic Soil Groups A, B, C and D where A has the highest infiltration and D the lowest infiltration. These data were added to GISHydro over time and were representative of different dates for each county in Maryland and Delaware.

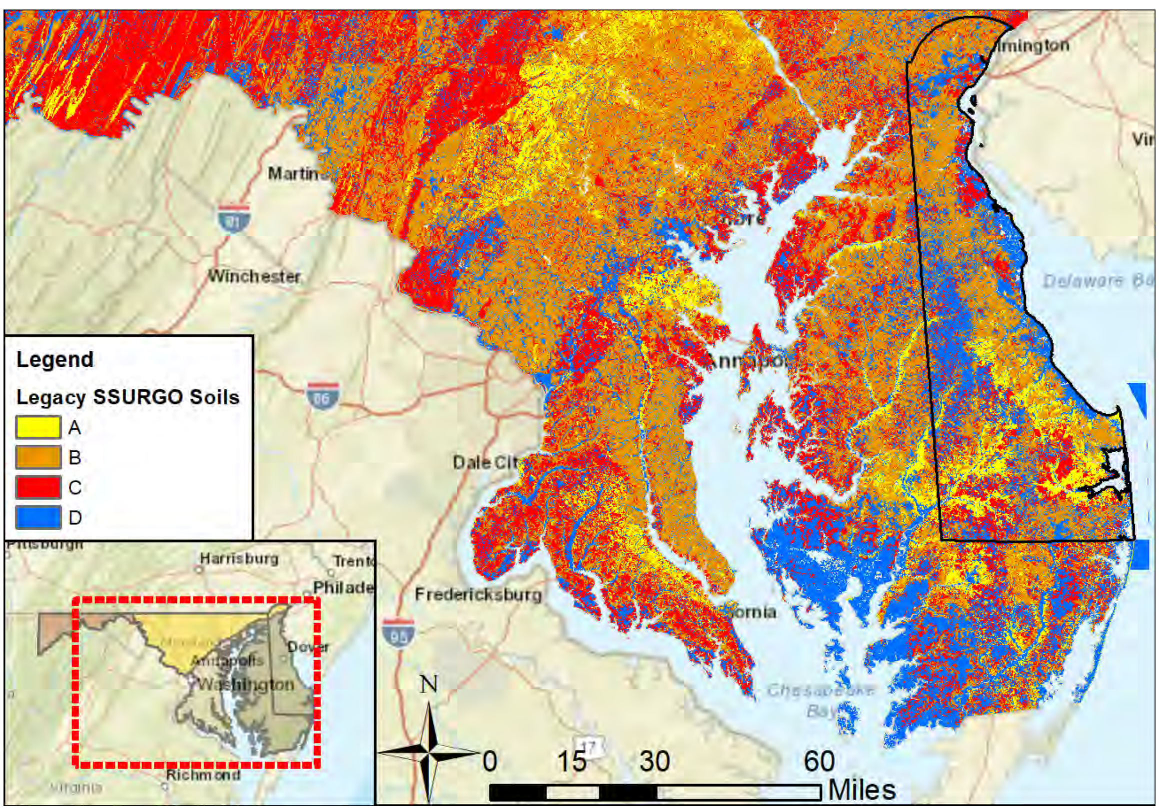


Figure A3-3: Legacy SSURGO soils data in GISHydroNXT

The latest SSURGO soils data downloaded from the NRCS Soil Survey web site in October 2021 and are shown in Figure A3-4. The NRCS procedures for estimating the Hydrologic Soil Groups (HSGs) were updated prior to 2009 and documented in the NRCS Part 630 Hydrology, National Engineering Handbook, Chapter 7, Hydrologic Soils Group (HSG) dated January 2009. The calculations for the new HSGs were completed for Maryland in 2014 and the updated HSGs were posted to the NRCS Soil Survey database in 2016. The SSURGO soils on the NRCS Web Soil Survey site are updated annually and data available in October 2021 were used in this current analysis. The new criteria for assigning HSGs use soil properties that influence runoff potential such as:

- Depth to a seasonal high-water table,
- Saturated hydraulic conductivity (Ksat) after prolonged wetting, and
- Depth to a layer with a very slow water transmission rate.

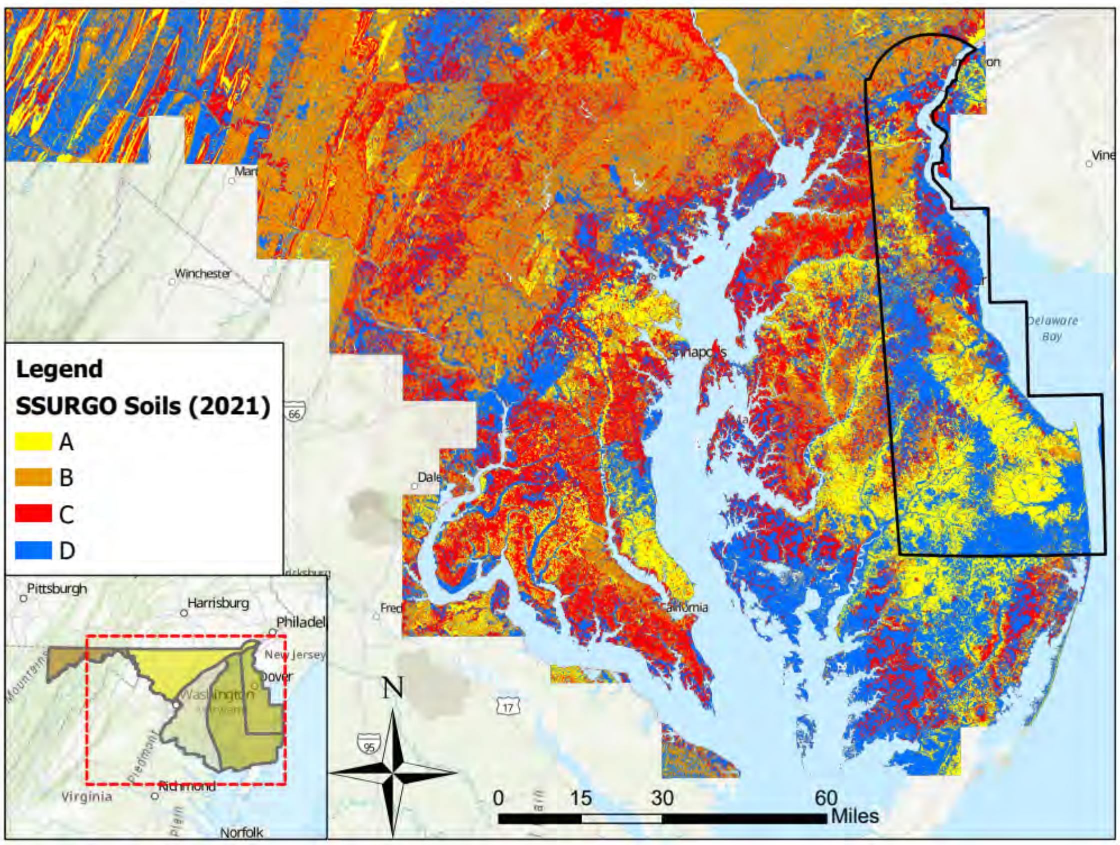


Figure A3-4: The October 2021 SSURGO soils data in GISHydro

Impervious area in percent of the watershed area was also evaluated as a potential explanatory variable even though for most watersheds in the ECP Region the impervious area is less than 5 percent. For this analysis, impervious area for Maryland streams was only estimated for 2010 land use conditions. Impervious area for 1985, 1990 and 1997 land use conditions for Maryland streams were available from the 2016 and 2020 Hydrology Panel reports. For Delaware streams, impervious area was only available for 2002 land use conditions based on land use data for the Delaware version of GISHydro. Impervious area indicative of the gaging station record was estimated for all gaging stations using the data described above.

For Maryland streams, the highest impervious area for any gaging station was 5.1 percent for Three Bridges Branch at Centerville, MD (01494150) with record from 2007 to 2017. The highest impervious area for any Delaware gaging station was 5.0 percent for Stockley Branch at Stockley, DE (01484500) with record from 1943 to 2004. There are three gaging stations in Delaware where the impervious area for 2002 land use conditions exceeds 10 percent. However, these gaging stations were discontinued in the 1970s and were rural watersheds during the time of data collection.

A regression analysis including impervious area revealed that impervious area is not statistically significant. The exponent on impervious area was negative which is not hydrologically rational. There is not enough variation in impervious area for this variable to be a significant explanatory variable in the regression analysis for the ECP Region.

A correlation analysis was performed to determine which explanatory variables were highly correlated. The objective of the regression analysis is to have the explanatory variables as independent as possible. For the soils data, only the A and D soils (extreme values) were evaluated because these hydrologic soil groups were statistically significant in previous analyses.

The correlation analysis was done for the logarithms of the topographic characteristics and flood discharges and the untransformed and the logarithmic transformation for the soils data. Only the A soil variable is shown in the correlation matrix in Figure 4 since it was the most statistically significant. The variable Aold is the A soil value from the legacy SSURGO data in GISHydro (Figure A3-3) and Acond is the value based on the October 2021 SSURGO data (Figure A3-4). A “l” before the variable name implies it is a logarithmic value (only A soil is evaluated for untransformed data).

Some pertinent correlations are highlighted in yellow in Figure A3-5 and include:

- The 100-year discharge (lq100) is more correlated with the untransformed A soil than the logarithmic transformed values,
- Drainage area (lda) and channel slope (lcsl) are negatively correlated (-0.533),
- Land slope (lslope) and basin relief (lbr) are highly correlated (0.750),
- Land slope (lslope) and channel slope (lcsl) are highly correlated (0.788),
- The 100-year discharge (lq100) is more highly correlated with basin relief (lbr) (0.616) than land slope (lslope) (0.334), and
- Acond and Aold soils are more correlated for the untransformed data (0.830) than the log transformed data (0.515).

The correlation matrix in Figure A3-5 is for the 35 gaging stations that were used in the final regression analysis but is indicative of all 41 stations originally considered for analysis.

Pearson Correlation Coefficients, N = 35
Prob > |r| under H0: Rho=0

	lq100	lda	lcsl	lslope	lbr	acond	lacond	aold	laold	lfor
lq100	1.00000	0.65594	-0.03598	0.33449	0.61572	-0.45464	-0.36594	-0.45914	-0.28196	0.04378
		<.0001	0.8374	0.0495	<.0001	0.0061	0.0306	0.0055	0.1008	0.8028
lda	0.65594	1.00000	-0.53526	-0.06146	0.41816	0.10587	0.20497	0.09355	0.28012	0.28583
	<.0001		0.0009	0.7258	0.0124	0.5450	0.2375	0.5930	0.1031	0.0960
lcsl	-0.03598	-0.53526	1.00000	0.78758	0.46237	0.02301	-0.17140	-0.05968	-0.26830	-0.03188
	0.8374	0.0009		<.0001	0.0052	0.8956	0.3249	0.7334	0.1191	0.8558
lslope	0.33449	-0.06146	0.78758	1.00000	0.75045	0.04779	-0.09010	-0.07424	-0.21058	0.05357
	0.0495	0.7258	<.0001		<.0001	0.7851	0.6067	0.6717	0.2247	0.7599
lbr	0.61572	0.41816	0.46237	0.75045	1.00000	0.10638	-0.00070	0.01184	-0.01399	0.21882
	<.0001	0.0124	0.0052	<.0001		0.5430	0.9968	0.9462	0.9364	0.2066
acond	-0.45464	0.10587	0.02301	0.04779	0.10638	1.00000	0.83007	0.83039	0.68708	0.46955
	0.0061	0.5450	0.8956	0.7851	0.5430		<.0001	<.0001	<.0001	0.0044
lacond	-0.36594	0.20497	-0.17140	-0.09010	-0.00070	0.83007	1.00000	0.55701	0.51545	0.55094
	0.0306	0.2375	0.3249	0.6067	0.9968	<.0001		0.0005	0.0015	0.0006
aold	-0.45914	0.09355	-0.05968	-0.07424	0.01184	0.83039	0.55701	1.00000	0.89294	0.28105
	0.0055	0.5930	0.7334	0.6717	0.9462	<.0001	0.0005		<.0001	0.1019
laold	-0.28196	0.28012	-0.26830	-0.21058	-0.01399	0.68708	0.51545	0.89294	1.00000	0.26904
	0.1008	0.1031	0.1191	0.2247	0.9364	<.0001	0.0015	<.0001		0.1181
lfor	0.04378	0.28583	-0.03188	0.05357	0.21882	0.46955	0.55094	0.28105	0.26904	1.00000
	0.8028	0.0960	0.8558	0.7599	0.2066	0.0044	0.0006	0.1019	0.1181	

Figure A3-5: Correlation matrix for the 100-year discharge (lq100) and selected watershed characteristics for 35 stations in the Eastern Coastal Plain Region of Maryland

The correlation matrix is helpful in explaining why variables are statistically significant in the regression analysis. Of two highly correlated explanatory variables, only one will likely be statistically significant in the regression equation since the two variables are explaining the same variability in the discharge variable. For example, one would not expect channel slope (lcsl) and land slope (lslope) to be statistically significant in the same equation since these variables are highly correlated (0.788). As shown in Figure A3-5, land slope (lslope) is more highly correlated with the 100-year discharge (lq100) than channel slope (lcsl) and land slope was more statistically significant in the regression analysis.

Development of Regression Equations

Multiple regression analyses were run using the watershed characteristics described earlier and the Statistical Analysis System (SAS) computer software developed by the SAS Institute, Inc., Cary, NC (https://www.sas.com/en_us/company-information.html). A and D soils based on the legacy and October 2021 SSURGO data were used in the regression analysis. For the soil parameters, the percent A soil was more statistically significant than the D soil similar to previous analyses. The Acond soil data (October 2021) provided a lower standard error than Aold soil (legacy) data and was used in the regression equations. The most statistically significant variables used in the final regression equations include drainage area, in square miles, ranging from 0.91 to 113.8 square miles; A soil, in percent, ranging from 0.5 to 82.7 percent; and land slope, in percent, ranging from 0.463 to 2.20 percent.

Land slope was estimated in feet per feet and then converted to percent to reduce the regression constant to a more reasonable value. **Therefore, the user must input land slope in percent in the regression equations.** The standard error is the same whether ft/ft or percent is used in the regression analysis. The watershed characteristics used in the regression equations are given in Attachment ECP- 2 for all 41 stations. The watershed characteristics evaluated but not used in the final regression equations are given in Attachment ECP-3. The forest cover and impervious area in Attachment ECP-3 are indicative of land use conditions at the midpoint of the gaging station record, not the current (2010) land use conditions. A comparison of the October 2021 and legacy SSURGO soils data is given in Attachment ECP-4.

The explanatory variables used in this analysis for the ECP Region are the same as historical analyses performed by Ries and Dillow (2006) and the 2010 Maryland Hydrology Panel report. Basin relief provides equations with about the same standard error as land slope but land slope was chosen for the equations since it is more independent of drainage area than basin relief (see Figure A3-5) and has a more uniform range of values.

The following regression equations were based on 35 stations minus six outlier stations: Silver Lake Tributary at Middletown, DE (01483155), Puncheon Branch at Dover, DE (01483700), Murderkill River Tributary near Felton, DE (01484002), Birch Branch at Sowell, MD (0148471320), Andrews Branch near Delmar, MD (01486100) and Toms Dam Branch near Greensboro, MD (01486980). Murderkill River Tributary had a large flood in a short record and the gaging station estimates were conservatively high. Birch Branch had an indeterminate drainage area and the other three stations had very low annual peaks for the size of the watershed.

The regression equations, standard errors of estimate (SE) and equivalent years of record are given below. As discussed earlier, the Acond variable was not transformed to logarithms so this variable is represented in the equations as an exponent to the base 10. The equivalent years of record is defined as the number of years of actual streamflow record required to achieve an accuracy equivalent to the standard error of the regression equation. Equivalent years of record are used to weight the gaging station estimates with the regression estimates following the approach described by Dillow (1996) and described in Chapter 2 of this report. The computation of equivalent years of record is described in Attachment ECP-5.

Equation	**Standard Error (%)**	**Eq. years**	
$Q_{1.25} = 35.3\ DA^{0.763}\ LSLOPE^{0.120}\ 10^{-0.00815*Acond}$	46.0	2.8	(A3-1)
$Q_{1.5} = 47.7\ DA^{0.762}\ LSLOPE^{0.195}\ 10^{-0.00900*Acond}$	44.1	3.0	(A3-2)
$Q_{2} = 67.1\ DA^{0.754}\ LSLOPE^{0.276}\ 10^{-0.00921*Acond}$	42.3	3.2	(A3-3)
$Q_{5} = 135.6\ DA^{0.738}\ LSLOPE^{0.470}\ 10^{-0.01032*Anew}$	40.0	6.8	(A3-4)
$Q_{10} = 202.4\ DA^{0.726}\ LSLOPE^{0.603}\ 10^{-0.01098*Acond}$	39.2	10	(A3-5)
$Q_{25} = 321.4\ DA^{0.705}\ LSLOPE^{0.795}\ 10^{-0.01161*Acond}$	39.6	19	(A3-6)
$Q_{50} = 432.1\ DA^{0.696}\ LSLOPE^{0.898}\ 10^{-0.01214*Acond}$	40.4	19	(A3-7)
$Q_{100} = 569.0\ DA^{0.687}\ LSLOPE^{0.996}\ 10^{-0.01263*Acond}$	42.1	21	(A3-8)
$Q_{200} = 736.0\ DA^{0.679}\ LSLOPE^{1.082}\ 10^{-0.01312*Acond}$	44.4	23	(A3-9)
$Q_{500} = 1033.8\ DA^{0.663}\ LSLOPE^{1.185}\ 10^{-0.01365*Acond}$	47.9	24	(A3-10)

For Equations A3-1 to A3-10, the drainage area exponent decreases with an increasing recurrence interval, consistent with earlier results. A possible explanation is that the storm rainfall for the larger storms varies considerably across a watershed and does not have a uniform impact across the entire watershed (that is, the effective drainage area is less). The exponent on land slope (LSLOPE) increases as the recurrence interval increases implying this variable becomes more significant as rainfall depth and runoff increases over the watershed. The exponent on Acond increases from the 1.25-year flood up to the 500-year flood implying the soils become more significant as rainfall depth and runoff increases over the watershed.

The explanatory variables drainage area and Acond are statistically significant for all recurrence intervals at the 5-percent level of significance, but land slope is only statistically significant at this level for the 5-year flood and larger. Land slope was maintained in the equations for the 1.25-, 1.5- and 2-year discharges to achieve consistency across recurrence intervals. The 5-percent level of significance is traditionally used in regression analysis for determining statistical significance because this implies there is less than a 5-percent chance of erroneously including a variable in the equation when it is not really significant.

The higher standard errors for the shorter recurrence interval (1.25- to 5-year) floods imply that some unknown explanatory variables other than drainage area, land slope, and percentage of A soils influence these floods. The time-sampling error (error in T-year flood discharge) is less for these smaller floods, so one would expect a lower standard error in the regression analysis. Instead, the standard errors of the regression equations for the smaller events are influenced by the model error, indicating that other important explanatory variables may be missing from the equations.

The regression equations are unbiased and provide estimates consistent with the gaging station estimates. The 100-year regression estimates from Equation A3-8 are plotted versus the 100-year gaging station estimates in Figure A3-6 for the 35 gaging stations used to develop the regression equation. The line shown in Figure A3-6 is the equal discharge line and the data points scatter uniformly about this line.

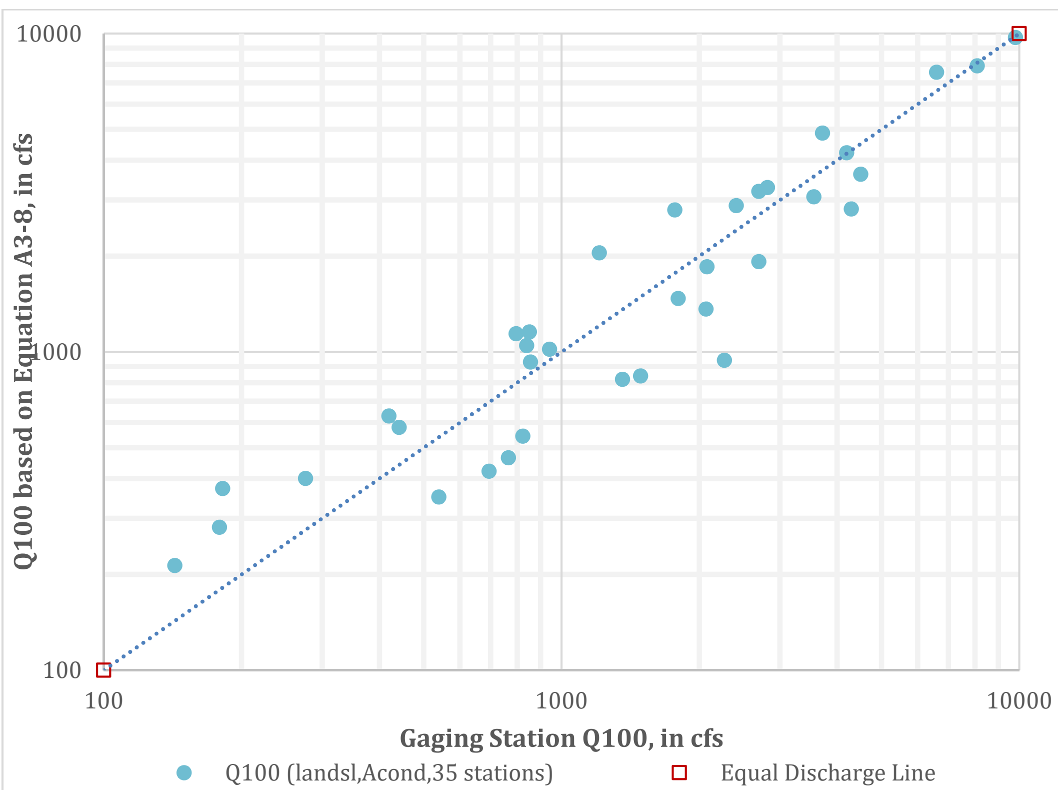


Figure A3-6: The 100-year regression estimates from Equation A3-8 plotted versus the 100-year estimates based on the gaging station data for 35 stations in the Eastern Coastal Plain Region

In Figure A3-6, the data points to the right of the equal discharge line are stations where the regression equation is underestimating the 100-year discharge and points to the left are indicative of the regression equation overestimating the 100-year discharge based on gaging station data. Although there is considerable scatter in Figure A3-6, there is no indication of bias in the regression equation except for the four smallest 100-year discharges where all four stations plot to the left of the equal discharge line. Three of these stations are located in the same area near the coast of Delaware in a high A soil area and are:

- Beaverdam Branch at Houston, DE (01484100), site 9 in Figure A3-2, Acond = 29.6 percent,
- Beaverdam Creek near Milton, DE (01484270), site 10 in Figure A3-2, Acond = 73.5 percent, and
- Sowbridge Branch near Milton, DE (01484300), site 11 in Figure A3-2, Acond = 82.7 percent.

The magnitude of the deviation from the equal discharge line is consistent with other stations so these three stations are not extreme outliers. Even though Acond is large for these stations, the regression equation still overestimates the 100-year discharge based on gaging station data. As shown in Figure A3-2, the three stations are in close proximity so there must be some other explanatory variable not in the regression equation that is impacting these watersheds. The variable Acond soil is still statistically significant in the 100-year equation even if the three stations above are omitted from the analysis. The decision was to keep these stations in the regression analysis since they reflect the impact of A soil.

The same analysis was performed for the 10-year flood with the 10-year regression estimates from Equation A3-5 plotted versus the 10-year gaging station estimates as shown in Figure A3-7. The three largest 10-year discharges plot to the right of the equal discharge line indicating the regression equation is underestimating the 10-year flood in comparison to the gaging station data. However, the departures from equal discharge line are small and Equation A3-5 is considered unbiased.

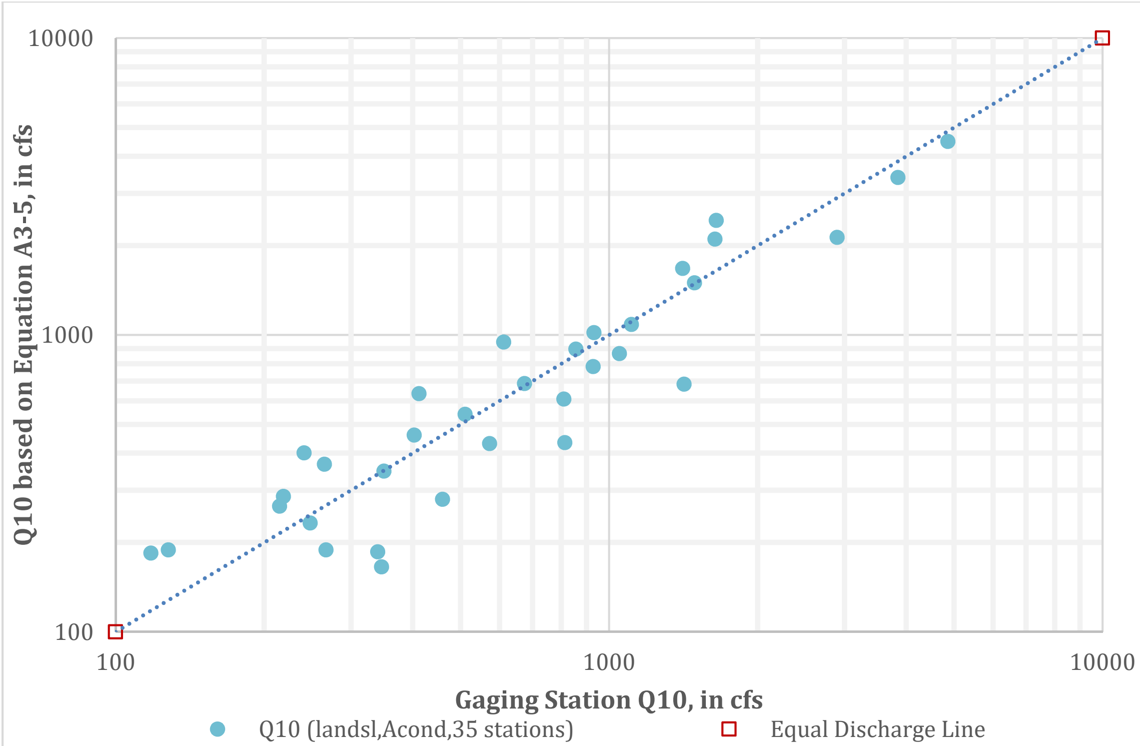


Figure A3-7: The 10-year regression estimates from Equation A3-5 plotted versus the 10-year estimates based on the gaging station data for 35 stations in the Eastern Coastal Plain Region

The 100-year regression estimates for the 2022 analysis (Equation A3-8) were also compared to the 100-year regression estimates for the equations developed in 2010 and documented in the July 2016 version of the Maryland Hydrology Panel report. The data are compared in Figure A3-8 for all 41 stations where the trend line is the equal discharge line.

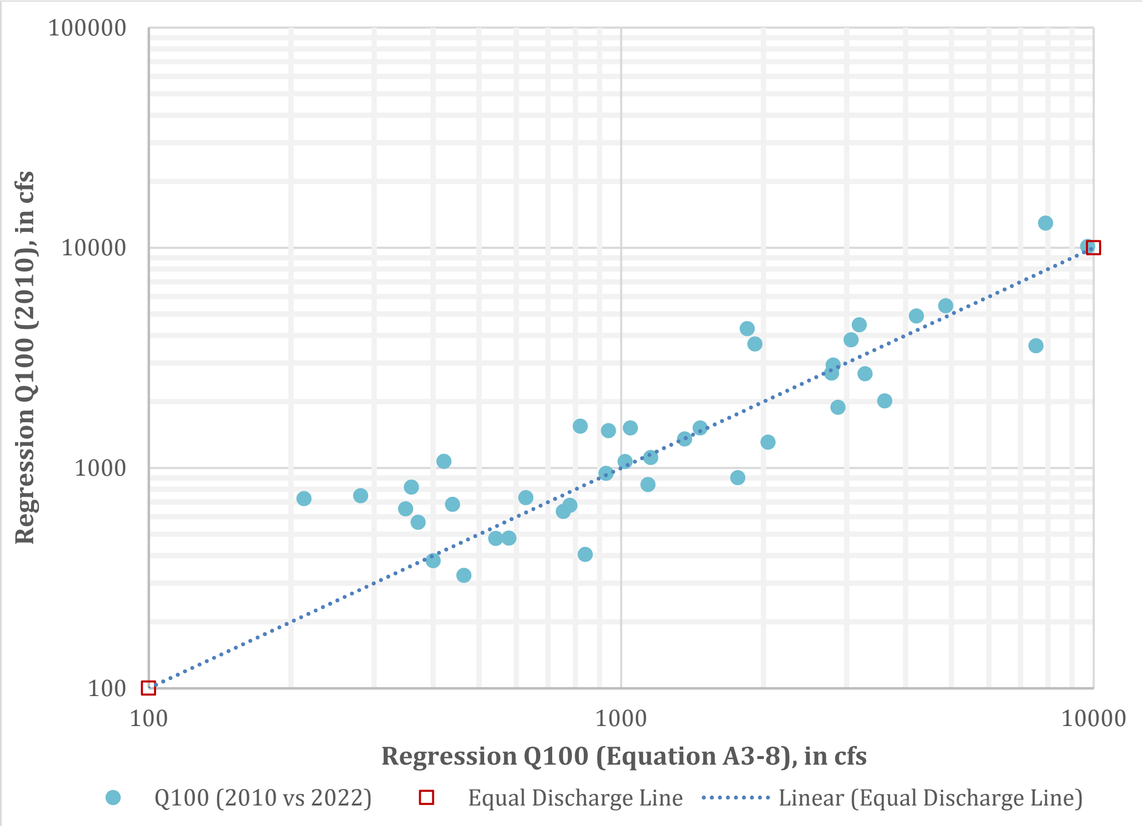


Figure A3-8: The 100-year regression estimates from the 2010 analysis versus the 100-year regression estimates based on the 2022 analysis (Equation A3-8) for 41 stations in the Eastern Coastal Plain Region

As shown in Figure A3-8, the 2010 equations for the 100-year discharge tend to predict slightly higher than Equation A3-8. On average, the 2010 equation is predicting a 100-year discharge about 10 percent higher than Equation A3-8. The largest differences between the 2010 and updated equations were evaluated and most of the time the updated equation gave estimates closest to the updated gaging station estimate. The 2010 and updated equations are based on the same variables (drainage area, land slope and A soil) but the updated estimates of drainage area and land slope are based on an updated DEM and the A soil data are from the October 2021 SSURGO data. Based on a comparison to updated gaging station data, Equation A3-8 is considered more accurate than the 2010 equation for estimating the 100-year discharge.

The 10-year regression estimates for Equation A3-5 were also compared to the 10-year regression estimates for the equations developed in 2010 and currently in use and documented in the July 2016 version of the Maryland Hydrology Panel report. The data are compared in Figure A3-9 for all 41 stations where the trend line is the equal discharge line.

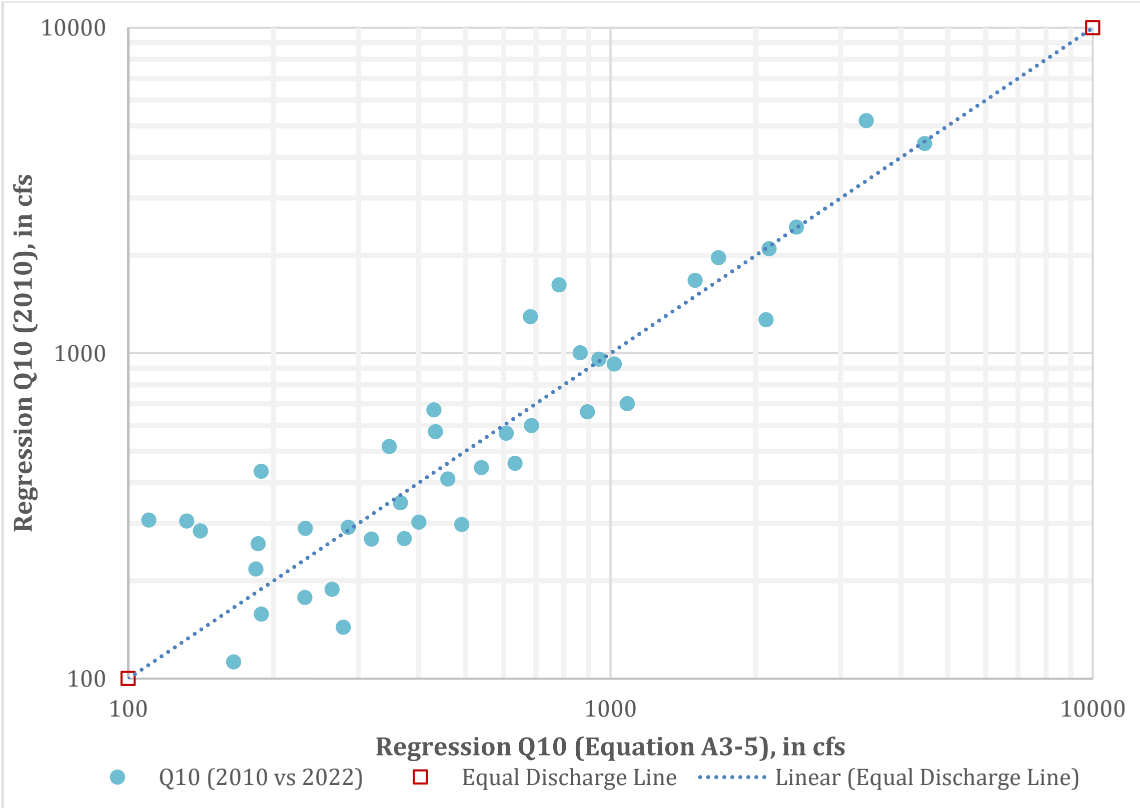


Figure A3-9: The 10-year regression estimates from the 2010 analysis versus the 10-year regression estimates based on Equation A3-5 for 41 stations in the Eastern Coastal Plain Region

As with the 100-year comparison, the 2010 equation for the 10-year discharge is providing slightly higher estimates of the 10-year discharge than Equation A3-5. On average, the 2010 estimates of the 10-year discharge are 8 percent higher than Equation A3-5. The largest differences between the 2010 and updated equations were evaluated and most of the time the updated equation gave estimates closest to the gaging station estimate. The 2010 and updated equations are based on the same variables (drainage area, land slope and A soil) but the updated estimates of drainage area and land slope are based on updated DEM data and the A soil data are from the October 2021 SSURGO data. Based on a comparison to gaging station data, Equation A3-5 is considered more accurate than the 2010 equation for estimating the 10-year discharge.

Summary and Conclusions

The Fixed Region regression equations for the Eastern Coastal Plain Region were updated using annual peak flow data through the 2017 water year. The updated flood discharges were based on Bulletin 17C (England and others, 2019). The regression equations were based on 35 gaging stations in Maryland and Delaware: 19 gaging stations in Maryland and 16 gaging stations in Delaware; 16 active stations and 19 discontinued stations as of 2017. Six gaging stations were considered outliers primarily because the annual peak flows were very low for the drainage area size and were not included in the regression analysis. The most statistically significant explanatory variables were drainage area, in square miles; land slope, in percent; and A soil data, in percent, based on the SSURGO data dated October 2021 from the NRCS Soil Survey web site. The legacy SSURGO data in GISHydro and the October 2021 SSURGO data were both evaluated in the regression analysis to determine which set of soils data provided the most accurate regression equations. The October 2021 SSURGO data provided the most accurate regression equations and is now the default soils data in GISHydro.

Nine stations had 62 to 74 years of record and seven of these stations had statistically significant upward trends due to large floods near the end of the record in 1989, 1999, 2011 and 2016. The impervious area of the watersheds was less than 10 percent of the drainage area so the upward trends were not related to urbanization. The upward trends were assumed to be climatic persistence or variability (not a permanent change in climate) and the entire period of record was used in the frequency analysis.

Equations A3-8 and A3-5 for the 100- and 10-year flood discharges, respectively, were compared to the respective gaging station estimates and shown to be reasonably unbiased. The updated regression estimates were also compared to the respective regression estimates from the 2010 analysis, regression equations that are documented in the July 2016 version of the Maryland Hydrology Panel report. The 2010 equations provided 100-year discharges that are about 10 percent higher, on average, than Equation A3-8. For the 10-year discharges, the 2010 equations provided estimates about 8 percent higher, on average, than Equation A3-5.

Attachment ECP-1. Flood discharges for the 1.25-, 1.5-, 2-, 5-, 10-, 25- 50-, 100-, 200- and 500-year events (in cubic feet per second) for 41 gaging stations in the Eastern Coastal Plain of Maryland. Map No. refers to Figure A3-2

Map No.	Station No.	Stream name	DA (mi^2)	Q1.25	Q1.5	Q2	Q5	Q10	Q25	Q50	Q100	Q200	Q500
1	01483155	Silver Lake Tributary at Middletown, DE	2.03	58	74	96	169	232	333	424	532	658	857
2	01483200	Blackbird Creek at Blackbird, DE	4.06	90	120	160	240	350	680	760	840	900	980
3	01483290	Paw Paw Branch Tributary near Clayton, DE	0.91	93	116	149	255	346	489	617	766	940	1210
4	01483500	Leipsic River near Cheswold, DE	9.21	120	156	211	412	612	964	1320	1770	2340	3340
5	01483720	Puncheon Branch at Dover, DE	2.41	77	103	141	268	381	562	727	922	1150	1510
6	01484000	Murderkill River near Felton, DE	12.64	137	191	271	552	810	1230	1620	2070	2610	3470
7	01484002	Murderkill River Trib near Felton, DE	0.96	11	15	22	51	81	137	196	273	372	550
8	01484050	Pratt Branch near Felton, DE	3.1	35	49	70	153	241	404	573	796	1090	1600
9	01484100	Beaverdam Branch at Houston, DE	3.31	34	43	56	95	128	179	224	276	335	428
10	01484270	Beaverdam Creek near Milton, DE	6.21	25	31	39	65	86	118	146	179	216	274
11	01484300	Sowbridge Branch near Milton, DE	7.45	25	29	36	56	72	96	118	143	171	215
12	01484500	Stockley Branch at Stockley, DE	4.8	76	88	111	171	219	290	352	420	497	859
13	01484550	Pepper Creek at Dagsboro, DE	8.31	166	204	254	400	511	669	800	942	1100	1320
14	01484695	Beaverdam Ditch near Millville, DE	2.71	57	72	94	160	215	295	365	442	529	660
15	0148471320	Birch Branch at Sowell, MD	6.38	418	495	598	887	1110	1420	1680	1960	2270	2720
16	01484719	Bassett Creek near Ironshire, MD	1.39	66	92	133	293	460	765	1080	1490	2020	2960
17	01485000	Pocomoke River near Willards, MD	51.61	494	589	717	1100	1410	1860	2260	2700	3200	3960
18	01485500	Nassawango Creek near Snow Hill, MD	45.47	362	459	600	1070	1490	2170	2800	3560	4470	5940
19	01486000	Manokin Branch near Princess Anne, MD	3.98	75	100	145	270	340	425	480	540	590	660
20	01486100	Andrews Branch near Delmar, MD	4.54	64	78	95	143	179	230	271	316	363	432
21	01486980	Toms Dam Branch near Greensboro, MD	5.97	25	31	38	59	75	98	116	136	157	188
22	01487000	Nanticoke River near Bridgeville, DE	71.99	391	506	670	1200	1650	2360	2990	3720	4570	5880

Map No.	Station No.	Stream name	DA (mi^2)	Q1.25	Q1.5	Q2	Q5	Q10	Q25	Q50	Q100	Q200	Q500
23	01487900	Meadow Branch near Delmar, DE	2.73	52	61	72	99	118	143	162	182	202	229
24	01488500	Marshyhope Creek near Adamsville, DE	46.47	970	1300	1650	2400	2900	3400	3800	4200	4500	5000
25	01489000	Faulkner Branch at Federalsburg, MD	8.06	108	159	238	532	814	1290	1730	2270	2910	3940
26	01490000	Chicamacomico River near Salem, MD	16.96	123	162	219	407	573	834	1080	1360	1700	2220
27	01490600	Meredith Branch near Sandtown, DE	8.76	136	178	241	464	674	1030	1380	1800	2330	3210
28	01490800	Oldtown Branch at Goldsboro, MD	4.45	111	139	178	301	403	558	695	851	1030	1300
29	01491000	Choptank River near Greensboro, MD	113.8	1090	1460	1990	3590	4860	6680	8190	9820	11600	14100
30	01491010	Sangston Prong near Whiteleysburg, DE	1.94	34	48	70	157	248	418	594	824	1120	1650
31	01491050	Spring Branch near Greensboro, MD	3.76	38	53	77	169	265	441	622	856	1160	1690
32	01491500	Tuckahoe Creek near Ruthaburg, MD	87.67	1100	1370	1740	2890	3850	5320	6620	8100	9800	12400
33	01492000	Beaverdam Branch at Matthews, MD	6.05	156	209	289	577	857	1340	1810	2410	3140	4390
34	01492050	Gravel Run at Beulah, MD	8.53	56	73	98	186	267	404	535	695	889	1210
35	01492500	Sallie Harris Creek near Carmichael, MD	8	133	188	274	602	932	1510	2090	2820	3730	5280
36	01492550	Mill Creek near Skipton, MD	4.24	70	94	132	273	412	657	901	1210	1600	2260
37	01493000	Unicorn Branch near Millington, MD	20.67	198	264	360	668	929	1330	1680	2080	2530	3220
38	01493112	Chesterville Branch near Crumpton, MD	6.14	99	151	241	640	1110	2040	3080	4510	6450	10100
39	01493500	Morgan Creek near Kennedyville, MD	12.73	192	272	405	976	1640	2980	4490	6600	9540	15200
40	01494000	Southeast Creek at Church Hill, MD	12.6	400	500	640	1110	1420	1850	2250	2700	3100	3800
41	01494150	Three Bridges Branch at Centerville, MD	8.24	100	155	250	640	1050	2000	3000	4300	5800	8800

Attachment ECP-2. Watershed characteristics for 41 gaging stations in the Eastern Coastal Plain Region of Maryland.

Asoil used in the regression equations is based on the October 2021 SSURGO data from the NRCS Soil Survey web site. Land slope (LSLOPE) was estimated in ft/ft and converted to percent for use in the regression analysis. Map No. from Figure A3-2.

Map No.	Station No.	Period of record	Years of record	DA (mi^2)	LSLOPE (ft/ft)	LSLOPE (%)	Asoil (%)
1	01483155	2001-2016	16	2.03	0.02045	2.045	2.3
2	01483200	1952-2017	65	4.06	0.01898	1.898	34.1
3	01483290	1966-1975	10	0.91	0.01053	1.053	6.5
4	01483500	1943-1975, 2017	34	9.21	0.0161	1.61	14.1
5	01483720	1966-1975	10	2.41	0.01334	1.334	46.4
6	01484000	1932-33, 1960-99, 2007-09, 2017	35	12.64	0.00949	0.949	28.1
7	01484002	1966-1975	10	0.96	0.01201	1.201	86.2
8	01484050	1966-1975	10	3.1	0.01292	1.292	11.6
9	01484100	1958-2017	60	3.31	0.0073	0.73	29.6
10	01484270	1966-1980, 2002-2005	19	6.21	0.01195	1.195	73.5
11	01484300	1957-1978	22	7.45	0.01045	1.045	82.7
12	01484500	1943-2004	62	4.8	0.00805	0.805	26.2
13	01484550	1960-1975	16	8.31	0.00463	0.463	3.6
14	01484695	1999-2017	19	2.71	0.0062	0.62	6.6
15	0148471320	2000-2017	18	6.38	0.00619	0.619	36.2
16	01484719	2003-2009, 2011-2013	10	1.39	0.01248	1.248	2.0
17	01485000	1950-2004, 2007-2017	66	51.61	0.00667	0.667	20.0
18	01485500	1950-2017	68	45.47	0.00841	0.841	26.3
19	01486000	1951-1971, 1975-2017	64	3.98	0.00544	0.544	28.5
20	01486100	1967-1976	10	4.54	0.01044	1.044	26.4
21	01486980	1966-1975	10	5.97	0.00593	0.593	14.6
22	01487000	1935, 1943-2017	75	71.99	0.00768	0.768	18.2
23	01487900	1967-1975	9	2.73	0.00575	0.575	19.4
24	01488500	1972-2017	45	46.47	0.00636	0.636	6.3
25	01489000	1950-1991, 2011	42	8.06	0.00805	0.805	24.6
26	01490000	1951-1980, 2001-2017	46	16.96	0.00757	0.757	44.8
27	01490600	1966-1975	10	8.76	0.00643	0.643	3.5
28	01490800	1967-1976	10	4.45	0.00951	0.951	9.2
29	01491000	1948-2017	71	113.8	0.00922	0.922	11.5
30	01491010	1966-1975	10	1.94	0.00699	0.699	5.0
31	01491050	1967-1976	10	3.76	0.01008	1.008	14.7
32	01491500	1952-1956, 2001-2017	22	87.67	0.01189	1.189	21.1
33	01492000	1950-1981, 2010-2011	34	6.05	0.01794	1.794	6.8
34	01492050	1966-1976	11	8.53	0.01385	1.385	72.1

Map No.	Station No.	Period of record	Years of record	DA (mi^2)	LSLOPE (ft/ft)	LSLOPE (%)	Asoil (%)
35	01492500	1952-1981, 2001-2017	47	8	0.01948	1.948	11.7
36	01492550	1966-1976	11	4.24	0.01814	1.814	10.5
37	01493000	1948-2005, 2007-2017	69	20.67	0.0127	1.27	39.2
38	01493112	1997-2017	13	6.14	0.01857	1.857	0.5
39	01493500	1951-2005, 2007-2017	66	12.73	0.02445	2.445	1.8
40	01494000	1952-1965	14	12.6	0.01893	1.893	39.9
41	01494150	2007-2017	11	8.24	0.022	2.2	21.9

Attachment ECP-3. Watershed characteristics not used in the final regression equations for 41 gaging stations in the Eastern Coastal Plain Region of Maryland.

The forest cover and impervious area are indicative of land use conditions at the midpoint of the gaging station record.

Map No.	Station No.	Channel slope (ft/ft)	Basin Relief (ft)	Forest cover (%)	Impervious area (%)
1	01483155	20.7	38.61	4	4.4
2	01483200	13.5	38.35	30	4.3
3	01483290	10.4	18.28	5.5	1
4	01483500	9.4	35.82	9.6	1
5	01483720	13.2	18.77	10.4	1
6	01484000	6.2	25.47	14.7	4.3
7	01484002	13.1	22.05	8.3	1.8
8	01484050	11	27.25	9.2	1
9	01484100	5.2	15.4	19.8	2
10	01484270	7.4	26.96	33.9	5.8
11	01484300	8.3	27.28	33.7	2.6
12	01484500	4.8	18.38	12.3	5
13	01484550	3.7	25.75	5.7	1.9
14	01484695	3.9	5.03	6.2	4.8
15	0148471320	2.5	29.61	30.2	0.9
16	01484719	14	22.71	29.3	0.9
17	01485000	2.4	17.81	24.9	1.2
18	01485500	3.1	28.54	72.8	2.3
19	01486000	6.4	16.02	46.9	2.2
20	01486100	6.9	19.17	82.5	2.5
21	01486980	2.6	8.85	28.3	1
22	01487000	3.2	29.04	18.4	4.2
23	01487900	3.2	4.02	10.5	1
24	01488500	3.3	24.51	9.3	1.9
25	01489000	6.3	23.82	20.7	1.8
26	01490000	5.6	19.89	43.2	0.9
27	01490600	6	15.52	12.6	2.2
28	01490800	8.5	20.95	39	2
29	01491000	3.3	49.74	21.3	3.9
30	01491010	5.7	13.84	13.6	0.3
31	01491050	6	16.13	23.6	0.7
32	01491500	3.3	45.72	31	1.3
33	01492000	14.2	42.29	28.3	2.2
34	01492050	9.8	37.71	16.7	2
35	01492500	9.9	48.61	29.3	1.3

Map No.	Station No.	Channel slope (ft/ft)	Basin Relief (ft)	Forest cover (%)	Impervious area (%)
36	01492550	19	43.24	10	1.4
37	01493000	6.2	54.19	31.7	1.3
38	01493112	14.6	48.87	7.9	0.4
39	01493500	9.9	55.26	7.2	1
40	01494000	11.8	49.9	25.6	1.6
41	01494150	15.7	54.69	28.5	5.1

Attachment ECP-4. Comparison of the October 2021 SSURGO soils data to the legacy SSURGO soils data for 41 gaging stations in the Eastern Coastal Plain Region of Maryland.

Map No.	Station No.	Oct 2021 Asoil (%)	Oct 2021 Bsoil (%)	Oct 2021 Csoil (%)	Oct 2021 Dsoil (%)	Legacy Asoil (%)	Legacy Bsoil (%)	Legacy Csoil (%)	Legacy Dsoil (%)
1	01483155	2.3	94.5	1.4	1.6	0	96.8	1.3	1.6
2	01483200	34.1	33.2	12.6	20.0	0	67.3	12.3	20
3	01483290	6.5	29.4	22.6	41.4	0	37.9	21	41.1
4	01483500	14.1	47.6	11.7	26.6	0	65.9	8.4	25.4
5	01483720	46.4	21.7	24.3	7.6	0	76.6	3.2	20.2
6	01484000	28.1	12.0	8.8	51.2	16.1	30.4	11.4	42.1
7	01484002	86.2	6.8	0	7.0	77	16.2	3	3.8
8	01484050	11.6	64.8	9.2	14.5	1	84.6	3.3	11.1
9	01484100	29.6	5.0	0	65.3	19.1	12.1	22.9	45.9
10	01484270	73.5	6.4	3.8	16.3	31.1	15	44.1	9.7
11	01484300	82.7	0.6	1.2	15.4	49.2	39.4	2	8.6
12	01484500	26.2	17.2	2.6	52.7	4.4	51.6	11	33
13	01484550	3.6	7.1	0.1	89.2	0.9	64.8	3.5	30.8
14	01484695	6.6	8.4	2.4	82.5	1.4	46.6	8.9	13
15	0148471320	36.2	5.6	10.7	47.6	9.6	21.8	29.9	38.7
16	01484719	2.0	9.1	70.3	18.6	1.6	37.6	45	15.5
17	01485000	20.0	3.1	0.2	76.7	4.8	51	15.6	28.6
18	01485500	26.3	3.5	0.9	69.2	11.9	31.1	26.3	30.6
19	01486000	28.5	10.9	11.1	49.5	0.8	38.8	11.3	49
20	01486100	26.4	0.2	0	73.4	10.8	35.1	24.8	29.2
21	01486980	14.6	6.2	27.9	51.3	9.4	24.7	32.3	33.7
22	01487000	18.2	22.5	17.2	42.0	10.1	34.3	19.9	35.7
23	01487900	19.4	6.5	0.8	73.4	0	9.5	30.5	60
24	01488500	6.3	6.7	9.4	77.7	1.4	16.5	13	69.1
25	01489000	24.6	37.6	19.6	18.2	0.6	60.4	21.4	16.5
26	01490000	44.8	19.9	3.8	31.3	14.6	51.2	3.5	30.5
27	01490600	3.5	8.0	13.0	75.6	0.1	11	16.6	72.2
28	01490800	9.2	31.0	19.0	40.7	0	50.4	12.7	36.8
29	01491000	11.5	15.0	12.8	60.6	4.9	25.9	13.3	55.7
30	01491010	5.0	19.4	21.6	54.0	0	21.6	22.2	56.3
31	01491050	14.7	41.8	22.0	21.5	0.3	69.2	13.2	17.2
32	01491500	21.1	22.3	30.2	26.3	0.5	53.3	19	27
33	01492000	6.8	36.4	39.1	17.7	0.4	57.7	28.9	12.9
34	01492050	72.1	14.7	4.2	8.4	28.3	57.4	3.7	8.8
35	01492500	11.7	13.0	56.9	18.3	0	64.2	17.5	18.1
36	01492550	10.5	54.8	26.8	8.0	1	70.5	20.8	7.6
37	01493000	39.2	21.4	14.7	24.4	0.3	64.2	10.8	24.2

Map No.	Station No.	Oct 2021 Asoil (%)	Oct 2021 Bsoil (%)	Oct 2021 Csoil (%)	Oct 2021 Dsoil (%)	Legacy Asoil (%)	Legacy Bsoil (%)	Legacy Csoil (%)	Legacy Dsoil (%)
38	01493112	0.5	28.6	65.8	4.9	0.2	76.6	16.8	6
39	01493500	1.8	21.2	72.6	3.9	1.5	37.6	54.9	5.6
40	01494000	39.9	14.8	25.7	19.6	0.2	64.7	14.7	20
41	01494150	21.9	17.8	40.8	19.3	0	64.9	17	17.5

Attachment ECP-5. Computation of the Equivalent Years of Record for Regression Equations for the Eastern Coastal Plain Region.

Computational Procedure

The variance (standard error squared (SE^2)) of the x-year flood at a gaging station is estimated as

$$SE_x^2 = (S^2/N) * R_x^2 \qquad \text{(A3-11)}$$

where S is the standard deviation of the logarithms (log units) of the annual peak discharges at the gaging station, N is the actual record length in years and R_x is a function of recurrence interval x and skew (G) at the gaging station. The standard error increases as the recurrence interval increases, given the same record length.

In Equation A3-11, the standard error of the x-year flood at a gaging station is inversely related to record length N and directly related to the variability of annual peak flows represented by S (standard deviation) and G (skew). If the standard error of the x-year flood is interchanged with the standard error of estimate (SE) of the regression equation, then Equation A3-11 can be used to estimate the years of record needed to obtain that standard error of estimate. Rearranging Equation A3-11 and solving for N gives Equation A3-12 below.

The equivalent years of record of the regression estimate is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of the regional regression equation. Equivalent years of record is used to weight the gaging station and regression estimates. The equivalent years of record (N_r) of a regression equation is computed as follows (Hardison, 1971):

$$N_r = (S/SE)^2 * R^2 \qquad \text{(A3-12)}$$

where S is an estimate of the standard deviation of the logarithms of the annual peak discharges at the ungaged site, SE is the standard error of estimate of the regional regression estimates in logarithmic units, and R^2 is a function of recurrence interval and skew and is computed as (Stedinger and others, 1993):

$$R^2 = 1 + G*K_x + 0.5 *(1 + 0.75 * G^2) * K_x^2 \qquad \text{(A3-13)}$$

where G is an estimate of the average skew for a given hydrologic region, and K_x is the Pearson Type III frequency factor for the x-year flood and skew G.

Computational Details

The equivalent years of record are estimated for the regional regression equations and using Equations A3-12 and A3-13 and an estimate of the average standard deviation and average skew for all gaging stations in a given region. For the Eastern Coastal Plain Region, the average standard deviation (S) is 0.3104 log units and the average skew (G) is 0.330.

Recurrence Interval (years)	K_x value	SE^2 (log units squared)	Eq. years of record
1.25	-0.853519	0.03630	2.8
1.50			(3.0)
2	-0.054904	0.03105	3.2
5	0.821553	0.02803	6.8
10	1.311565	0.02694	10
25	2.18039	0.02749	19
50	2.225966	0.02855	19
100	2.565564	0.03079	21
200	2.881452	0.03398	23
500	3.280295	0.03900	24

Regression Equations for Rural and Urban Watersheds in the Western Coastal Plain

Introduction

Fixed region regression equations are used to estimate flood discharges for bridge and culvert design and floodplain mapping in Maryland by several state and local agencies. These empirical equations are developed based on relations between flood discharges at gaging stations and watershed characteristics that can be estimated from available digital data layers. For ungaged locations, the watershed characteristics are used in the regression equations to predict the flood discharges. The MDOT SHA uses the regression equations to primarily evaluate the reasonableness of flood discharges estimated using the TR-20 watershed model (Maryland Hydrology Panel, 2020). The objective of the current analysis is to update the Fixed Region regression equations for the Western Coastal Plain Region for estimating the 1.25-, 1.5-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year flood discharges using the following data:

- Annual peak flow data through the 2017 water year, if available,
- Flood frequency analyses using Bulletin 17C (England and others, 2019),
- Watershed characteristics computed using GISHydro, land use data for various time periods, 30-meter Digital Elevation Model (DEM) data dated May 2018, 1:100,000 National Hydrography Data (NHD) dated May 2018 and legacy SSURGO data in GISHydro, and
- SSURGO data downloaded from the Natural Resources Conservation Service (NRCS) Soil Survey web site in October 2021.

Documentation for GISHydro is located at (http://www.gishydro.eng.umd.edu/document.htm). Both sets of SSURGO data were evaluated as explanatory variables to determine which data set is most applicable for estimating flood discharges using regression equations.

Previous Studies in the Western Coastal Plain Region

Several studies have been completed since 1980 that developed regional regression equations for Maryland. Following is a brief description of the data used in the development of previous regression equations for the Western Coastal Plain Region (WCP) of Maryland:

- U.S. Geological Survey (USGS) Open-File Report 80-1016 (Carpenter, 1980) – used only drainage area as the explanatory variable and annual peak flow data through the 1977 water year,

- USGS Water-Resources Investigations Report 95-4154 (Dillow, 1996) – used drainage area and percent forest as explanatory variables and annual peak flow data through the 1990 water year,
- Maryland Hydrology Panel report (2006) and Moglen and others (2006) – used drainage area, percent impervious area for 1985 land use conditions, percent D soils based on **STATSGO** soils data as explanatory variables and annual peak flow data through the 1999 water year,
- Maryland Hydrology Panel report (2010) – used drainage area, percent impervious area for land use conditions near the middle of the gaging station record, sum of the percent of C and D soils based on **SSURGO** soils data as explanatory variables and annual peak flow data through the 2008 water year, and
- Maryland Hydrology Panel report (2020) – used drainage area, percent impervious area for land use conditions near the middle of the gaging station record, percent A soils based on **SSURGO** soils data as explanatory variables and annual peak flow data through the 2017 water year.

A water year is from October 1 to September 30 with the ending month determining the water year. For example, the 2017 water year is from October 1, 2016 to September 30, 2017. The 2022 update of the Western Coastal Plain regression equations just involved the update of the SSURGO soils data. The flood frequency data and other explanatory variables remained the same as in the 2020 analysis.

Flood Frequency Analyses at Gaging Stations

Flood frequency estimates were updated at the gaging stations with annual peak flow data through 2017, if available, using the USGS PeakFQ program (https://water.usgs.gov/software/PeakFQ/) that implements Bulletin 17C (England and others, 2019). For gaging stations that are still active in 2017, this represents an increase of nine years of record since 2008 (end of record used in the 2010 analysis). Flood data were compiled and analyzed for 27 gaging stations in the WCP: 11 active stations and 16 discontinued station; 15 rural stations (less than 10 percent impervious area) and 12 urban stations. The locations of the gaging stations in the WCP are shown in Figure A3-14 that defines the four major hydrologic regions in Maryland: Appalachian Plateau and Allegheny Ridge, Blue Ridge-Piedmont, Western and Eastern Coastal Plains. The gaging stations are numbered in Figure A3-14 in terms of their USGS downstream order with the station names and numbers identified in Attachment WCP-4 at the end of this report.

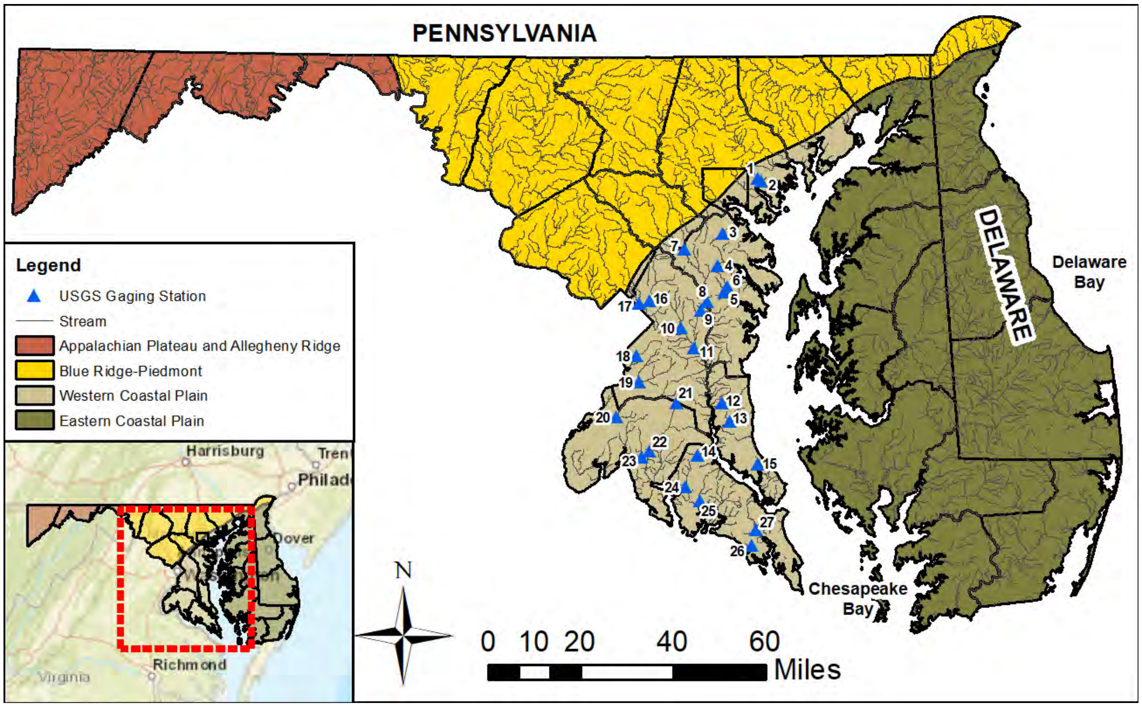


Figure A3-14: Location of 27 gaging stations in the Western Coastal Plain Region

Regional Skew Analysis

Bulletin 17C flood frequency guidance (England and others, 2019) recommends fitting a Pearson Type III distribution to the logarithms of the annual peak flows using the method of moments. The Pearson Type III distribution is defined by three sample moments: mean, standard deviation and skew of the logarithms of the annual peak flows. To reduce the uncertainty in the sample or station skew, Bulletin 17C recommends weighting the station skew with a regional or generalized skew determined from unregulated long-term records in the region. Frequency analyses were first performed using station skew to get an updated estimate of skew at all gaging stations in order to estimate a regional or generalized skew value. The analyses were performed at rural gaging stations with 19 or more years of record. Urban gaging stations were not used because each site represents different land use conditions over the period of record. There are only eight rural gaging stations in the WCP with 19 or more years of record. The mean skew for the eight stations was 0.380 and the standard deviation of the skew was 0.386. This is a small sample for estimating skew so the eight stations in the WCP were combined with 15 rural stations from the Eastern Coastal Plain (ECP) Region with 19 or more years of record. The mean skew for the 23 stations was also 0.38 and the standard deviation or standard error of the skew was 0.38, essentially the same as for the limited sample of WCP stations. The mean skew of 0.38 with standard error of 0.38 was used to weight with the station skew in the final frequency analyses for all rural stations in the WCP. For the urban gaging stations where the impervious area exceeded 10 percent at the midpoint of the gaging record, station skew was used in the final frequency analyses.

Trend Analysis

The time series of annual peak flows exhibited upward trends at several of the WCP gaging stations because of increasing urbanization and/or major floods near the end of record. A common test for trend in a time series is the Mann-Kendall test (Helsel and Hirsch, 2002). This test uses Kendall's tau as the test statistic to measure the strength of the monotonic relation between annual peak flows and the year in which it occurred. The Mann-Kendall test is nonparametric and does not require the data to conform to any specific statistical distribution and does not utilize the actual magnitude of the peak flows. All peak flows are compared to those following it in time and the number of increasing and decreasing flows are recorded. The test statistic is based on the number of increasing or decreasing flows with time.

The USGS PeakFQ program includes the Mann-Kendall test and Kendall's test statistic is provided as part of the standard output. Time series graphs of annual peak flows for the 11 active gaging stations are given in Attachment WCP-1 along with Kendall's tau and comments on what is causing any upward trend in annual peak flows. All 11 active stations have 22 years or more of record with eight of the stations having more than 40 years of record. Time series were not provided for the discontinued stations because those records generally ended in 1990 or before and generally are rural watersheds with limited urbanization and generally short records. Hence, trends in the annual peak flows were not an issue or the record was too short to adequately evaluate if a trend existed for the discontinued stations.

Of the 11 active stations shown in Attachment WCP-1, seven stations are urban watersheds where impervious area was greater than 10 percent near the midpoint of the gaging station record. Six of those urban stations had statistically significant upward trends (at the five percent level of significance) when analyzing the full record due to increasing urbanization and major floods near the end of the record. The trend was accommodated as following for the six urban stations:

- **Sawmill Creek at Glen Burnie (station 01589500) –** drainage area = 5.04 square miles with record from 1945 to 2017. The upward trend is related to increasing urbanization and the peak of record in 2014. The more homogeneous period from 1984 to 2017 was used in the final frequency analysis.
- **Patuxent River near Bowie (station 01594440) –** drainage area = 350.21 square miles with record from 1972 to 2017. The upward trend is related to increasing urbanization and four large floods from 2006 to 2014. A time-varying mean approach was used for the final frequency analysis as described in Kilgore and others (2019). This approach is briefly discussed in Attachment WCP- 2.
- **Western Branch at Upper Marlboro (station 01594526) –** drainage area = 89.38 square miles with record from 1986 to 2017. The upward trend is related to

increasing urbanization and large floods in 2008 and 2011. A time-varying mean approach was used for the final frequency analysis.

- **Northeast Branch Anacostia River at Riverdale (01649500) –** drainage area = 73.2 square miles with record from 1933 to 2017. The upward trend is related to increasing urbanization and a large flood in 2006. A time-varying mean approach was used for the final frequency analysis.
- **Northwest Branch Anacostia River near Hyattsville (01651000) –** drainage area = 49.33 square miles with record from 1939 to 2017. The upward trend is related to increasing urbanization and large floods in 2006 and 2014. The more homogeneous period 1972 to 2017 was used in the final frequency analysis.
- **Piscataway Creek at Piscataway (01653600) –** drainage area = 39.43 square miles with record from 1966 to 2017. The upward trend is related to increasing urbanization and five large floods from 1999 to 2014. For this station, the upward trend was barely significant at the five percent level and the bigger issue was that the Pearson Type III distribution did not fit the data very well. The final frequency analysis was based on a graphical analysis.

The six stations with significant upward trends in annual peak flows are all urban watersheds and the increasing urbanization with time contributes to that trend. None of the long-term rural stations in the WCP exhibited significant trends.

There was one small-stream station, Clark Run near Bel Alton (01660930), where rainfall-runoff modeling results were available from an earlier study by Carpenter (1980). The flood discharges as determined by Carpenter (1980) were used in this study because these estimates were more reasonable than estimates based on 11 years of data (1966-76).

For the active gaging stations, nine additional years of record were added to analysis since 2008, the end of the record used in the 2010 analysis. There were some major floods in the period 2009 to 2017 particularly major floods in 2011 (Tropical Storm Lee or Hurricane Irene) and 2014. In general, the flood discharges increased for the active stations implying that the new regression equations may provide increased estimates. The updated flood discharges for the 1.25-, 1.5-. 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year events are given in Attachment WCP-3 for all 27 stations.

Watershed Characteristics Evaluated for the Regression Analysis

The watershed characteristics evaluated for the regression analysis included those that were statistically significant in previous regression analyses and were estimated using the digital data in GISHydro (http://www.gishydro.eng.umd.edu/document.htm) as of May 2018 and updated SSURGO soils data added in October 2021. The watershed characteristics included:

- Drainage area (DA), in square miles, computed as the number of pixels covering the watershed area times the pixel's area or cell size,

- Channel slope (CSL), in feet per mile, computed as the difference in elevation between two points located 10 and 85 percent of the distance along the main channel from the outlet divided by the distance between the two points,
- Land slope (LSLOPE), in feet per feet, sometimes referred to as watershed slope, computed as the average of all neighborhood slopes determined along the steepest direction of flow for all pixels in the watershed (used as a percentage in the regression analysis),
- Basin Relief (BR), in feet, computed as the average elevation of all pixels within the watershed minus the elevation at the outlet of the watershed,
- Percent impervious area (IA) near the middle of the gaging station record, impervious area is available for 1985, 1990, 1997, 2002 and 2010 land use conditions,
- Forest cover (FOR), in percent of the drainage area at the middle of the gaging station record, available for 1985, 1990, 1997, 2002, and 2010 land use conditions,
- Percent A, B, C and D SSURGO soils based on the legacy data in GISHydro, and
- Percent A, B, C and D SSURGO soils based on soils data downloaded from the NRCS web site in October 2021.

The legacy SSURGO soils data in GISHydro are shown in Figure A3-15 for the four Hydrologic Soil Groups A, B, C and D where A has the high infiltration rate and D the lowest infiltration rate. These data were added to GISHydro over time and were representative of different dates for each county in the state.

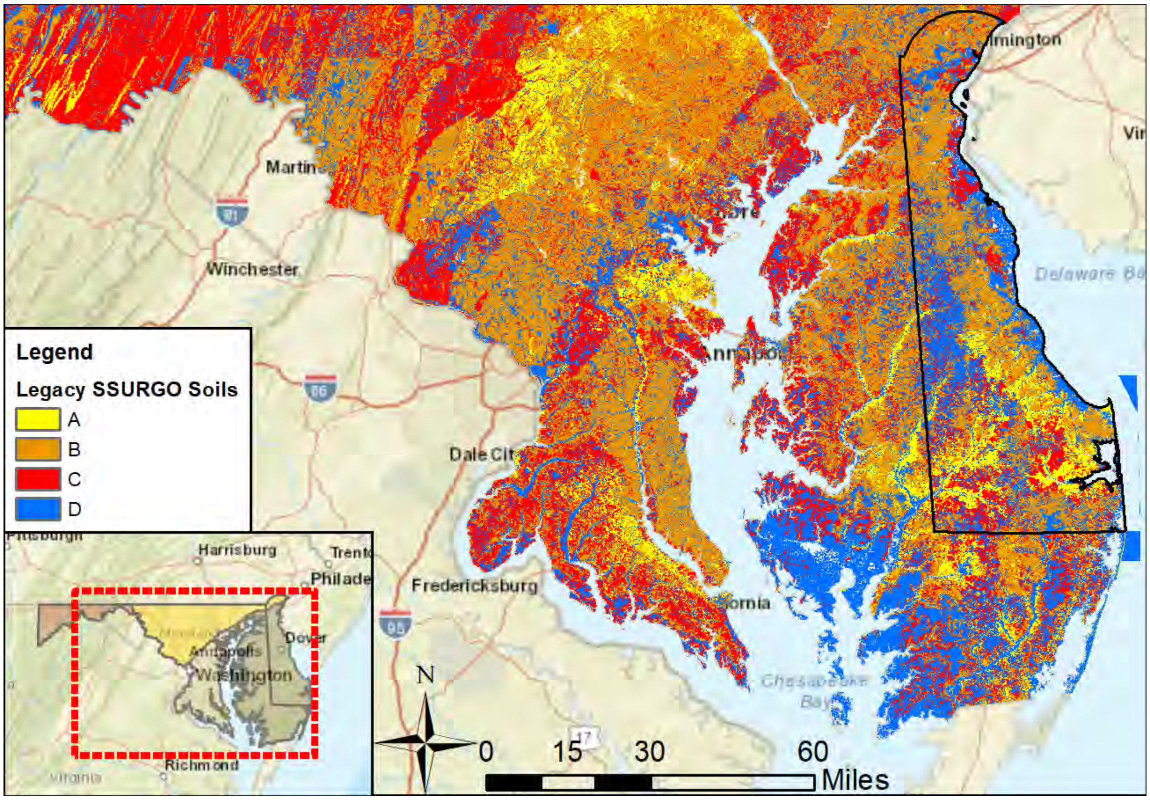


Figure A3-15: Legacy SSURGO soils data in GISHydro

The SSURGO soils data downloaded from the NRCS soil survey web site in October 2021 using the Dominant Condition approach for aggregating the soils data are shown in Figure A3-16. The NRCS procedures for estimating the Hydrologic Soils Groups (HSGs) were updated prior to 2009 and documented in the NRCS Part 630 Hydrology, National Engineering Handbook, Chapter 7, Hydrologic Soils Group (HSG) dated January 2009. The calculations for the new HSGs were completed for Maryland in 2014 and the updated HSGs were posted to the NRCS Web Soil Survey database in 2016. The SSURGO soils on the NRCS Web Soil Survey site are updated annually and data available in October 2021 were used in this current analysis. The new criteria for assigning HSGs use soil properties that influence runoff potential such as:

- Depth to a seasonal high-water table,
- Saturated hydraulic conductivity (Ksat) after prolonged wetting, and
- Depth to a layer with a very slow water transmission rate.

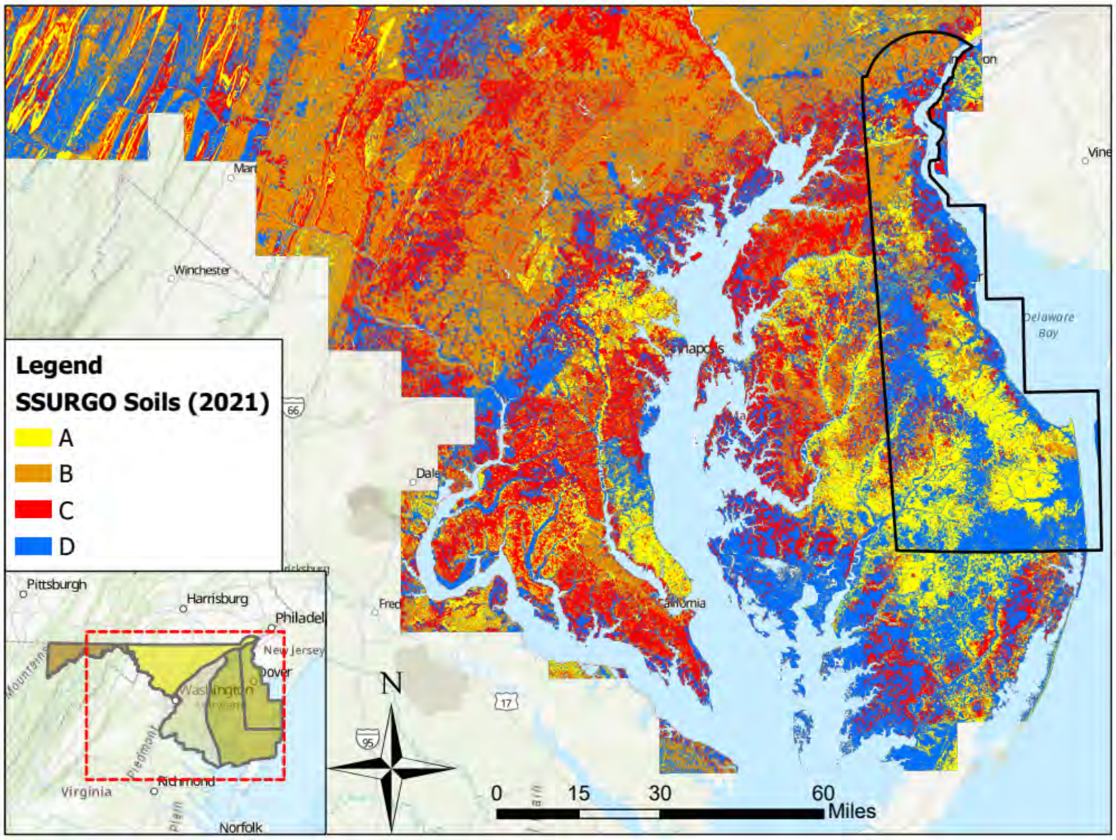


Figure A3-16: The October 2021 SSURGO soils data

Development of Regression Equations

Multiple regression analyses were performed using all 27 gaging stations and the list of explanatory variables discussed earlier using the Statistical Analysis System (SAS) computer software developed by the SAS Institute, Inc., Cary, NC (https://www.sas.com/en_us/home.html). In this process, four gaging stations (all discontinued stations) were identified as outliers because the flood discharges for these stations were low for the size of the drainage area. A brief description follows as to why these stations were considered outliers:

1. **Dorsey Run near Jessup (01594400)**, 11.91 square miles, 16.7 percent IA (1985), 7.9 percent A soils (May 2018), 40.9 percent forest cover. There are 20 years of record from 1949-68, and 2009. Largest flood is 1,730 cfs in 2009. The gaging station 100-year flood is 2,690 cfs and the regression estimate is 5,710 cfs when this station is in the analysis.
2. **Western Branch near Largo (01594500)**, 30.04 square miles, 11.4 percent IA (1985), 19.8 percent A soils (May 2018), 41.6 percent forest cover. There are 25 years of record from 1950-74. Largest flood is 1,760 cfs in 1971. The gaging

station 100-year flood is 2,600 cfs and the regression estimate is 7,270 cfs when this station is in the analysis.

3. **Killpeck Creek at Huntersville (01594710),** 3.46 square miles, 7.8 percent IA (1990), 60.3 percent A soils (May 2018), 60.4 percent forest cover. There are 12 years of record from 1986-97. Largest flood is 255 cfs in 1990. The gaging station 100-year flood is 356 cfs and the regression estimate is 816 cfs when this station is in the analysis.
4. **Glebe Branch at Valley Lee (01661430),** 0.24 square miles, 2.1 percent IA (1985), 2.6 percent A soils (May 2018), 42.5 percent forest cover. There are 11 years of record from 1968-78. Largest flood is 110 cfs in 1969. The gaging station 100-year flood is 148 cfs and the regression estimate is 382 cfs when this station is in the analysis.

The regression analysis proceeded with 23 gaging stations. Separate sets of regression equations were developed using the legacy SSURGO soils data and the October 2021 SSURGO soils data. All flood discharges and topographic explanatory variables were transformed to logarithms prior to the regression analysis because tradition has shown that the logarithms of flood discharges are linearly related to logarithms of the watershed characteristics. The percent impervious area and percent soils data were evaluated for the logarithmic transformed data and untransformed data. Based on several regression analyses, the following observations are pertinent:

- The percent A, B and D soils based on the legacy SSURGO data in GISHydro were **NOT** statistically significant in the same equation with percent impervious area. The percent C soils based on legacy SSURGO data was statistically significant (range of C soils from 0.8 to 64.6 percent).
- The percent B, C and D soils based on the October 2021 SSURGO data were **NOT** statistically significant in the same equation with impervious area.
- The percent A soils based on the October 2021 SSURGO was statistically significant at the five percent level in the same equation with percent impervious area (both logs and untransformed) from the 1.25- to the 500-year flood. Range of the October 2021 A soils is 1.7 to 85.2 percent.
- The percent A soils data are a better predictor when **NOT** transformed to logarithms. Note in Figure A3-17, the correlation between the logarithm of the 100-year discharge (lq100) and A soils is highest for the untransformed A soils data (Acond = October 2021 soils data). The “l” in Figure A3-17 before the variable name denotes logarithm.
- The percent forest cover is not statistically significant when used in the same equation with percent impervious area due to their high correlation (-0.76 for log transformed values as shown in Figure A3-17). Forest cover and impervious area are based on the same date of the land use data.
- The topographic/slope variables, land slope and basin relief, are not statistically significant when used in the same equation with drainage area. Basin relief is

highly correlated with drainage area (0.81) as shown in Figure A3-17. Land slope is not highly correlated with drainage area (0.10) as shown in Figure A3-17 so that makes it a better explanatory variable.

- The percent impervious area **NOT** transformed to logarithms is a better predictor up to the 10-year flood; percent impervious area transformed to logarithms is a better predictor for the 25- to 500-year flood. The log transformation is considered best for developing the regression equations since the larger floods are more important for design. Note in Figure A3-17, the correlation between the logarithm of the 100-year discharge (lq100) and impervious area is highest for the logarithmic transformed data (lia).

The correlation matrix of the explanatory variables and the logarithm of the 100-year discharge (lq100) is given in Figure A3-17 for 23 gaging stations. Acond is the SSURGO soils data dated October 2021. Only the Acond soils data are shown in Figure A3-17 since it is statistically significant. An “l” at the beginning of the variable number refers to the logarithms of the data. The highly significant or most important correlations are highlighted in yellow.

Pearson Correlation Coefficients, N = 23
Prob > |r| under H0: Rho=0

	lq100	lda	lbr	lslope	lacond	acond	lia	ia	lfor
lq100	1.00000	0.82244	0.73297	-0.15114	-0.59306	-0.62511	0.49194	0.30455	-0.17808
		<.0001	<.0001	0.4912	0.0029	0.0014	0.0171	0.1577	0.4163
lda	0.82244	1.00000	0.80679	0.10460	-0.20208	-0.18344	0.20169	0.04266	0.15669
	<.0001		<.0001	0.6348	0.3551	0.4021	0.3561	0.8467	0.4752
lbr	0.73297	0.80679	1.00000	0.32497	-0.30352	-0.22867	0.38170	0.26857	-0.12057
	<.0001	<.0001		0.1303	0.1591	0.2939	0.0723	0.2153	0.5837
lslope	-0.15114	0.10460	0.32497	1.00000	0.28705	0.35685	-0.24348	-0.26892	0.27732
	0.4912	0.6348	0.1303		0.1842	0.0946	0.2629	0.2147	0.2001
lacond	-0.59306	-0.20208	-0.30352	0.28705	1.00000	0.88568	-0.29953	-0.26644	0.42008
	0.0029	0.3551	0.1591	0.1842		<.0001	0.1650	0.2191	0.0460
acond	-0.62511	-0.18344	-0.22867	0.35685	0.88568	1.00000	-0.29864	-0.20494	0.37669
	0.0014	0.4021	0.2939	0.0946	<.0001		0.1663	0.3482	0.0764
lia	0.49194	0.20169	0.38170	-0.24348	-0.29953	-0.29864	1.00000	0.92246	-0.75753
	0.0171	0.3561	0.0723	0.2629	0.1650	0.1663		<.0001	<.0001
ia	0.30455	0.04266	0.26857	-0.26892	-0.26644	-0.20494	0.92246	1.00000	-0.76281
	0.1577	0.8467	0.2153	0.2147	0.2191	0.3482	<.0001		<.0001
lfor	-0.17808	0.15669	-0.12057	0.27732	0.42008	0.37669	-0.75753	-0.76281	1.00000
	0.4163	0.4752	0.5837	0.2001	0.0460	0.0764	<.0001	<.0001	

Figure A3-17: Correlation matrix of logarithm of 100-year discharge (lq100) and potential explanatory variables for 23 gaging stations

The statistical significance of the explanatory variables in a regression analysis is dependent on the correlation with other variables. If two variables are highly correlated, then only one of the variables will be significant in reducing the standard error of the regression equation.

Based on the regression analyses, the October 2021 SSURGO soils is a better predictor than the legacy SSURGO data for the WCP Region. The October 2021 SSURGO data are now the default SSURGO data in GISHydro. The three most significant explanatory variables for the WCP are: logarithm of drainage area, logarithm of percent impervious area, and percent A soils (October 2021 SSURGO data with no transformation). The watershed characteristics used in the regression analysis are given in Attachment WCP-4 for all 27 stations. The date of the land use data for determining impervious area is also given in Appendix 4 along with the period of record for the gaging stations. The watershed characteristics not used in the final regression equations are given in Attachment WCP-5. The October 2021 and legacy SSURGO soils data are compared in Attachment WCP-6.

The regression equations are based on 23 stations for the WCP Region and the following variables where DA = drainage area, in square miles, ranging from 0.96 to 350.21 square miles; IA = impervious area, in percent, ranging from 0.0 to 36.8 percent; and Acond = the October 2021 A soils data, in percent, ranging from 1.7 to 85.2 percent. Forest cover is an important explanatory variable for rural watersheds but is highly correlated with impervious area (-0.76), and not statistically significant in the regression analysis which means the standard error is not significantly reduced by including forest cover in the equations. In the regression analysis, forest cover data were used for the same date as the impervious area land use data. For example, if 1985 data were used for impervious area, then 1985 data were used for forest cover. Comparisons were made for the 10- and 100-year estimates for equations with and without forest cover and the regression estimates were essentially the same for the 23 gaging stations used for the final equations.

All the explanatory variables in the final equations are statistically significant at the five percent level of significance for all recurrence intervals. Two measures of accuracy are standard error of estimate in percent and equivalent years of record. The standard error of estimate is the standard deviation of the residuals about the regression equation and is indicative of how well the equations fit the gaging station estimates. The equivalent years of record (Eq. years)) is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of estimate of the regression equation. Eq. years is used to weight the gaging station estimates with the regression estimates following the approach documented by Dillow (1996) and described in Chapter 2 of this report. The computation of equivalent years of record is described in Attachment WCP-7.

Equation	**Standard error (%)**	**Eq. years**	
$Q_{1.25} = 33.0\ DA^{0.709}\ (IA+1)^{0.389}\ 10^{-0.00734*Acond}$	50.8	2.3	(A3-14)
$Q_{1.5} = 46.7\ DA^{0.696}\ (IA+1)^{0.374}\ 10^{-0.00778*Acond}$	49.2	2.4	(A3-15)
$Q_{2} = 69.0\ DA^{0.680}\ (IA+1)^{0.357}\ 10^{-0.00819*Acond}$	48.4	2.5	(A3-16)
$Q_{5} = 164.1\ DA^{0.645}\ (IA+1)^{0.321}\ 10^{-0.00908*Acond}$	40.8	6.4	(A3-17)
$Q_{10} = 272.0\ DA^{0.630}\ (IA+1)^{0.303}\ 10^{-0.00960*Acond}$	34.7	13	(A3-18)
$Q_{25} = 493.1\ DA^{0.592}\ (IA+1)^{0.279}\ 10^{-0.00994*Acond}$	29.2	28	(A3-19)
$Q_{50} = 736.9\ DA^{0.567}\ (IA+1)^{0.262}\ 10^{-0.01010*Acond}$	27.0	43	(A3-20)
$Q_{100} = 1{,}065.3\ DA^{0.547}\ (IA+1)^{0.248}\ 10^{-0.01022*Acond}$	28.0	50	(A3-21)
$Q_{200} = 1{,}529.3\ DA^{0.521}\ (IA+1)^{0.234}\ 10^{-0.01030*Acond}$	32.6	45	(A3-22)
$Q_{500} = 2{,}418.7\ DA^{0.489}\ (IA+1)^{0.215}\ 10^{-0.01041*Acond}$	42.7	34	(A3-23)

The impervious area at the middle of the gaging station record was used in developing the regression equations but the impervious area for existing land use conditions (latest data are based on 2010) should be used in application of the equations for ungaged watersheds.

For Equations A3-14 to A3-23, the drainage area exponent decreases with an increasing recurrence interval, consistent with earlier results. A possible explanation is that the storm rainfall for the larger storms varies considerably across a watershed and does not have a uniform impact across the entire watershed (that is, the effective drainage area is less). The exponent on impervious area decreases with increasing recurrence interval, implying that impervious area has less influence as the floods become larger. This is a well-known result in which soils become more saturated for the larger floods, and impervious area has relatively less impact on runoff volumes. The exponent on Acond increases from the 1.25-year flood up to the 25-year flood and then is fairly constant up to the 500-year flood. This implies the soils become more significant as storm rainfall increases until the 25-year flood when the soils may become saturated.

The higher standard errors for the shorter recurrence interval (1.25- to 5-year) floods imply that some unknown explanatory variables other than drainage area, the percentage of impervious area, and percentage of A soils influence these floods. The time-sampling error (error in T-year flood discharge) is actually less for these smaller floods, so one would expect a lower standard error in the regression analysis. Instead, the standard errors of the regression equations for the smaller events are influenced by the model error, indicating that other important explanatory variables may be missing from the equations.

The 100-year regression estimates (Q100 from Equation A3-21) are plotted versus the 100-year gaging station estimates in Figure A3-18 for the equation based on 23 stations. The trend (best-fit) line is close to the equal discharge line indicating the regression estimates are reasonably unbiased for all gaging stations.

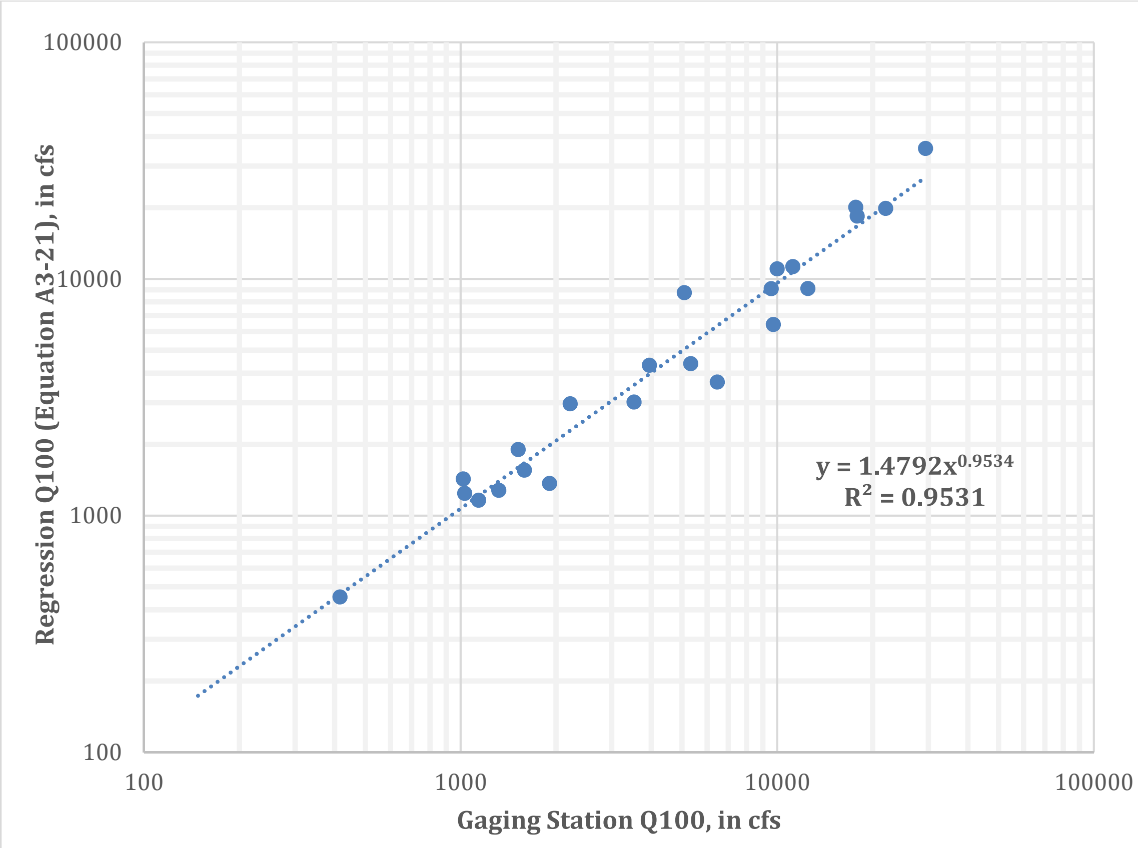


Figure A3-18: The 100-year regression estimates from Equation A3-21 plotted versus the 100-year estimates based on gaging station data for 23 stations in the Western Coastal Plain Region

The 10-year regression estimates (Q10 from Equation A3-18) are plotted versus the 10-year gaging station estimates in Figure A3-19 for the equations based on 23 stations. The trend line is close to the equal discharge line indicating the regression estimates are reasonably unbiased. For the larger discharges, there is a tendency for the regression equation to underestimate the gaging station data by about 10 percent. For a gaging station estimate of 10,000 cfs, the regression equation is predicting about 9,000 cfs, on average.

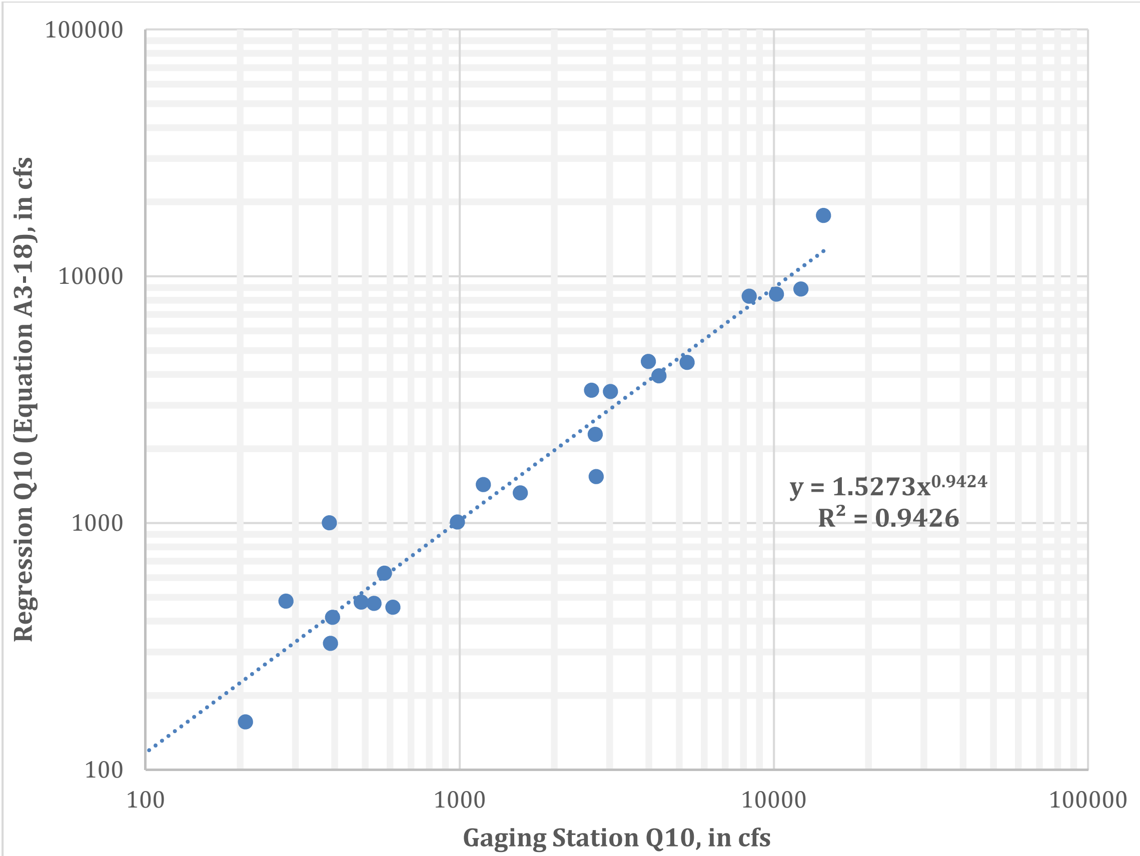


Figure A3-19: The 10-year regression estimates from Equation A3-18 plotted versus the 10-year estimates based on gaging station data for 23 stations in the Western Coastal Plain Region

The 2010 regression equations are compared to Equation A3-21 in Figure A3-20 for the 100-year flood using data for 23 gaging stations. There is a tendency for Equation A3-21 estimates to be higher than the 2010 estimates for the larger discharges. For example, when Equation A3-21 is predicting 10,000 cfs for the 100-year discharge, the 2010 regression equation is predicting about 8,000 cfs, on average.

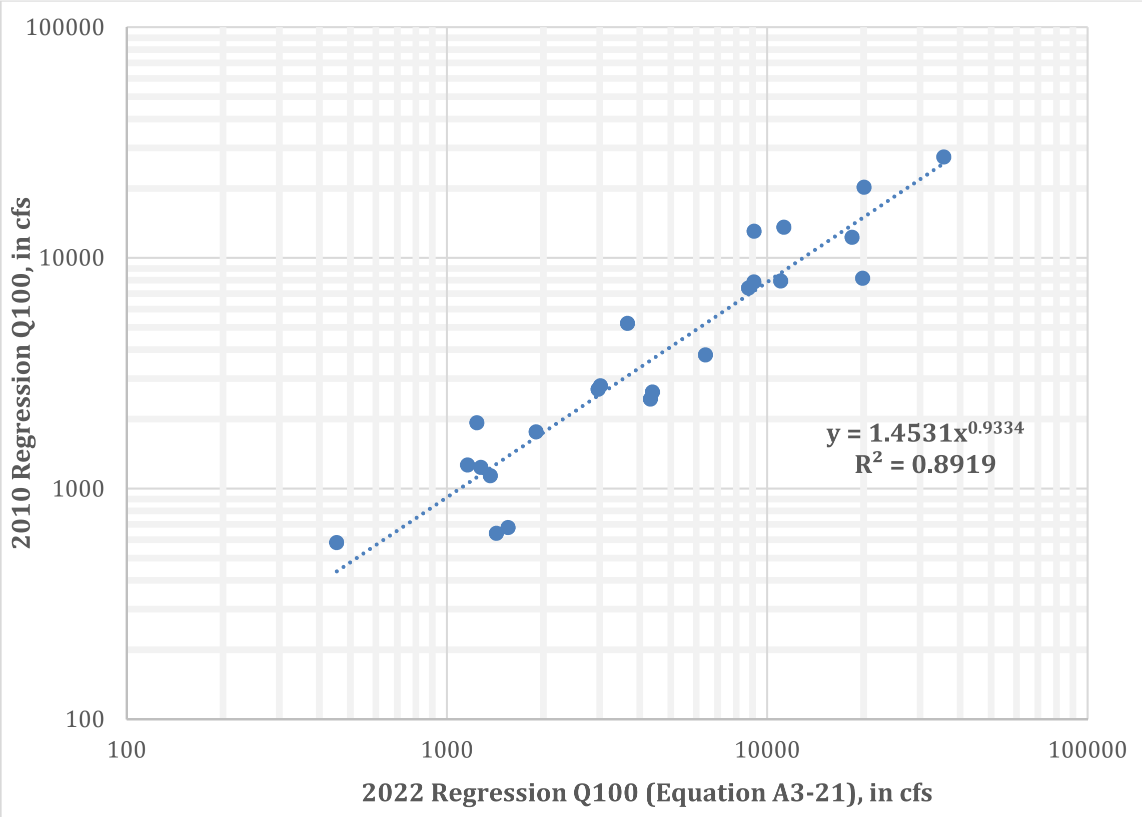


Figure A3-20: Comparison of the 100-year flood discharges for the 2010 and 2022 regression equations

The 2010 regression equations are compared to estimates from Equation A3-18 in Figure A3-21 for the 10-year flood using data for 23 gaging stations. There is a tendency for Equation A3-18 estimates to be higher than the 2010 estimates for the larger discharges. For example, when Equation A3-18 predicts 10,000 cfs for the 10-year discharge, the 2010 regression equation is predicting about 7,000 cfs, on average.

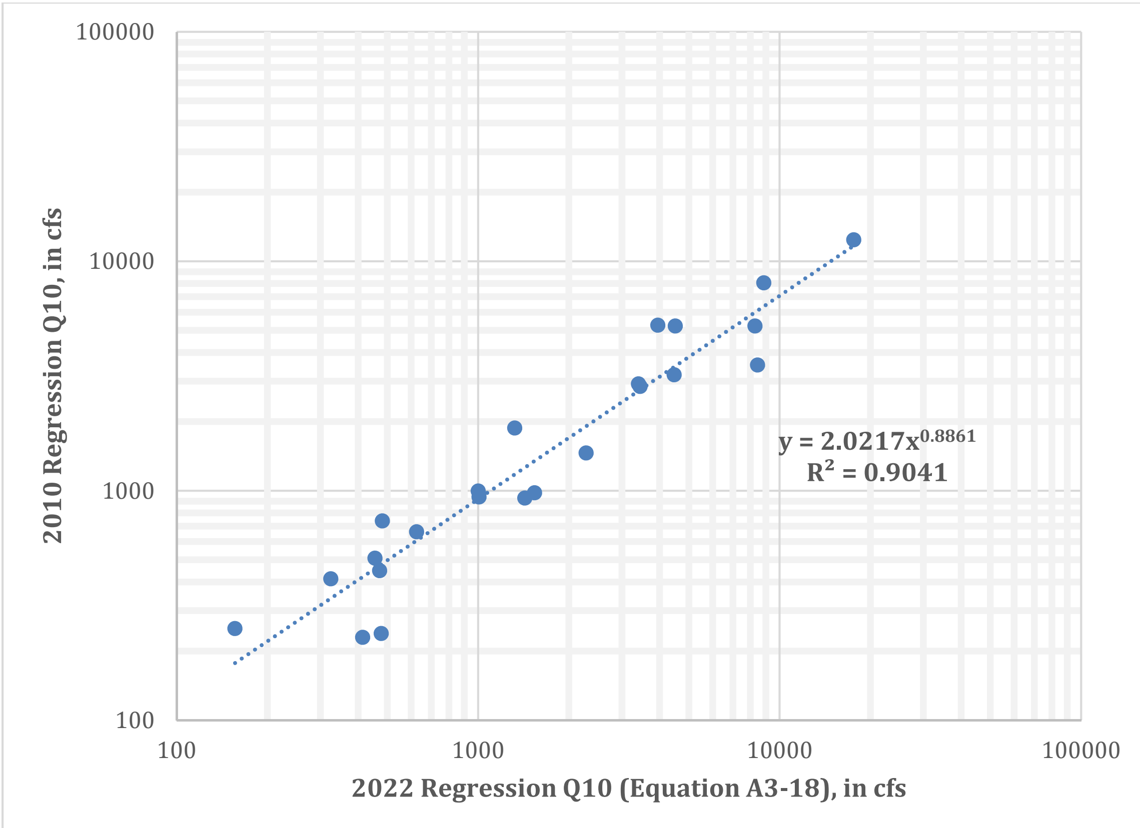


Figure A3-21: Comparison of the 10-year flood discharges for the 2010 and 2022 regression equations

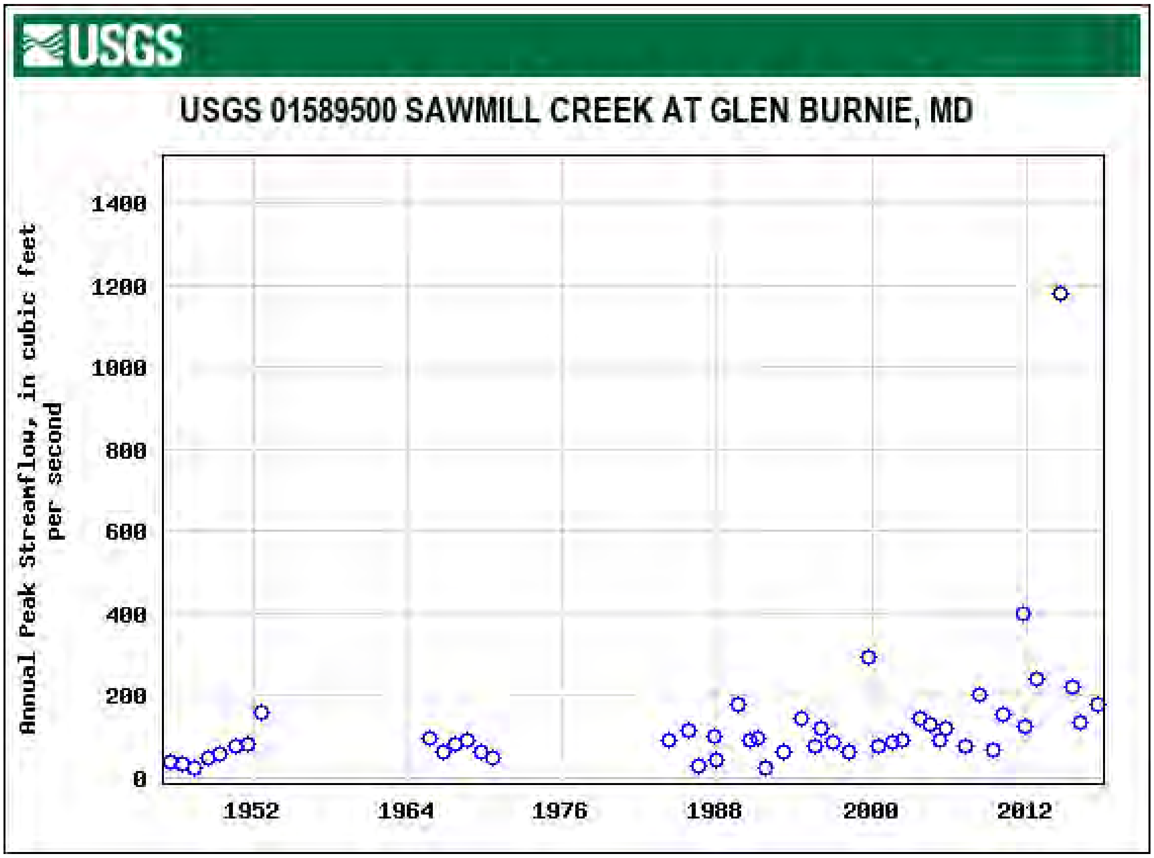
Summary and Conclusions

The Fixed Region regression equations for the Western Coastal Plain Region of Maryland were updated using annual peak flow data through the 2017 water year using Bulletin 17C (England and others, 2019). The regression equations were based on 23 stations (11 active and 12 discontinued stations) and the statistically significant explanatory variables were drainage area, in square miles; percent impervious area at the midpoint of the gaging station record; and percent of A soils based on SSURGO data downloaded from the NRCS soil survey web site in October 2021. The legacy SSURGO data in GISHydro and the October 2021 SSURGO data were both evaluated in the regression analysis to determine which set of soils data provided the most accurate regression equations. The October 2021 SSURGO data provided the most accurate regression equations and is now the default soils data in GISHydro.

Of the 11 active gaging stations, six stations had statistically significant upward trends in the annual peak flow data due to increasing urbanization and major floods near the end of the record. These trends were accounted for by using a time-varying mean and using a more homogeneous period of record.

The regression estimate for the 100-year discharge was compared to gaging station data and shown to be unbiased. For the 10-year discharge, the regression estimates tend to be about 10 percent less than the gaging station estimates for the largest discharges (watersheds). The 2022 regression equations for the 10- and 100-year flood discharges were also compared to the 2010 regression equations that were based on annual peak flow data through the 2008 water year. The 2022 regression estimates tend to be higher than the 2010 estimates, particularly for the larger watersheds. This is consistent with the increase in flood discharges for the active gaging stations that tend to be larger watersheds. The increase in flood discharges for the active stations are related to increasing urbanization and major floods in 2011 and 2014 at many of the stations.

Attachment WCP-1. Time series graphs of the annual peak flows for the 11 active gaging stations in the Western Coastal Plain Region.



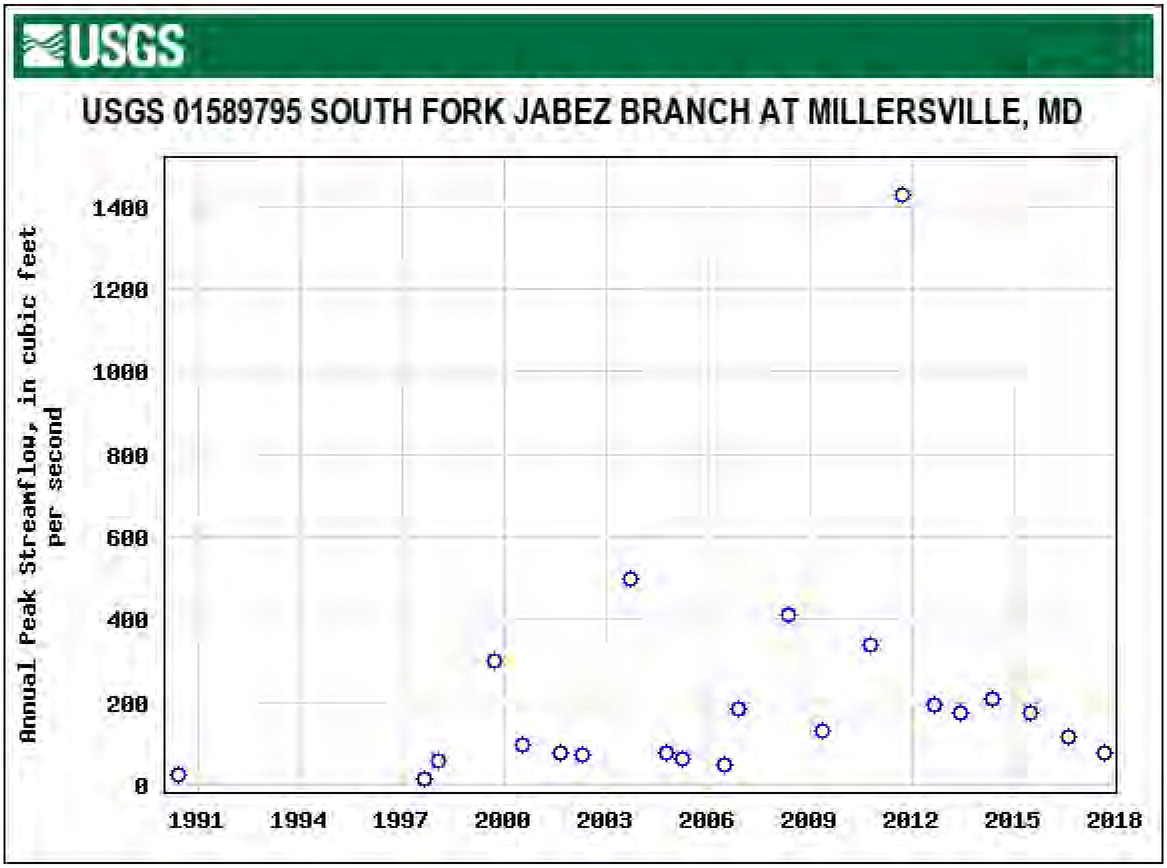
48 years of record – 1945 to 2017; drainage area = 5.04 square miles

Kendall's Tau = 0.455 for 1945 to 2017; Kendall's Tau = 0.360 for 1984 to 2017

P value = 0.00 for 1945 to 2017; P value = 0.003 for 1984 to 2017

Conclusion: Significant upward trend in annual peak flows since the P value is less than 0.05 (five percent level of significance).

Comments: This is an urban watershed where the impervious area went from 11.7 percent in 1985 to 29.7 percent in 2002 to 33.5 percent in 2010. The flood of record is 1180 cfs in 2014 near the end of the record. These are contributing factors to the upward trend. The more homogeneous period 1984-2017 was used for the frequency analyses (34 years). Still an upward trend due primarily to the 2014 flood. Used IA02 = 29.7 percent in the regression analysis.



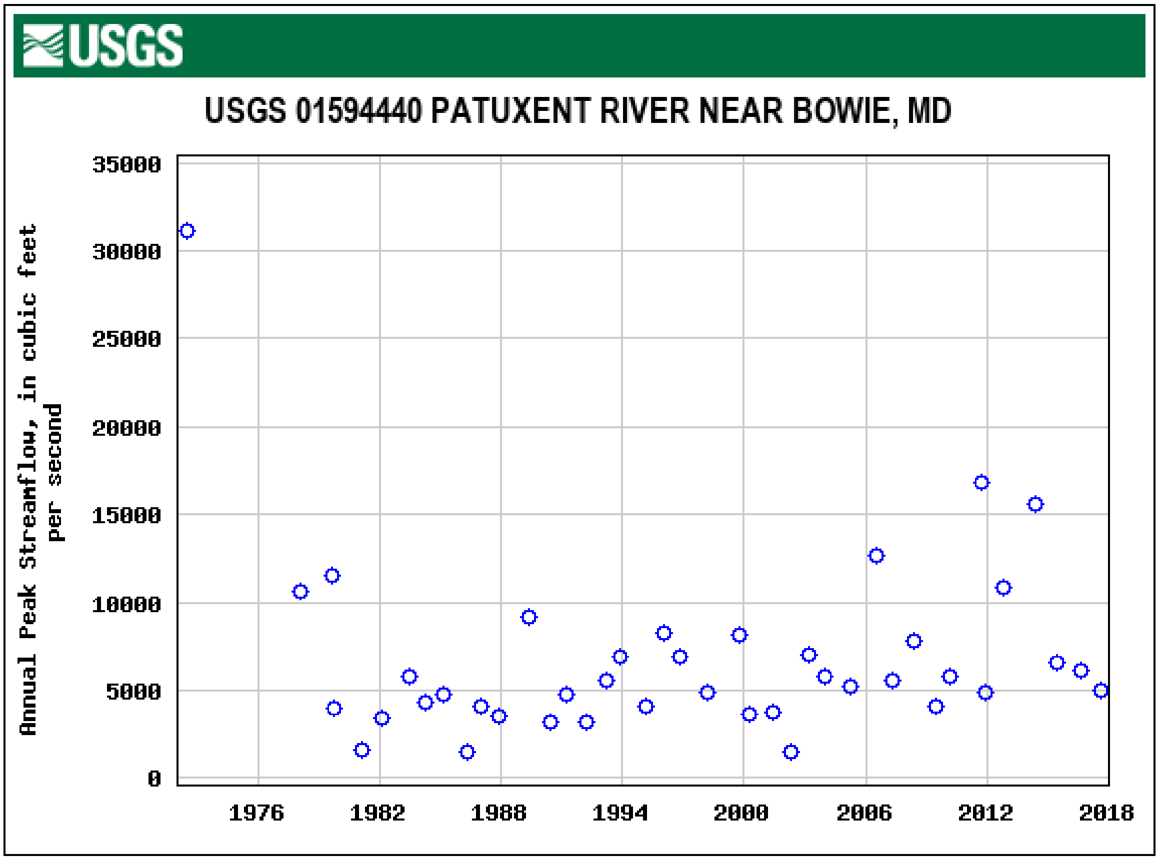
22 years of record – 1990, 1997-2017; drainage area = 0.96 square miles

Kendall's tau = 0.0.264

P value = 0.091

Conclusion: No significant upward trend in annual peak flows since the P value is greater than 0.05 (five percent level of significance).

Comments: No upward trend even though the flood of record (1,490 cfs) occurred in 2011 near the end of the record. IA85 = 8.2%, IA02 = 16.8% and IA10 = 20.0%. IA10 = 20.0% was used in the regression analysis.



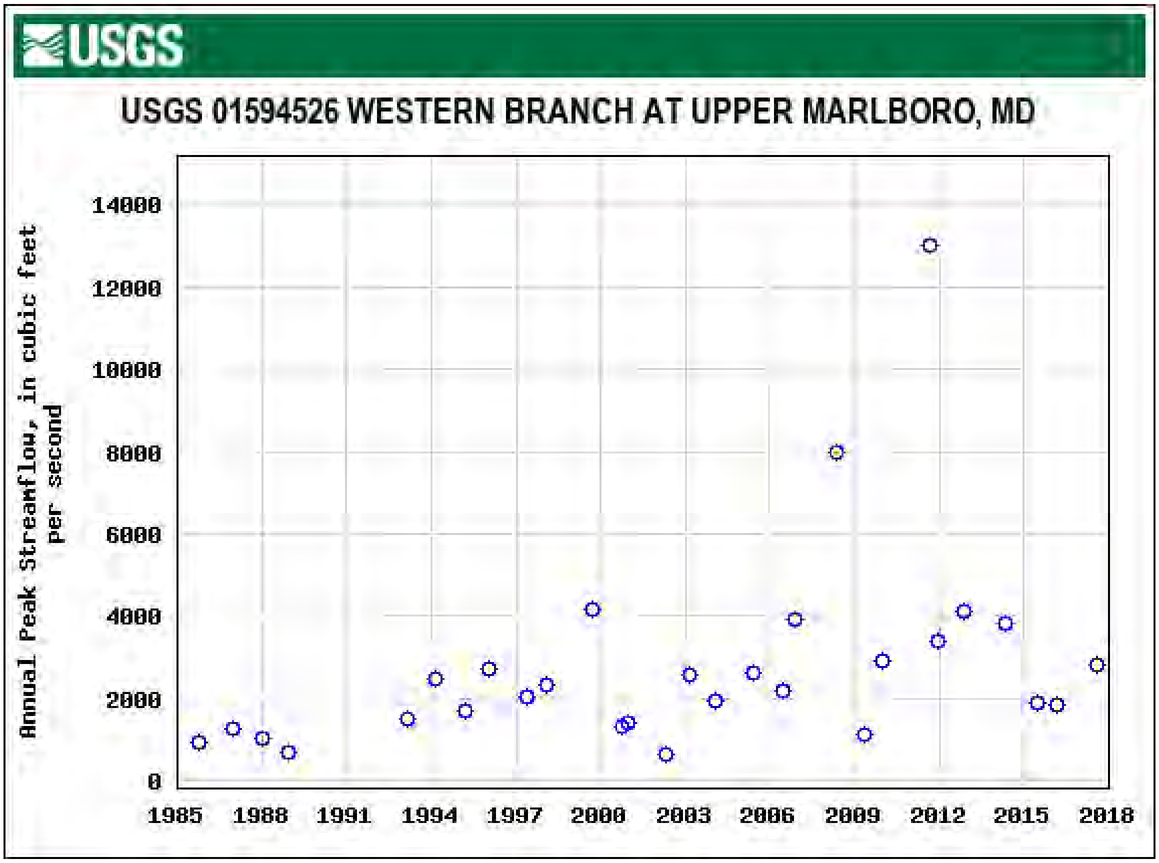
41 years of record 1972 (Tropical Storm Agnes) 1978-2017; drainage area = 350.21 square miles

Kendall's Tau = 0.241

P value = 0.029

Conclusion: Significant upward trend in annual peak flows since the P value is less than 0.05 (five percent level of significance).

Comments: This is an urban watershed with IA85 = 8.6%, IA90 = 10.7%, IA97 = 12.9%, IA02 = 14.9% and IA10 = 17.6%. Upward trend partly related to increased urbanization and four large floods from 2006 to 2014. Used the time-varying mean for the frequency analysis and used IA02 = 14.9% in the regression analysis.



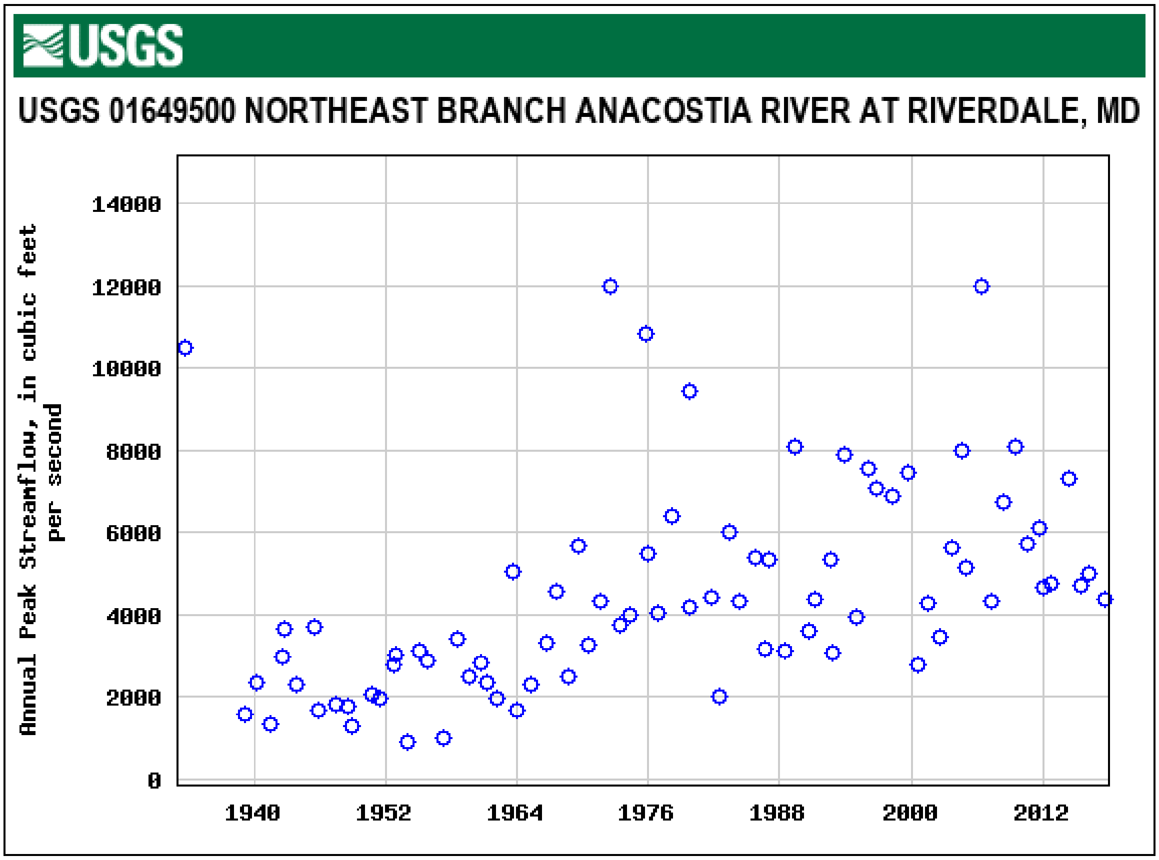
29 years of record from 1986-2017; drainage area = 89.38 square miles

Kendall's Tau = 0.384

P value = 0.004

Conclusion: Significant upward trend in annual peak flows since the P value is less than 0.05 (five percent level of significance).

Comments: This is an urban watershed with IA85 = 9.5%, IA90 = 11.8%, IA97 = 17.5%, IA02 = 21.4%, IA10 = 24.6% (ultimate development = 35.9 percent). Upward trend partly related to increased urbanization but mostly to big floods in 2011 (13,000 cfs) and 2008 (7,980 cfs). The time-varying mean approach was chosen for frequency analysis. Used IA02 = 21.4% in regression analysis.



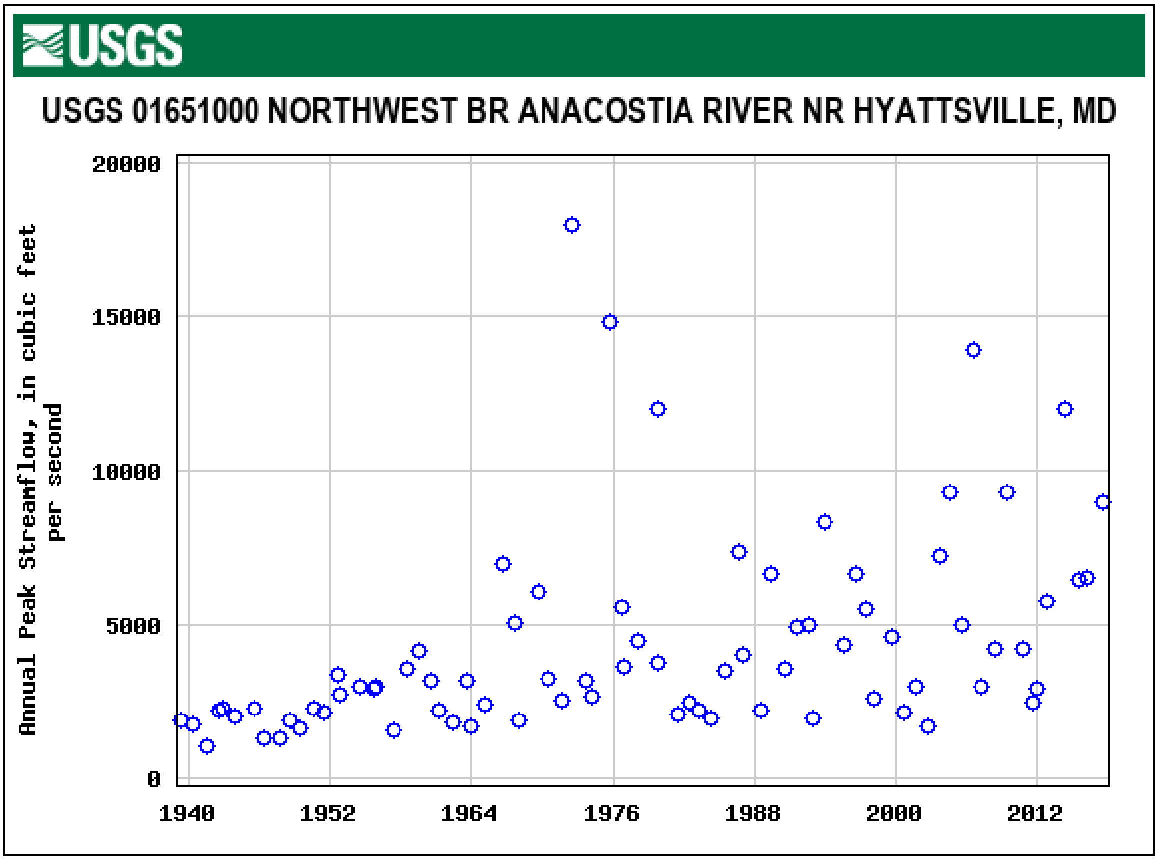
80 years of record 1933, 1939 – 2017; drainage area = 73.2 square miles

Kendall's Tau = 0.055

P value = 0.596

Conclusion: No significant upward trend in annual peak flows for 1972 to 2017 since the P value is greater than 0.05 (five percent level of significance).

Comments: This is an urban watershed with IA85 = 18.9%, IA90 = 21.4%, IA97 = 24.8%, IA02 = 27.4% and IA10 = 28.4% (ultimate development = 34.9 percent). Upward trend for the full period of record related to increased urbanization. A more homogeneous period (1972-2017) was analyzed as well. The time-varying mean approach was chosen for frequency analysis rather than the more homogeneous period. Used IA97 = 24.8% in regression analysis.



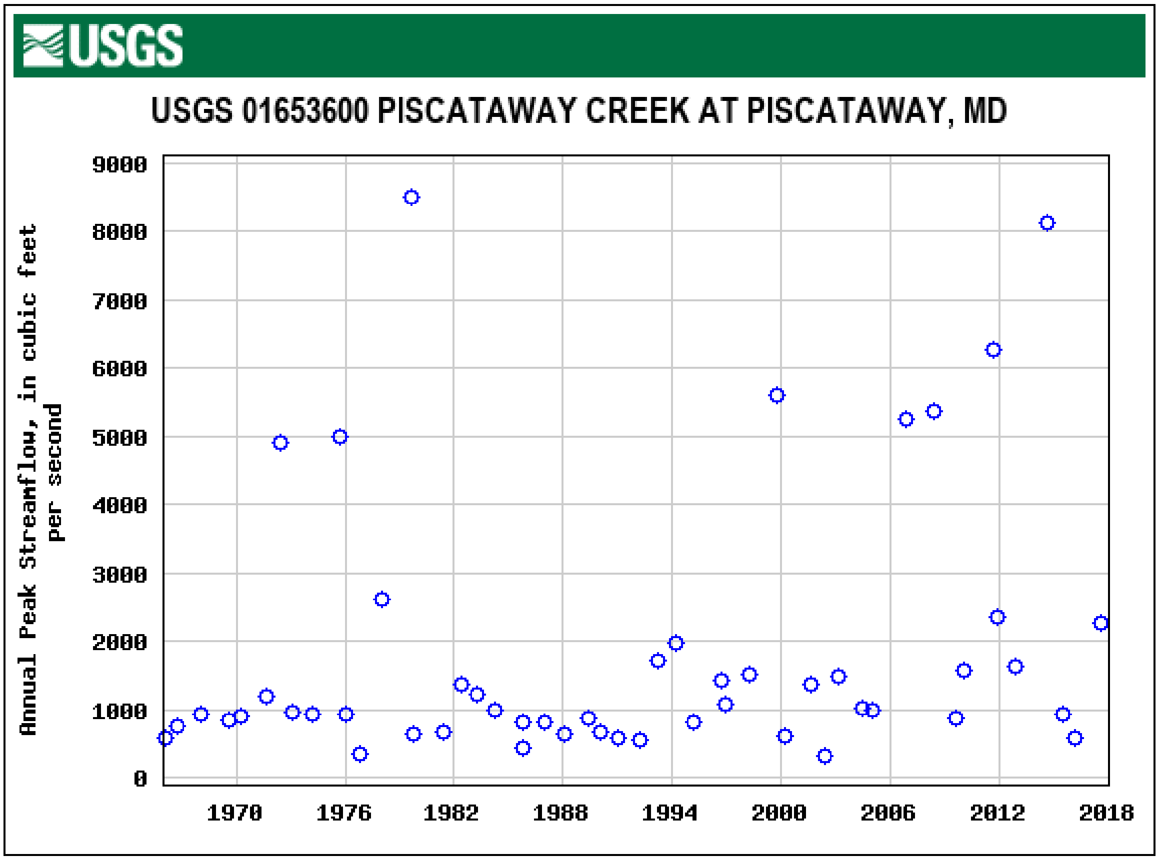
79 years of record 1939– 2017; drainage area = 49.3 square miles

Kendall's Tau = 0.106

P value = 0.302

Conclusion: No significant upward trend in annual peak flows for 1972 to 2017 since the P value is greater than 0.05 (five percent level of significance).

Comments: This is an urban watershed with IA85 = 22.3%, IA90 = 25.1%, IA97 = 27.8%, IA02 = 28.4% and IA10 = 30.3%. Upward trend for the full length of record related to increased urbanization. The more homogeneous period (1972-2017) was chosen for frequency analysis. Used IA97 = 27.8% in regression analysis.



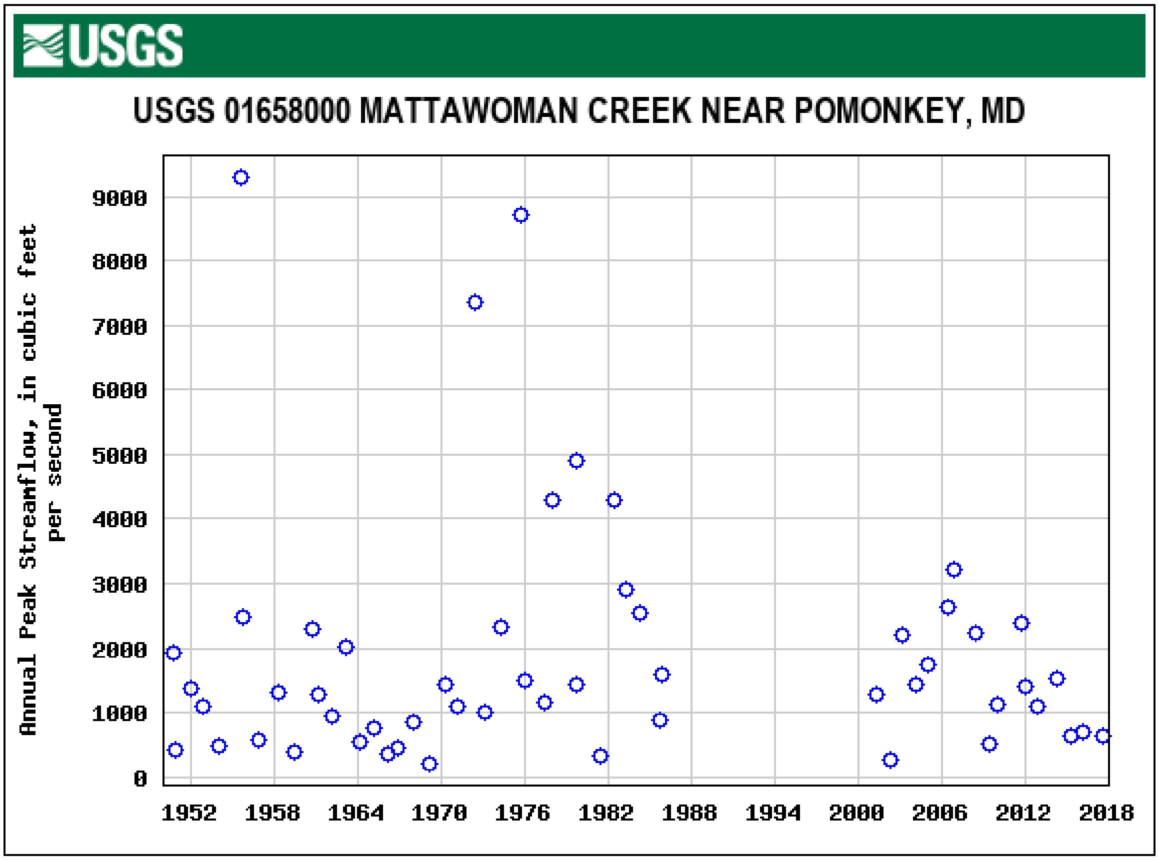
51 years of record – 1966 to 2017; drainage area = 39.4 square miles

Kendall's Tau = 0.196

P value = 0.043

Conclusion: Significant upward trend in annual peak flows since the P value is slightly less than 0.05 (five percent level of significance).

Comments: This is a watershed where the impervious area went from IA85 = 7.7%, IA90 = 9.9%, IA97 = 11.6%, IA02 = 14.3%, IA10 = 17.0%. Five large floods from 1999 to 2014 and the increased urbanization contribute to the upward trend. Used IA97 = 11.6% in the regression analysis.



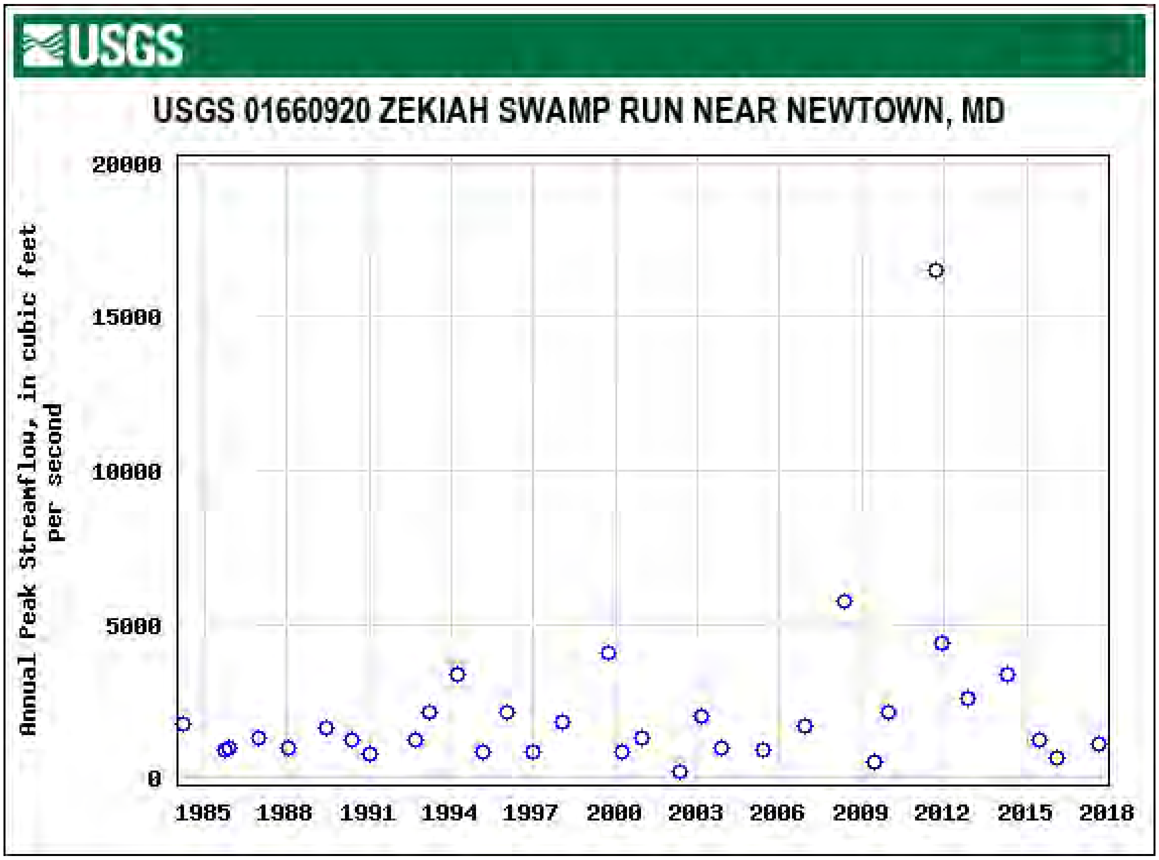
54 years of record – 1950 to 2017; drainage area = 55.6 square miles

Kendall's Tau = 0.052

P value = 0.586

Conclusion: No significant upward trend in annual peak flows since the P value is greater than 0.05 (five percent level of significance).

Comments: No upward trend since the major floods occurred early in the record and no significant increase in urbanization. Large floods prior to 1975 occurred when watershed was mostly rural. IA85 = 5% was used in the regression analysis. The most recent impervious area IA10 = 15.3%.



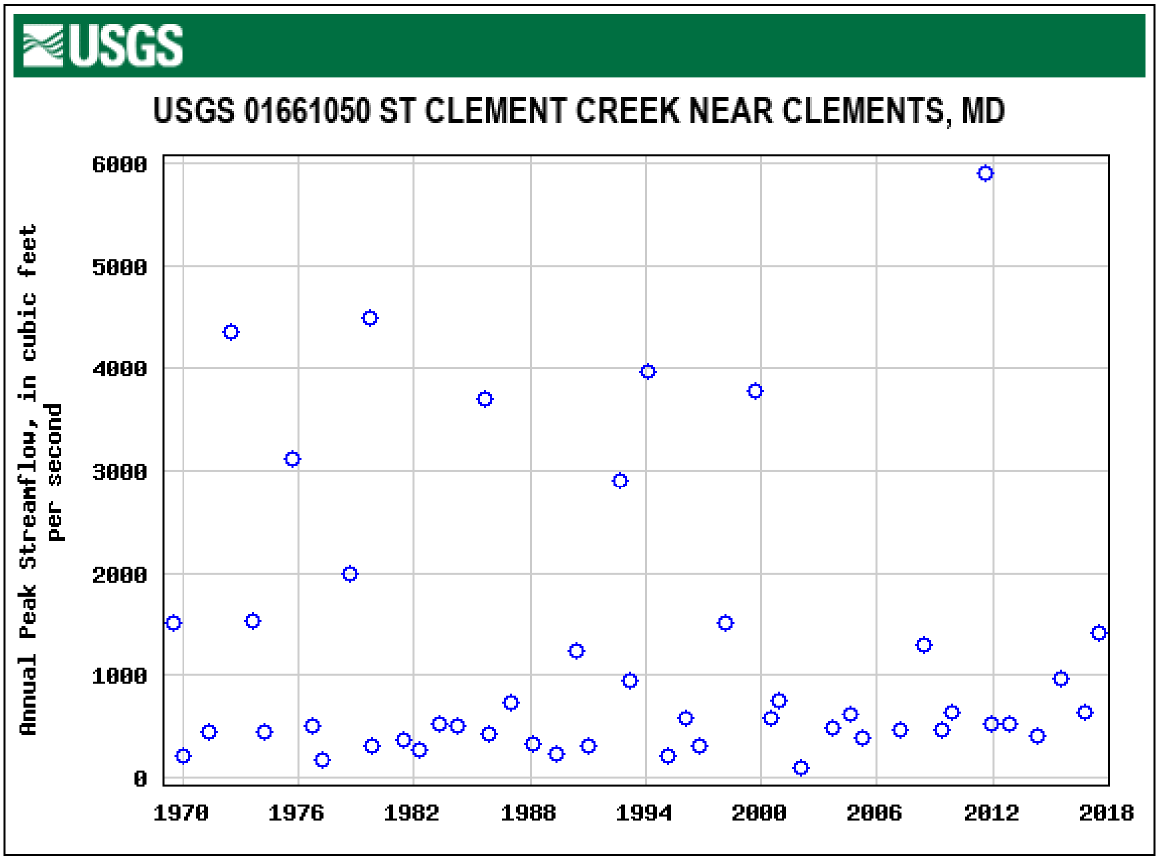
33 years of record – 1984-2017; drainage area = 81.6 square miles

Kendall's tau = 0.119

P value = 0.337

Conclusion: No significant upward trend in annual peak flows since the P value is greater than 0.05 (five percent level of significance).

Comments: No upward trend even though the flood of record (16,500 cfs) occurred in 2011 near the end of the record. IA85 = 4.0%, IA90 = 5.3%, IA97 = 6.7% IA02 = 7.2% and IA10 = 9.2%. No significant increase in urbanization. IA02 = 7.2% was used in the regression analysis.



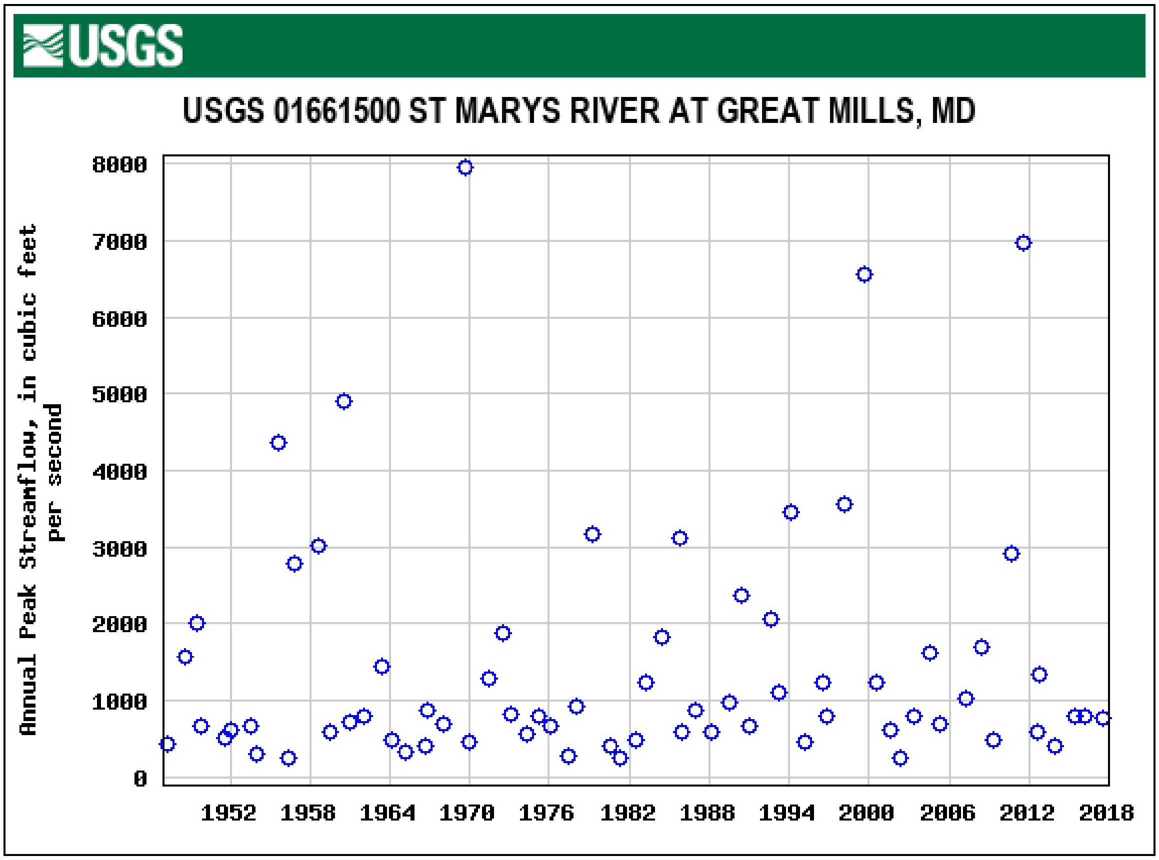
48 years of record – 1969 to 2017; drainage area = 18.2 square miles

Kendall's Tau = 0.045

P value = 0.657

Conclusion: No significant upward trend in annual peak flows since the P value is greater than 0.05 (five percent level of significance).

Comments: No upward trend since the major floods occurred throughout the record and no significant increase in urbanization. IA97 = 3.4% was used in the regression analysis with IA10 = 6.0%.



70 years of record – 1947 to 2017; drainage area = 25.3 square miles

Kendall's Tau = 0.084

P value = 0.306

Conclusion: No significant upward trend in annual peak flows since the P value is less than 0.05 (five percent level of significance).

Comments: No upward trend since the major floods occurred throughout the record and no significant increase in urbanization. IA90 = 6.1% was used in the regression analysis with IA85 = 4.0 % and IA10 = 14.9%.

Attachment WCP-2. Brief description of the time-varying mean approach for frequency analysis.

The time-varying mean approach is described and illustrated using data for Western Branch at Upper Marlboro, Maryland (station 01594526). The annual peak flows from 1986 to 2017 are plotted in Figure A3-22.

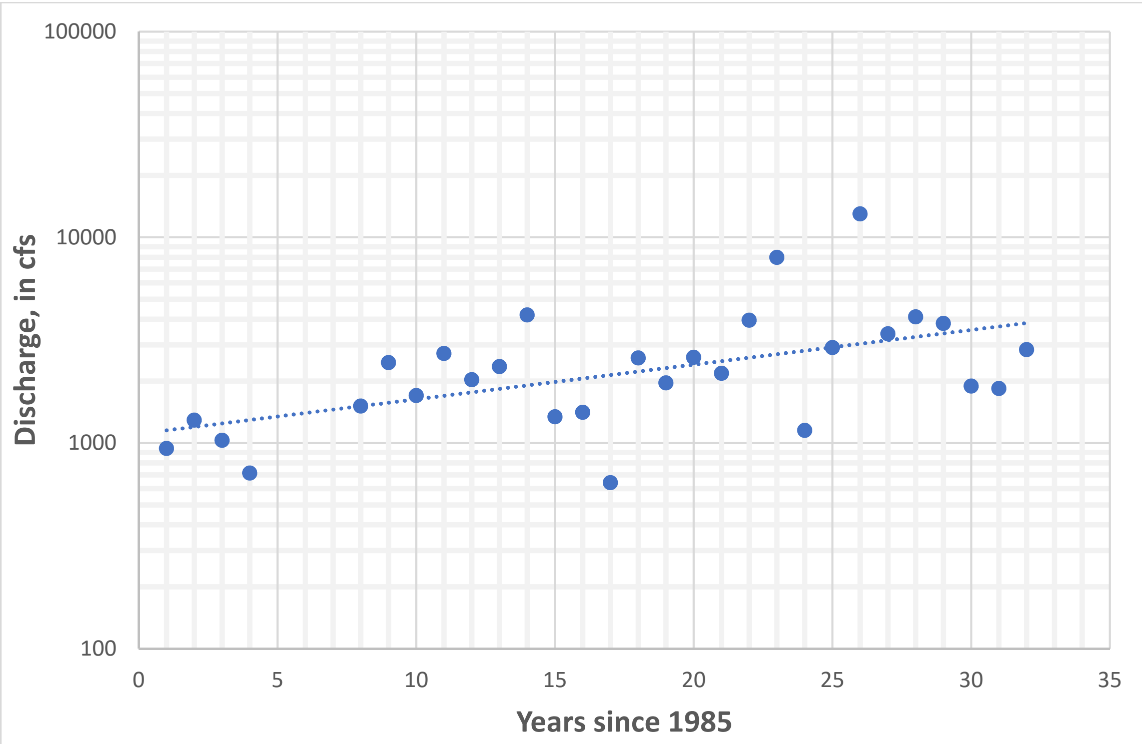


Figure A3-22: Relation between annual peak flows and years since 1985

An equation for the trend line in Figure A3-22 is:

$$\log_{10}(Q) = 3.04502 + 0.01682 * t \qquad (A3\text{-}24)$$

where Q is the annual peak flow in cubic feet per second (cfs) and t is the time in years since 1985. The coefficient for t indicates the annual peak flows are increasing 1.68 percent a year. The trend in the peak flows is statistically significant (based on a Kendall's Tau value of 0.384). The upward trend indicates the annual time series is not stationary and independent which violates an assumption of conventional flood frequency analysis.

The time-varying mean approach utilizes the trend line (Equation A3-24) in Figure A3-22 and provides an estimate of the flood discharges that accounts for changing land use. The approach for the time-varying mean is described by Kilgore and others (2019).

Equation A3-24 can be rewritten using the mean of $\log_{10}(Q)$ ($\overline{LQ}$) and the mean of t ($\bar{t}$) as follows:

$$\log_{10}(Q) = \overline{LQ} + 0.01682\ (t - \bar{t}) \qquad \text{(A3-25)}$$

where t ranges from 1 to 60 with $\bar{t} = \frac{(n+1)}{2}$ and n is the years of record. Equation A3-25 can be rewritten as:

$$\log_{10}(Q) = \overline{LQ} + 0.01682\ \left(t - \frac{(n+1)}{2}\right) \qquad \text{(A3-26)}$$

The equation for estimating the x-percent chance flood discharge ($\log_{10}(Q_x)$) assuming the logarithms are Pearson Type III distributed is:

$$\log_{10}(Q_x) = \overline{LQ} + 0.01682\ \left(t - \frac{(n+1)}{2}\right) + K_x\, S \qquad \text{(A3-27)}$$

where:

- $\overline{LQ}$ = mean of the logarithms = 3.340877 log units,
- S is the standard deviation of the logarithmic residuals about Equation A3-24 = 0.24189 log units.
- K_x is the Pearson Type III frequency factor that is a function of the percent chance exceedance (x) and skew, and
- skew = 0.508 for the logarithms of the annual peak flows.

The Bulletin 17C analysis using station skew and the results of the time-varying mean approach are compared in Table A3-1 for Western Branch at Upper Marlboro (station 01594526) for selected recurrence intervals. The increases in flood discharges using the time-varying mean ranges from 78 percent for the 2-year flood to 37 percent for the 100-year flood to 28 percent for the 500-year flood.

Table A3-1: Comparison of Bulletin 17C analysis and time-varying mean analysis for Western Branch at Upper Marlboro, Maryland (station 01594526)

Recurrence Interval (year)	Gaging station analysis	Gaging station analysis
	Bulletin 17C analysis for 1986-2017 (ft^3/s)	Time-varying mean approach (ft^3/s)
2	2,140	3,810
10	5,415	8,350
25	7,930	11,600
50	10,300	14,500
100	13,100	17,900
500	21,900	28,000

Attachment WCP-3. Flood discharges for the 1.25-, 1.5-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year events (in cubic feet per second) for 27 gaging stations in the Western Coastal Plain Region of Maryland.

Station No.	Stream name	DA (mi^2)	Q1.25 (cfs)	Q1.5 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
01585300	Stemmers Run at Rossville	4.54	790	991	1260	2080	2720	3660	4450	5330	6290	7720
01585400	Brien Run at Stemmers Run	1.96	188	237	316	633	984	1680	2450	3530	5030	7930
01589500	Sawmill Branch at Glen Burnie	5.04	69	81	101	184	280	475	703	1030	1510	2480
01589795	SF Jabez Branch at Millersville	0.96	51	78	122	300	486	823	1160	1590	2140	3050
01590000	North River near Annapolis	8.63	92	102	122	231	385	767	1300	2220	3800	7760
01590500	Bacon Ridge Branch at Chesterfield	6.97	112	149	204	396	576	879	1170	1520	1950	2660
01594400	Dorsey Run near Jessup	11.91	326	379	459	750	1030	1530	2040	2690	3520	5000
01594440	Patuxent River near Bowie	350.21	3880	4900	6370	10800	14400	18300	22700	29400	35100	43800
01594445	Mill Branch near Mitchellville	1.25	73	99	137	270	394	598	790	1020	1300	1750
01594500	Western Branch near Largo	30.04	601	724	880	1300	1590	1980	2280	2600	2920	3370
01594526	Western Branch at Upper Marlboro	89.38	2480	3000	3810	6265	8350	11600	14500	17900	21800	28000
01594600	Cocktown Creek near Huntington	3.9	71	99	145	331	534	923	1340	1910	2660	4040
01594670	Hunting Creek near Huntingtown	9.33	149	193	255	450	613	860	1080	1320	1600	2020
01594710	Killpeck Creek at Huntersville	3.46	123	139	159	209	243	287	321	356	392	441
01594800	St. Leonard Creek near St. Leonard	7.23	62	77	98	159	208	282	345	416	496	616
01649500	NE Branch Anacostia River at Riverdale	73.2	5090	6000	7350	10300	12200	14500	16100	17700	19200	21100
01651000	NW Br Anacostia River near Hyattsville	49.33	2760	3460	4450	7570	10200	14250	17800	22000	26800	34200
01653500	Henson Creek at Oxon Hill	17.19	756	952	1220	2010	2630	3520	4270	5090	5990	7310
01653600	Piscataway Creek at Piscataway	39.43	650	840	990	2200	5300	7400	8700	10000	11000	12500
01658000	Mattawoman Creek near Pomonkey	55.57	630	877	1260	2650	3990	6280	8500	11200	14500	20000
01660900	Wolf Den Branch near Cedarville	2.31	70	92	128	258	388	617	847	1140	1510	2160

Station No.	Stream name	DA (mi^2)	Q1.25 (cfs)	Q1.5 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
01660920	Zekiah Swamp Run near Newtown	81.61	782	1040	1440	2880	4310	6820	9320	12500	16500	23300
01660930	Clark Run near Bel Alton	11.27	240	312	430	954	1560	2810	4280	6470	9650	16100
01661000	Chaptico Creek at Chaptico	10.23	195	260	362	763	1190	1980	2830	3950	5440	8160
01661050	St. Clement Creek near Clements	18.18	325	466	697	1650	2700	4700	6840	9720	13500	20500
01661430	Glebe Branch at Valley Lee	0.24	16	20	26	46	64	92	117	148	184	241
01661500	St. Marys River at Great Mills	25.29	481	653	923	1960	3020	4970	6970	9570	12900	18900

Attachment WCP-4. Watershed characteristics for 27 gaging stations in the Western Coastal Plain Region of Maryland.

The Map Number corresponds to the numbering of the stations in Figure A3-14. A soils in the regression equations is based on the October 2021 SSURGO data from the NRCS Soil Survey web site. Impervious area (IA) is defined at the midpoint of the gaging station record with date of the land use identified.

Map No.	Station No.	Stream name	Period of record	Years of record	DA (mi^2)	A soils (%)	IA (%)	Year for IA
1	1585300	Stemmers Run at Rossville	1960-1989	29	4.54	9.5	25.3	1985
2	1585400	Brien Run at Stemmers Run	1957-1987	29	1.96	9.7	36.8	1985
3	1589500	Sawmill Branch at Glen Burnie	1984-2017	34	5.04	67.2	29.7	2002
4	1589795	SF Jabez Branch at Millersville	1990, 1997-2017	22	0.96	15.1	20.0	2010
5	1590000	North River near Annapolis	1932-1974	43	8.63	20.4	2.7	1985
6	1590500	Bacon Ridge Branch at Chesterfield	1944-1990	35	6.97	30.2	1.5	1985
7	1594400	Dorsey Run near Jessup	1949-1968, 2009	20	11.91	10.5	16.7	1985
8	1594440	Patuxent River near Bowie	1972, 1978-2017	41	350.21	16.2	14.9	2002
9	1594445	Mill Branch near Mitchellville	1966-1976	11	1.25	10.7	4.5	1985
10	1594500	Western Branch near Largo	1950-1974	25	30.04	21.9	11.4	1985
11	1594526	Western Branch at Upper Marlboro	1986-1989, 1993-2017	29	89.38	16.1	21.4	2002
12	1594600	Cocktown Creek near Huntington	1958-1976	19	3.9	45.0	8.7	1985
13	1594670	Hunting Creek near Huntingtown	1989-1998	10	9.33	57.1	2.4	1990
14	1594710	Killpeck Creek at Huntersville	1986-1997	12	3.46	16.8	7.8	1990
15	1594800	St. Leonard Creek near St. Leonard	1958-1968, 2001-2003	14	7.23	85.2	0.3	1985
16	1649500	NE Branch Anacostia River at Riverdale	1939-2016	78	73.2	9.3	24.8	1997
17	1651000	NW Branch Anacostia River near Hyattsville	1972-2017	46	49.33	1.7	27.8	1997
18	1653500	Henson Creek at Oxon Hill	1949-1978	30	17.19	11.6	26.5	1985
19	1653600	Piscataway Creek at Piscataway	1966-2017	51	39.43	12.8	11.6	1997
20	1658000	Mattawoman Creek near Pomonkey	1950-1986, 2001-2017	54	55.57	12.0	5.0	1985

Map No.	Station No.	Stream name	Period of record	Years of record	DA (mi^2)	A soils (%)	IA (%)	Year for IA
21	1660900	Wolf Den Branch near Cedarville	1967-1980	14	2.31	15.8	0.0	1985
22	1660920	Zekiah Swamp Run near Newtown	1984-2017	33	81.61	33.3	7.2	2002
23	1660930	Clark Run near Bel Alton	1966-1976	11	11.27	24.9	6.4	1985
24	1661000	Chaptico Creek at Chaptico	1948-1972	25	10.23	5.8	1.9	1985
25	1661050	St. Clement Creek near Clements	1969-2017	48	18.18	6.7	3.4	1997
26	1661430	Glebe Branch at Valley Lee	1968-1978	11	0.24	2.0	2.1	1985
27	1661500	St. Marys River at Great Mills	1947-2017	70	25.29	4.6	6.1	1990

Attachment WCP-5. Watershed characteristics not used in the final regression equations (except impervious area for a given year) for 27 gaging stations in the Western Coastal Plain Region of Maryland.

The given year for the impervious area and forest cover is the midpoint of the gaging station record and given in the last column of the table.

Map No.	Station No.	Channel Slope (ft/mi)	Land Slope (ft/ft)	Basin Relief (ft)	2010 Forest Cover (%)	2010 Impervious Area (%)	Impervious area (%) given year	Forest Cover (%) given year	Given year of data
1	01585300	61.1	0.062	155.84	17.5	37.1	25.3	29.9	1985
2	01585400	35.5	0.035	61.17	18.9	52.1	36.8	21.4	1985
3	01589500	31.3	0.036	75.52	28.8	33.5	29.7	33.5	2002
4	01589795	46.6	0.048	75.49	23.7	20.0	20.0	23.7	2010
5	01590000	29.0	0.101	105.06	55.7	9.3	2.7	54.6	1985
6	01590500	20.0	0.114	103.64	59.7	9.7	1.5	61.2	1985
7	01594400	34.2	0.051	128.94	30.5	40.4	16.7	40.9	1985
8	01594440	10.1	0.064	356.86	35.3	17.6	14.9	36.3	2002
9	01594445	36.9	0.033	44.32	16.2	38.1	4.5	18.1	1985
10	01594500	10.3	0.047	89.38	26.1	27.9	11.4	41.6	1985
11	01594526	7.7	0.055	125.25	33.0	24.6	21.4	38.1	2002
12	01594600	30.0	0.094	80.36	35.4	16.8	8.7	52.7	1985
13	01594670	19.8	0.098	91.17	66.9	8.5	2.4	73.4	1990
14	01594710	48.0	0.080	95.87	47.5	15.8	7.8	60.4	1990
15	01594800	26.7	0.099	100.99	63.7	8.4	0.3	77.8	1985
16	01649500	27.5	0.055	201.41	28.8	28.3	24.8	29.9	1997
17	01651000	21.0	0.065	298.80	17.6	30.3	27.8	19.9	1997
18	01653500	21.3	0.059	163.46	22.5	37.2	26.5	34.2	1985
19	01653600	15.8	0.057	184.24	45.8	17.0	11.6	48.3	1997
20	01658000	9.9	0.034	139.41	51.4	15.3	5.0	67.1	1985
21	01660900	21.9	0.029	48.60	73.8	5.6	0.0	81.8	1985
22	01660920	9.2	0.044	129.92	58.0	9.2	7.2	58.1	2002
23	01660930	19.7	0.049	112.03	55.3	10.3	6.4	59.2	1985
24	01661000	20.2	0.069	123.21	45.3	6.0	1.9	56.3	1985
25	01661050	12.2	0.059	100.28	55.2	6.0	3.4	55.0	1997
26	01661430	56.1	0.032	52.64	29.3	5.4	2.1	42.5	1985
27	01661500	12.2	0.041	78.92	58.5	14.9	6.1	68.0	1990

Attachment WCP-6. Comparison of October 2021 and legacy SSURGO soils data for 27 gaging stations in the Western Coastal Plain Region of Maryland.

Only the A soil values for October 2021 were used in the final regression equations.

Map No.	Station No.	October 2021 SSURGO A Soil (%)	October 2021 SSURGO B Soil (%)	October 2021 SSURGO C Soil (%)	October 2021 SSURGO D Soil (%)	Legacy SSURGO A Soil (%)	Legacy SSURGO B Soil (%)	Legacy SSURGO C Soil (%)	Legacy SSURGO D Soil (%)
1	01585300	9.5	8.8	53.8	27.8	3.9	58.6	23.8	13.7
2	01585400	9.7	9.0	27.5	53.8	4.6	25.3	52.3	17.7
3	01589500	67.2	0.5	17.4	14.8	64.2	6.0	15.0	14.8
4	01589795	15.1	46.2	14.4	24.3	1.8	66.0	28.1	3.4
5	01590000	20.4	31.9	36.7	11.0	0.1	54.0	35.3	10.4
6	01590500	30.2	33.3	25.2	11.1	0.5	63.8	25.1	10.4
7	01594400	10.5	10.9	34.8	43.7	2.0	36.7	12.7	48.5
8	01594440	16.2	41.7	23.8	17.3	3.0	59.7	16.4	19.6
9	01594445	10.7	14.0	60.1	14.2	0.0	70.7	10.2	18.0
10	01594500	21.9	30.5	13.2	33.7	0.5	53.0	30.7	15.1
11	01594526	16.1	31.2	30.3	21.9	1.0	62.4	20.3	15.9
12	01594600	45.0	10.2	1.3	43.5	0.3	79.8	8.1	11.7
13	01594670	57.1	4.6	3.4	34.9	0.9	81.5	6.4	11.1
14	01594710	16.8	51.4	24.5	7.2	56.1	18.5	16.8	8.6
15	01594800	85.2	0.2	8.3	6.2	6.4	83.9	0.8	8.9
16	01649500	9.3	26.5	24.7	39.0	3.1	31.0	45.0	20.6
17	01651000	1.7	68.0	10.3	19.8	0.1	73.9	10.0	15.8
18	01653500	11.6	12.4	56.0	19.8	0.7	49.3	31.0	18.9
19	01653600	12.8	10.6	60.9	15.5	2.3	55.7	28.3	13.6
20	01658000	12.0	2.4	51.5	33.8	2.6	24.0	51.1	21.9
21	01660900	15.8	3.7	57.3	22.8	0.0	23.5	64.6	11.2
22	01660920	33.3	2.0	44.0	20.6	16.6	28.8	35.2	19.2
23	01660930	24.9	0.8	63.7	10.5	2.3	28.6	52.0	16.9
24	01661000	5.8	27.9	55.6	10.7	20.0	41.8	26.6	11.6
25	01661050	6.7	31.2	48.6	13.5	15.4	41.0	30.0	13.6
26	01661430	2.0	21.2	72.2	4.7	2.9	66.1	21.1	9.1
27	01661500	4.6	13.0	68.3	12.5	7.9	20.8	57.7	13.4

Attachment WCP-7. Computation of the Equivalent Years of Record for Regression Equations for the Western Coastal Plain Region.
Computational Procedure

The variance (standard error squared (SE^2)) of the x-year flood at a gaging station is estimated as

$$SE_x^2 = (S^2/N) * R_x^2 \qquad \text{(A3-28)}$$

where S is the standard deviation of the logarithms (log units) of the annual peak discharges at the gaging station, N is the actual record length in years and R_x is a function of recurrence interval x and skew (G) at the gaging station. The standard error increases as the recurrence interval increases, given the same record length.

In Equation A3-28, the standard error of the x-year flood at a gaging station is inversely related to record length N and directly related to the variability of annual peak flows represented by S (standard deviation) and G (skew). If the standard error of the x-year flood is interchanged with the standard error of estimate (SE) of the regression equation, then Equation A3-28 can be used to estimate the years of record needed to obtain that standard error of estimate. Rearranging Equation A3-28 and solving for N gives Equation A3-29 below.

The equivalent years of record of the regression estimate is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of the regional regression equation. The equivalent years of record are used to weight the gaging station and regression estimates. The equivalent years of record (N_r) of a regression equation is computed as follows (Hardison, 1971):

$$N_r = (S/SE)^2 * R^2 \qquad \text{(A3-29)}$$

where S is an estimate of the standard deviation of the logarithms of the annual peak discharges at the ungaged site, SE is the standard error of estimate of the regional regression estimates in logarithmic units, and R^2 is a function of recurrence interval and skew and is computed as (Stedinger and others, 1993):

$$R^2 = 1 + G*K_x + 0.5 *(1 + 0.75*G^2) * K_x^2 \qquad \text{(A3-30)}$$

where G is an estimate of the average skew for a given hydrologic region, and K_x is the Pearson Type III frequency factor for the x-year flood and skew G.

Computational Details

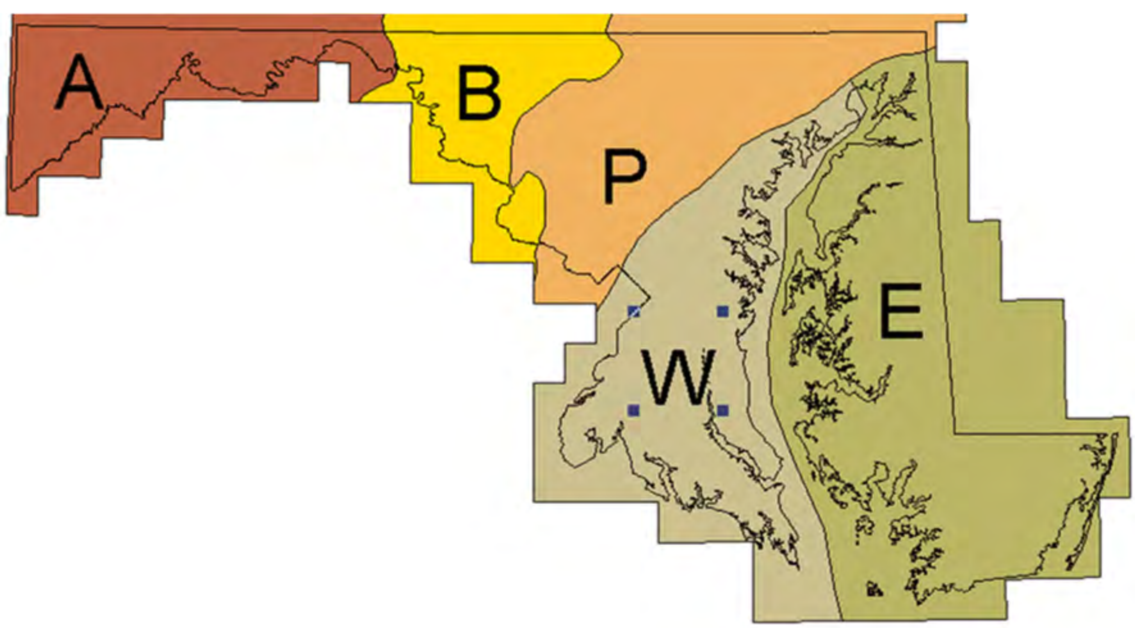
The equivalent years of record are estimated for the regional regression equations and using Equations A3-29 and A3-30 and an estimate of the average standard deviation and average skew for all gaging stations in a given region. For the Western Coastal Plain Region, the average standard deviation (S) is 0.3196 log units and the average skew (G) is 0.541.

Recurrence Interval	K_x value	SE^2 (log units squared)	Equivalent years of record (years)
1.25	-0.856796	0.04325	2.3
1.50			(2.4) Estimated
2	-0.089756	0.03974	2.5
5	0.804686	0.02899	6.4
10	1.325308	0.02149	13
25	1.922003	0.01544	28
50	2.330713	0.01330	43
100	2.714182	0.01426	50
200	3.078453	0.01909	45
500	3.537124	0.03156	34

Regression Equations for Rural and Urban Watersheds in the Piedmont-Blue Ridge and Appalachian Plateau Regions

Previous Investigations

Dillow (1996) and Moglen and others (2006) defined separate sets of regression equations for the Piedmont and Blue Ridge Regions (Figure A3-23). In both analyses, it was assumed that the area of carbonate/limestone rock was confined to the Blue Ridge Region as defined by Dillow (1996). Investigations by the Maryland Hydrology Panel for the September 2010 version (Third Edition) of the Hydrology Panel report determined that the carbonate rock extends eastward into the Piedmont Region. For the September 2010 version of the Hydrology Panel report, rural gaging stations in the Piedmont and Blue Ridge Regions were combined into a single analysis. However, since there are no urban gaging stations (impervious area greater than 10 percent) in the Blue Ridge Region, the urban regression equations documented in the September 2010 version of the Hydrology Panel report were only applicable to the Piedmont Region.



A = Appalachian Plateau and Allegheny Ridge
B = Blue Ridge and Great Valley
P = Piedmont
W = Western Coastal Plain
E = Eastern Coastal Plain

Figure A3-23: Hydrologic regions for Maryland Used by Dillow (1996) and Moglen and others (2006)

For the September 2010 version of the Hydrology Panel report, a new carbonate or limestone rock map was developed that extends into Carroll County in the Piedmont Region. This map is shown in Figure A3-24 and was used for defining the rural regression equations for the combined Piedmont-Blue Ridge Region in the September 2010 Hydrology Panel report.

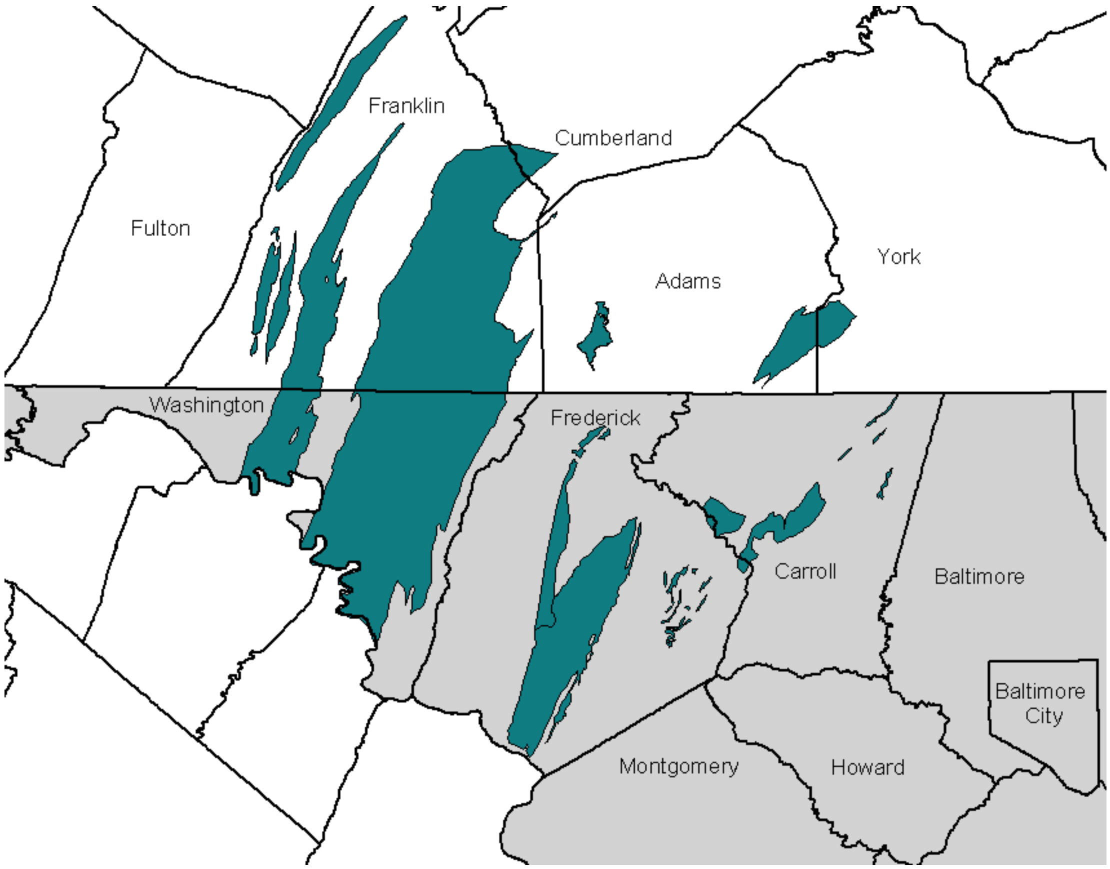


Figure A3-24: Distribution of underlying carbonate/limestone rock in the Piedmont and Blue Ridge Regions of Maryland

The regression equations for the combined Piedmont-Blue Ridge Region and the Appalachian Plateau Region were updated by Thomas and Moglen (2016). Figure A3-24 was used to determine the percentage of carbonate/limestone rock for the Piedmont-Blue Ridge Region. In addition, rural and urban watersheds were combined into a single analysis for the Piedmont-Blue Ridge Region. The Appalachian Plateau Region was shown to have different flood characteristics and remained a separate region. The four hydrologic regions used by Thomas and Moglen (2016) are given in Figure A3-1. In 2020, the regression equations defined by Thomas and Moglen (2016) were revised to give the current regression equations for these regions. These revisions are described in the following sections.

Flood Discharges at the Gaging Stations

Thomas and Moglen (2016) performed frequency analyses using Bulletin 17B for 133 gaging stations, including all current and discontinued stations in the three western regions that have 10 or more years of essentially unregulated annual peak flows through the 2012 water year (Interagency Advisory Committee on Water, 1982). The regression equations for the Appalachian Plateau, Blue Ridge, and Piedmont Regions, documented in the September 2010 version of the Hydrology Panel report, were based on annual peak data through the 1999 water year. Some of the gaging stations have 13 additional years of record through the 2012 water year. The 133 gaging stations used initially in the frequency analysis included the following:

- 55 stations that were discontinued prior to 1999;
- 52 stations with additional data since 1999; and
- 26 new stations with at least 10 years of record.

The locations for the 133 stations in western Maryland are shown in Figure A3-25.

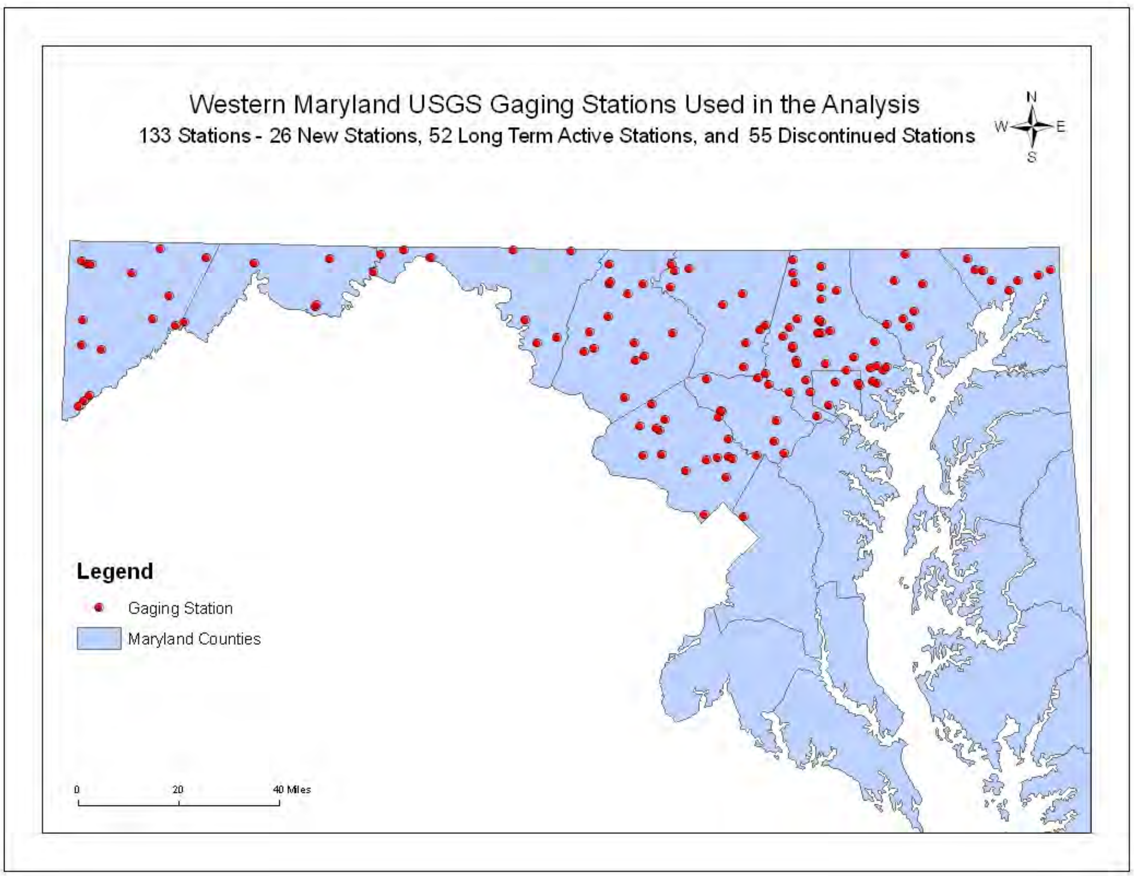


Figure A3-25: Map showing the location of the 133 stations available for updating the regression equations for western Maryland

Regional Skew Analysis

A regional skew analysis was performed by plotting the station skews on a map for 47 **rural** stations (10 percent or less impervious area) with 23 or more years of record. The geographic distribution of the station skews is shown in Figure A3-26. Stations for areas where a significant portion of the watershed was underlain with limestone were omitted from the regional analysis.

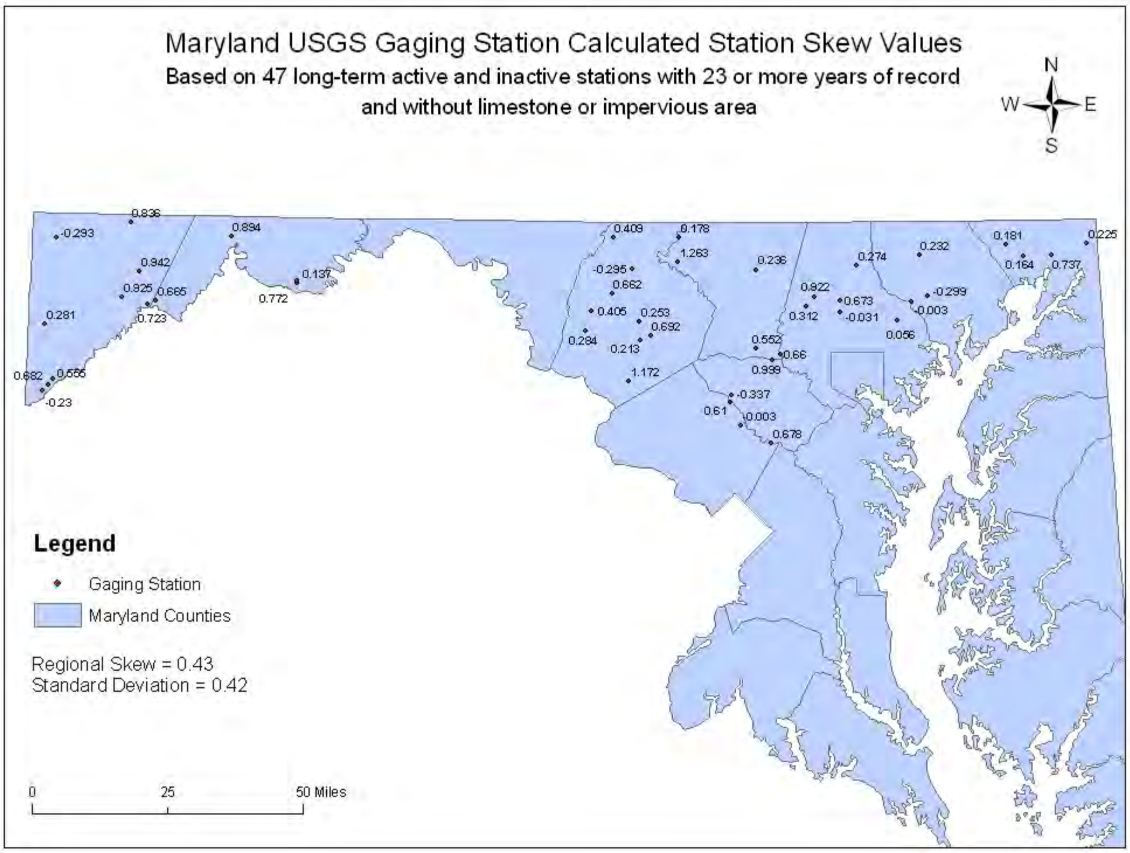


Figure A3-26: Geographic distribution of station skews for 47 long-term stations in Maryland

There is no geographic pattern to the station skews, as shown in Figure A3-26. The average station skew for the 47 stations is 0.43, with a standard deviation (standard error) of 0.42. This contrasts with the regional skew of 0.55 and standard error of 0.45 that were used in the development of the 2006 regression equations for western Maryland.

The station skews were plotted against drainage area, as shown in Figure A3-27, and there was no trend with drainage area. A multiple linear regression analysis for skew indicated that the only statistically significant variables for estimating skew were land slope and the percentage of forest cover. Land slope had an inverse relation with skew (steeper slope, smaller skew) and forest cover had a direct relation (higher forest cover, larger skew). Intuitively, the regression equation did not make sense. Land slope and forest cover are highly correlated, and this correlation may impact the rationality of the regression equation. The average skew of 0.43 with a standard error of 0.42, as defined above, was considered a more defensible approach for defining the regional skew.

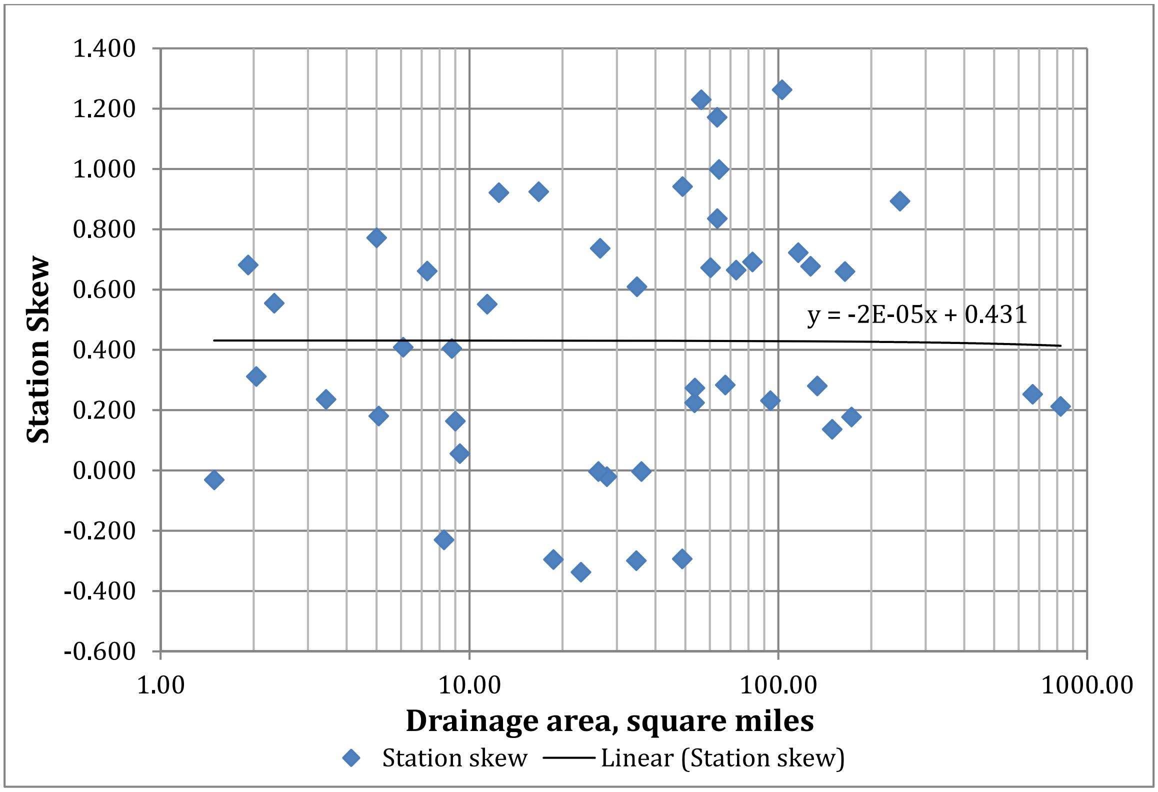


Figure A3-27: Relation between station skew and drainage area for 47 rural stations in Western Maryland

Final Flood Frequency Analysis

The flood frequency curves were rerun using a weighted skew (combination of station and regional skew) for the rural watersheds. The station and regional skew were weighted inversely proportional to the Mean Square Error (standard error squared) using procedures described in Bulletin 17B (Interagency Advisory Committee on Water Data, 1982). Station skew was generally used for the urban watersheds, unless the flood discharges based on the weighted skew were more reasonable based on engineering judgment. The following statistics describe the urban gaging stations with impervious area greater than 10 percent (based on Maryland Department of Planning generalized land use for different time periods):

- 37 stations with impervious area greater than 10 percent;
- 25 stations with impervious area greater than 20 percent;
- 18 stations with impervious area greater than 30 percent;
- 11 stations with impervious area greater than 40 percent; and

- 1 station with impervious area greater than 50 percent (53.5 percent).

For six stations, the log-Pearson Type III distribution did not provide a reasonable fit to the annual peak flows; therefore, the data were plotted on lognormal probability paper and the frequency curves defined by drawing a smooth curve through the plotting positions. These stations were generally short record stations (17 or fewer years of data) or stations where there appeared to be excessive floodplain storage. The six stations are listed below:

- Mingo Branch near Hereford (01581940), 10 years of record;
- North Fork Whitemarsh Run near White Marsh (01585095), 17 years of record;
- Moores Run Tributary near Todd Avenue at Baltimore (01585225), 16 years of record;
- Gwynns Falls at Glyndon (01589180), 14 years of record;
- Cabin Branch near Boyds (01644380), 9 years of record (a few stations used in the analysis had 9 years of record); and
- Bear Creek at Friendsville (03076600), 48 years of record (an S-shaped frequency curve likely related to floodplain storage).

In addition, records were extended at four short-record stations to obtain estimated flood discharges that were more representative of long-record stations. This record extension was accomplished by establishing a graphical relationship between concurrent peak flows at the short- and long-term stations and using the T-year flood discharges at the long-term station to estimate comparable values at the short-term station. The four stations with record extensions and the nearby long-term stations are listed below:

- Great Seneca Creek near Quince Orchard (01644600), drainage area of 53.9 square miles, using the long-term record at Seneca Creek at Dawsonville (01645000), drainage area of 102.2 square miles;
- North Branch Rock Creek near Norbeck (01647720), drainage area of 9.68 square miles, using the long-term record at the Northwest Branch Anacostia River near Coleville (01650500), drainage area of 21.2 square miles;
- Little Youghiogheny River Tributary near Deer Park (03075450), drainage area of 0.55 square miles, using the long-term record at the Youghiogheny River near Oakland (03075500), drainage area of 134 square miles; and
- North Branch Casselman River Tributary at Foxtown (03077700), drainage area of 1.07 square miles, using the long-term record at the Casselman River at Grantsville (03078000), drainage area of 62.5 square miles.

The latter two stations are in the Appalachian Plateau Region, and their annual peak data are from 1965 to 1976. This was a drought period in this region, and the flood discharges based on the short period of record are very low. Even though the drainage area of the long-term station is much larger than that of the short-term station, the flood discharges based on the extended record are considered more accurate than the short-term estimates, due to a reasonable correlation between the annual peak flows for the two stations.

Adjustment of Flood Frequency Curves for Small Rural Gaging Stations

The regression equations developed by Thomas and Moglen (2016) were published in the 2016 Hydrology Panel report and were used for designing bridge sites in Maryland for the last four years. Over this period of time, it was observed that the regression equations tended to give conservatively high flood discharges for small (less than 10 square miles) rural (10 percent or less impervious area) watersheds in the Piedmont-Blue Ridge Region. The reason being that most of the small rural gaging stations had annual peak flow record for the period 1965 to 1977 when a few large floods occurred in this short period of record. The conservatively high flood discharges influenced the regression equations.

Adjustment factors for the flood frequency estimates were developed for small rural gaging stations following procedures in Carpenter (1980). Fifteen rural gaging stations were identified with long records (most stations > 60 years) that included the period 1965-77. Ratios of T-year flood discharges for the long period (beginning of record to 2012) divided by flood discharges for the period 1965-77 were estimated for recurrence intervals from 1.25 to 500 years. The ratios based on the current analysis and those estimated by Carpenter (1980) are given in Table A3-2 for the Piedmont-Blue Ridge Region. Carpenter (1980) ratios are only available for the 2-, 5-, 10-, 25-, 50- and 100-year discharges and were based on data through 1977. The current (2020) ratios are based on annual peak data through 2012.

Table A3-2. Ratio of T-year flood discharges for long period divided by T-year flood discharges for the period 1965-77 based on 15 rural gaging stations in the Piedmont-Blue Ridge Region

Recurrence interval, years	Current (2020) ratio	Ratio from Carpenter (1980)
1.25	0.96	Not Available
1.5	0.94	Not Available
2	0.92	0.84
5	0.83	0.75
10	0.77	0.70
25	0.70	0.65
50	0.65	0.62
100	0.61	0.59
200	0.57	Not Available
500	0.52	Not Available

Thirteen small (less than 10 square miles) rural (10 percent or less impervious area) gaging stations with records confined to the period 1965-77 were used in developing the Piedmont-Blue Ridge regression equations. The adjustment approach is to multiply the T-year flood discharges for the short stations (based on 1965 to 1977) by the ratios in Table

A3-2 before developing the regression equations. These adjusted flood estimates were then used in the regression analysis to get revised 2020 regression equations for the Piedmont-Blue Ridge Region that are documented later.

Flood frequency estimates were adjusted for the following 13 short-term stations:

1. 01490680 – Northeast River Tributary near Charleston, drainage area = 1.75 square miles, impervious area = 1.5 percent, period of record 1967-76,
2. 01578800 – Basin Run at West Nottingham, MD, drainage area = 1.25 square miles, impervious area = 2.5 percent, period of record 1967-76,
3. 01582510 – Piney Creek near Hereford, MD, drainage area = 1.39 square miles, impervious area = 2.4 percent, period of record 1966-79,
4. 01583495 – Western Run Tributary at Western Run, MD, drainage area = 0.23 square miles, impervious area = 0.0 percent, period of record 1967-76,
5. 01587050 – Haymeadow Branch Tributary at Popular Springs, MD, drainage area = 0.49 square miles, impervious area = 10.0 percent, period of record 1966-76,
6. 01637600 – Hollow Road Creek near Middletown, MD, drainage area = 2.32 square miles, impervious area = 1.5 percent, period of record 1965-77,
7. 01640700 – Owens Creek Tributary near Rocky Ridge, MD, drainage area = 1.12 square miles, impervious area = 0.0 percent, period of record 1967-77,
8. 01642400 – Dollyhyde Creek at Libertytown, MD, drainage area = 2.67 square miles, impervious area = 0.1 percent, period of record 1967-76,
9. 01644420 – Bucklodge Branch Tributary near Barnesville, MD, drainage area = 0.28 square miles, impervious area = 0.0 percent, period of record 1967-76,
10. 01647720 – North Branch Rock Creek near Norbeck, MD, drainage area = 9.68 square miles, impervious area = 9.9 percent, period of record 1967-77,
11. 01650050 – NW Branch Anacostia River at Norwood, MD, drainage area = 2.51 square miles, impervious area = 5.1 percent, period of record 1967-76,
12. 01650085 – Nursery Run at Cloverly, MD, drainage area = 0.35 square miles, impervious area = 3.8 percent, period of record 1967-76, and
13. 01650190 – Batchellors Run at Oakdale, MD, drainage area = 0.49 square miles, impervious area = 5.4 percent, period of record 1967-76.

The T-year flood discharges for all gaging stations used in the regression analysis for Western Maryland (WM) are given in Attachment WM-1.

Overview of the Regional Regression Analysis

Data Used in the Regression Analysis

Watershed characteristics were determined for all stations using GISHydro2000 (http://www.gishydro.eng.umd.edu/). The watershed characteristics that were evaluated in the regression analysis included:

- Drainage area, in square miles;
- Channel slope, in feet per mile;
- Land or watershed slope, in feet per foot;
- Percentage of the watershed underlain by limestone;
- Percentage of the watershed with A, B, C, and D soils using the latest SSURGO data; and
- Percentage of the watershed with forest, storage, and impervious area for 1985, 1990, 1997, 2000, 2002, and 2010 land use conditions.

For the 55 gaging stations discontinued before 1999, the land use conditions for 2000, 2002, and 2010 were not determined. With the exception of the percentage of soils, the watershed characteristics documented in the September 2010 version of the Hydrology Panel report were used for these 55 discontinued stations. The percentages of A, B, C, and D soils, based on SSURGO data, were determined for the 55 discontinued stations because the SSURGO data were not available at the time of the previous regression analysis.

The percentage of forest cover and percentage of impervious area used in the regression analysis for the current stations were the values near the middle of the gaging station record to be most representative of the annual peak flows. For the stations discontinued before 1999, the 1985 forest cover and impervious area were used, as was the case for the previous regression analysis.

Initially, regression analyses were performed for all 133 stations in one regional analysis with qualitative variables identifying stations in the three physiographic regions (Appalachian Plateau, Blue Ridge, and Piedmont). The qualitative variable for the Appalachian Plateau was statistically significant, implying that the flood discharges for this region were different from those of the other two regions after accounting for the effects of the watershed characteristics. The qualitative variables for the Blue Ridge and Piedmont Regions were not statistically significant, implying that the flood characteristics for the two regions are similar. This result was consistent with that of previous regression analysis, as the Blue Ridge and Piedmont Regions were combined in the 2010 analysis, and a separate region was defined for the Appalachian Plateau.

Several regression analyses were performed for the Piedmont - Blue Ridge Region and the Appalachian Plateau Region, and 11 stations were identified as outliers. Ten outlier stations were in the Piedmont - Blue Ridge Region, and one station was in the Appalachian Plateau Region. The outlier stations were those where the predicted and observed flood discharges differed by a factor of 2 or more; that is, the predicted values were either more than twice the observed value or less than half of the observed value (criteria based on engineering judgment).

The 11 stations and the reasons they were omitted from the regression analysis are given below:

- Grave Run near Beckleysville (01581830) – drainage area of 7.56 square miles, 13 years of record, impervious area of 5.4 percent – low annual peaks for the drainage area;
- Slade Run near Glyndon (01583000), drainage area of 2.05 square miles, 36 years of record, impervious area of 1.2 percent – low annual peaks for the drainage area;
- Pond Branch at Oregon Ridge (01583570) – drainage area of 0.131 square miles, 13 years of record, impervious area of 0.0 percent – low annual peaks for the drainage area and significant storage in the watershed;
- Beaverdam Run at Cockeysville (01583600) – drainage area of 20.9 square miles, 29 years of record, impervious area of 22.0 percent – low annual peaks for the drainage area;
- Beaver Run near Finksburg (01586210) – drainage area of 14.1 square miles, 30 years of record, impervious area of 11.9 percent – low annual peaks for the drainage area;
- Gwynns Falls Tributary at McDonogh (01589238) – drainage area of 0.027 square miles, 13 years of record, impervious area of 0.0 percent – very small drainage area with one large flood in a short record, and difficult to get reasonable estimates of the flood discharges;
- Patuxent River near Burtonsville (01592000) – drainage area of 127.0 square miles, 32 years of record, impervious area of 3.1 percent – low annual peaks for the drainage area;
- Little Patuxent River at Guilford (01593500) – drainage area of 38.1 square miles, 80 years of record, impervious area of 18.5 percent – low annual peaks for drainage area;
- Marsh Run at Grimes (01617800) – drainage area of 18.3 square miles, 48 years of record, impervious area of 3.4 percent – 100 percent of watershed underlain with limestone and an outlier even with limestone in the regression equation;
- Piney Creek Tributary at Taneytown (01639095) – drainage area of 0.61 square miles, 10 years of record, impervious area of 11.4 percent – low annual peaks for drainage area; and
- Youghiogheny River Tributary near Friendsville (03076505) – drainage area of 0.21 square miles, 12 years of record, impervious area of 0.0 percent – low annual peaks for the drainage area.

The first five outlier stations are located in an area north of Baltimore, and all have a high percentage of A and B soils. However, the sum of A and B soils was not statistically significant in the regression analysis. The close proximity of these stations suggests there may be a common factor as to why the annual peaks are low. Further research beyond this project is warranted to determine what variables may be causing the low annual peak flows for these stations north of Baltimore.

In addition, two stations were combined with nearby stations due to the small differences in drainage area. The annual peak flows for the short record stations were adjusted using a drainage area ratio and combined with data for the stations with the longer record.

The following stations were combined with upstream or downstream stations:

- Patapsco River at Woodstock (01588500), with a drainage area of 251 square miles, was combined with the downstream station 01589000 at Hollofield, with a drainage area of 284.7 square miles and used in the regression analysis; and
- Cattail Creek at Roxbury Mills (01591500), with a drainage area of 27.7 square miles, was combined with the upstream station 01591400 near Glenwood, with a drainage area of 22.9 square miles and used in the regression analysis.

Station 01589000 at Hollofield had a combined record length of 23 years of unregulated annual peak flows, including three historical peak flows. Station 01591400 near Glenwood had a combined record length of 46 years. Therefore, a total of 120 stations were used in the regression analysis, 96 stations in the Blue Ridge and Piedmont Regions, and 24 stations in the Appalachian Plateau. The watershed characteristics used in the regression analysis for the Western Maryland (WM) regions are given in Attachment WM-2 for the Piedmont-Blue Ridge Region and in Attachment WM-3 for the Appalachian Plateau Region.

Development of Regression Equations in the Piedmont-Blue Ridge Region

Regression equations developed by Thomas and Moglen (2016) were based on 96 stations and the most significant watershed characteristics were drainage area (DA) in square miles, percentage of limestone (LIME), percentage of impervious area (IA) and percentage of forest cover (FOR). For the revised regression equations, forest cover was less significant due to high correlation with impervious area and was omitted from the final equations. All variables were converted to logarithms, and a multiple linear regression analysis was performed using the Statistical Analysis System (SAS) package. Regression analyses were also performed without converting LIME, IA, and FOR to logarithms, and the regression equations had essentially equal accuracy to the logarithmic transformed analysis. The exponents in the regression equations varied more logically by recurrence interval with the logarithmic transformation, and those results were used. The equations for the 1.25- to 500-year flood discharges were then converted to exponential form for easier use. They are presented below with the associated standard error of estimate (percent) and the equivalent years of record:

Equation	Standard Error (%)	Eq. years	
$Q_{1\_25} = 63.0\ DA^{0.685}\ (LIME+1)^{-0.090}\ (IA+1)^{0.284}$	53.1	2.0	(A3-31)
$Q_{1\_50} = 89.8\ DA^{0.669}\ (LIME+1)^{-0.100}\ (IA+1)^{0.253}$	48.3	2.4	(A3-32)
$Q_{2} = 131.7\ DA^{0.653}\ (LIME+1)^{-0.112}\ (IA+1)^{0.225}$	43.6	2.8	(A3-33)
$Q_{5} = 283.7\ DA^{0.625}\ (LIME+1)^{-0.136}\ (IA+1)^{0.184}$	35.2	8.3	(A3-34)
$Q_{10} = 434.7\ DA^{0.610}\ (LIME+1)^{-0.148}\ (IA+1)^{0.166}$	31.6	14	(A3-35)
$Q_{25} = 683.3\ DA^{0.599}\ (LIME+1)^{-0.164}\ (IA+1)^{0.153}$	30.0	24	(A3-36)
$Q_{50} = 929.3\ DA^{0.591}\ (LIME+1)^{-0.174}\ (IA+1)^{0.145}$	30.8	29	(A3-37)
$Q_{100} = 1{,}240.1\ DA^{0.584}\ (LIME+1)^{-0.184}\ (IA+1)^{0.139}$	33.0	32	(A3-38)
$Q_{200} = 1{,}616.8\ DA^{0.578}\ (LIME+1)^{-0.193}\ (IA+1)^{0.134}$	36.6	31	(A3-39)
$Q_{500} = 2{,}252.2\ DA^{0.571}\ (LIME+1)^{-0.205}\ (IA+1)^{0.129}$	42.9	29	(A3-40)

The standard error of estimate, expressed in percent, is the standard deviation of the residuals about the regression equation. It is a measure of the agreement between the regression estimates and the gaging station data used in the analysis. The equivalent years of record are defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of estimate for the regression equations. Equivalent years of record are used to weight the regression estimate with the gaging station estimate, as described in Chapter 2 of this report. The computation of the equivalent years of record is described in Attachment WM-4.

All explanatory variables are significant at the 5-percent level of significance with the exception that limestone is statistically significant at the 10-percent level for the 1.25-year flood. The 5-percent level of significance, typically used for including explanatory variables in the regression equations, means there is less than a 5-percent chance of erroneously including a variable in the regression equation.

Rationale for Regression Equations in the Piedmont-Blue Ridge Region

For Equations A3-31 to A3-40, the drainage area exponent decreases with an increasing recurrence interval, consistent with earlier results. A possible explanation is that the storm rainfall for the more intense storms varies considerably across a watershed and does not have a uniform impact across the entire watershed (that is, the effective drainage area is less). The limestone exponent is an increasing negative value (inverse relation) with the recurrence interval, implying that the percentage of limestone becomes more important for the larger floods. A likely reason is that the increased rainfall depth in the larger floods leads to more abstraction in the karst watersheds and results in relatively lower runoff volumes. The exponents on impervious area decrease with the recurrence interval, implying that impervious area has less influence as the floods become larger. This is a well-known result in which soils become more saturated for the larger floods, and impervious area has relatively less impact on runoff volumes.

The higher standard errors for the shorter recurrence interval (1.25- to 2-year) floods imply that explanatory variables other than drainage area and the percentage of limestone, and impervious area influence these floods. The time-sampling error (error in T-year flood discharge) is actually less for these smaller floods, so one would expect a lower standard error in the regression analysis. Instead, the standard errors of the regression equations for the smaller events are influenced by the model error, indicating that other important explanatory variables may be missing from the equations.

As noted above and shown in Figure A3-28 the correlation between the logarithms of the percentage of forest cover (lfor) and the logarithm of the percentage of impervious area (lia) is -0.51. This correlation value is statistically different from zero, as indicated by the small p-level of < 0.0001. The relatively high correlation between impervious area and forest cover is one reason why forest cover was not statistically significant and included as an explanatory variable.

Figure A3-28 indicates several other high correlations between explanatory variables, which explain why other variables, such as channel slope and land slope, were not included in Equations A3-31 to A3-40. For example, the following significant correlations are highlighted in Figure A3-28:

- Channel slope (lchansl) is inversely correlated with drainage area (lda) (correlation = -0.84) because small watersheds have large channel slopes and vice versa;
- Land slope (lslope) is inversely correlated with impervious area (lia) (correlation = -0.61), implying that steep land slopes are not conducive to development; and
- Land slope (lslope) and forest cover (lfor) are directly correlated (correlation = 0.66), implying that steep land slopes are conducive to forest cover.

Pearson Correlation Coefficients, N = 96
Prob > | r | under H_0: $\rho = 0$

	lda	lia	llime	lfor	lslope	lchansl
lda	1.00000	-0.17224	0.28320	0.33216	0.26567	-0.83946
		0.0899	0.0047	0.0008	0.0082	< 0.0001
lia	-0.17224	1.00000	-0.20246	-0.51137	-0.60763	-0.02424
	0.0899		0.0456	< 0.0001	< 0.0001	0.8127
llime	0.28320	-0.20246	1.00000	0.07473	0.21665	-0.17498
	0.0047	0.0456		0.4646	0.0321	0.0848
lfor	0.33216	-0.51137	0.07473	1.00000	0.65964	-0.03247
	0.0008	< 0.0001	0.4646		< 0.0001	0.7510
lslope	0.26567	-0.60763	0.21665	0.65964	1.00000	0.13963
	0.0082	< 0.0001	0.0321	< 0.0001		0.1703
lchansl	-0.83946	-0.02424	-0.17498	-0.03247	0.13963	1.00000
	< 0.0001	0.8127	0.0848	0.7510	0.1703	

Figure A3-28: Correlation matrix for selected watershed characteristics for the 96 stations in the Piedmont Piedmont-Blue Ridge Region

The percentages of the watershed in A, B, C, and D soils, based on SSURGO data, were also evaluated as explanatory variables. The percent soils were not statistically significant for the Piedmont-Blue Ridge Region.

Equations A3-31 to A3-40 are applicable to rural and urban watersheds for the following ranges of the explanatory variables:

- Drainage area ranging from 0.111 to 816.4 square miles;
- Percentage of limestone ranging from 0.0 to 81.7 percent; and
- Percentage of impervious area ranging from 0.0 to 53.5 percent.

Figure A3-29 compares the 100-year regression estimates from Equation A3-38 to the gaging station estimates. Note the slope of the line in Figure A3-29 is close to 1.0 and the intercept is close to zero implying the revised equation based on drainage area, impervious area and limestone is relatively unbiased. The scatter about the trend is uniform throughout the range of discharges.

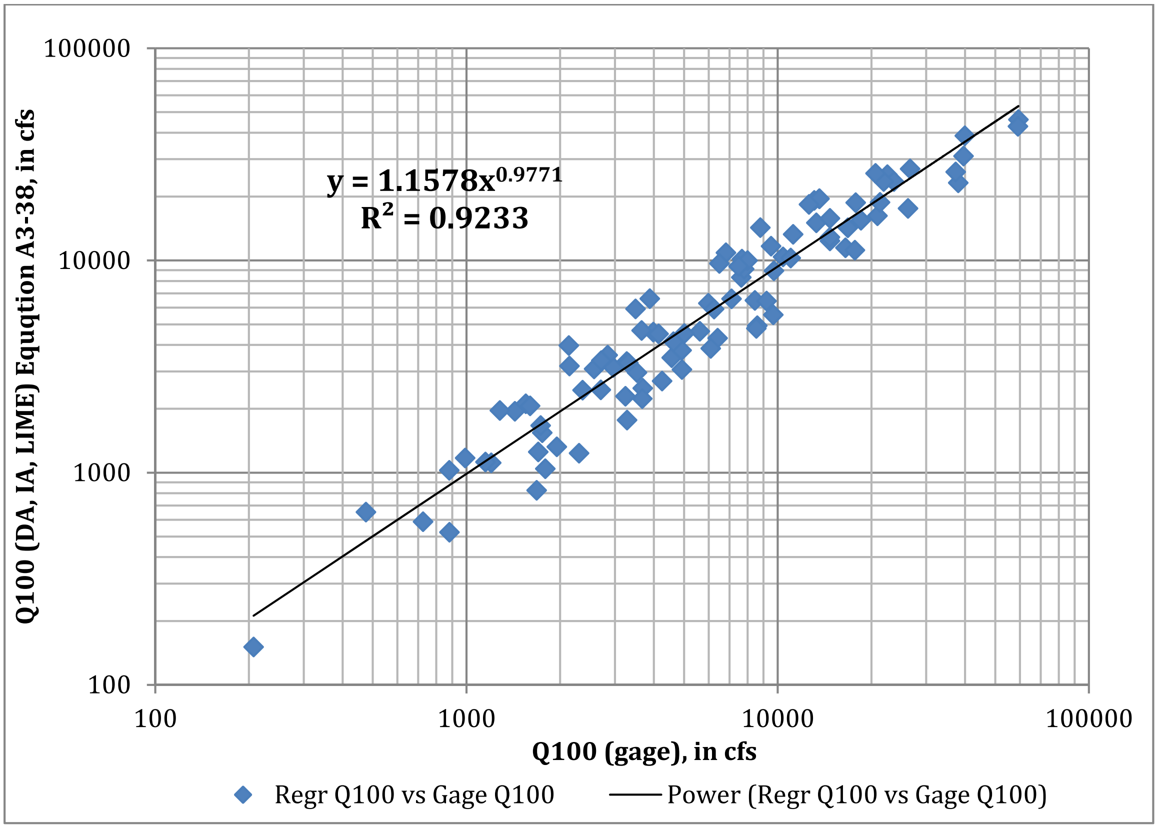


Figure A3-29: Comparison of the estimated 100-year discharges from Equation A3-38 to the gaging station estimates for the Piedmont-Blue Ridge Region

Figure A3-30 compares the 10-year regression estimates from Equation A3-35 to the gaging station estimates. Note the slope of the line in Figure A3-30 is close to 1.0 and the intercept is close to zero implying the revised equation based on drainage area, impervious area and limestone is relatively unbiased. The scatter about the trend is uniform throughout the range of discharges.

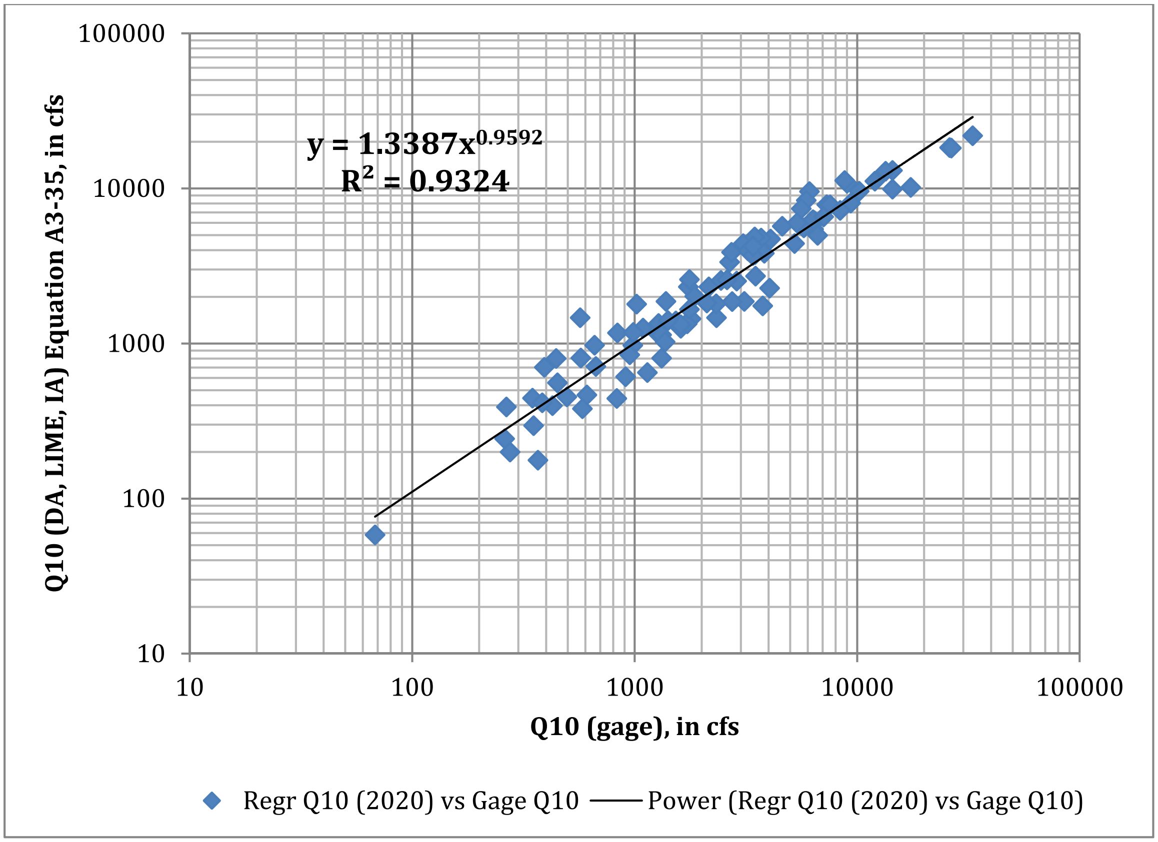


Figure A3-30: Comparison of the estimated 10-year discharges from Equation A3-35 to the gaging station estimates for Piedmont-Blue Ridge Region

<u>Comparison of Revised Piedmont-Blue Ridge Equations to the 2016 Published Equations</u>

The differences in the 100-year discharges for the 2016 published equation and the revised Equation A3-38 based on drainage area, impervious area and limestone are shown in Figure A3-31.

The trend line in Figure A3-31 has a R-squared value of 0.9971 indicating close agreement between the two equations but note that Equation A3-38 is, on average, consistently less than the 2016 equation. For example:

- When the 100-year discharge from the 2016 equation is 500 cfs, Equation A3-38 is predicting, on average, 413.7 cfs, a 17.3 percent reduction.
- When the 100-year discharge from the 2016 equation is 1,000 cfs, Equation A3-38 is predicting, on average, 860.1 cfs, a 14 percent reduction.
- When the 100-year discharge from the 2016 equation is 10,000 cfs, Equation A3-38 is predicting, on average, 9,782.7 cfs, a 2.3 percent reduction.

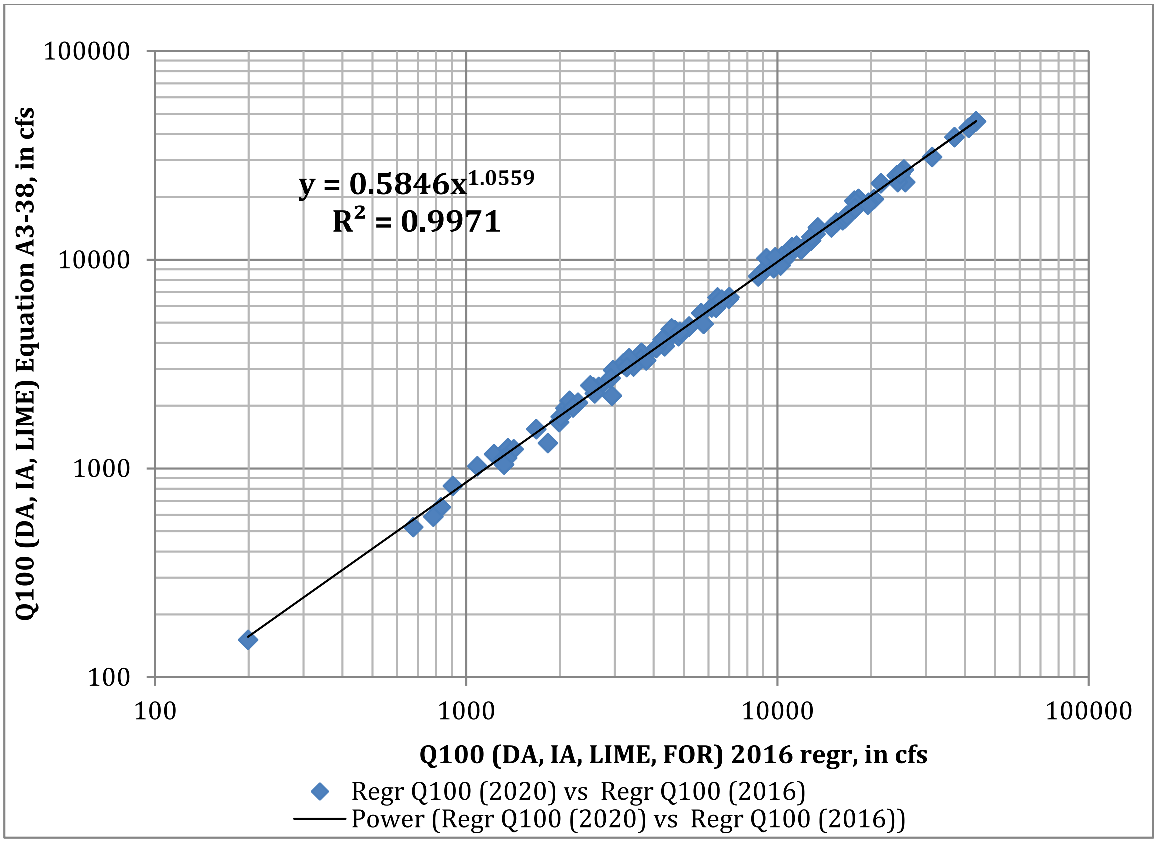


Figure A3-31: Comparison of 100-year flood discharges from Equation A3-38 to estimates from the published 2016 equation for the Piedmont-Blue Ridge Region

A comparison for the 10-year flood is given in Figure A3-32 for Equation A3-35 and the 2016 equation. There is good agreement between the two equations as evidence by an R-squared value of 0.9876. Equation A3-35 gives a slightly lower estimate as it should be because of the adjustment to the flood frequency values for the small rural gaging stations. For example:

- When the 10-year discharge from the 2016 equation is 100 cfs, Equation A3-35 is predicting, on average, 93 cfs, a 7 percent reduction.
- When the 10-year discharge from the 2016 equation is 1,000 cfs, Equation A3-35 is predicting, on average, 966.6 cfs, a 3.3 percent reduction.
- When the 10-year discharge from the 2016 equation is 10,000 cfs, Equation A3-35 is predicting, on average, about the same discharge.

The reduction for the 10-year flood should be less than the 100-year flood because the 10-year discharges for the small rural gaging stations were adjusted by 0.77 as compared to the 100-year discharges value of 0.61 as shown in Table A3-2.

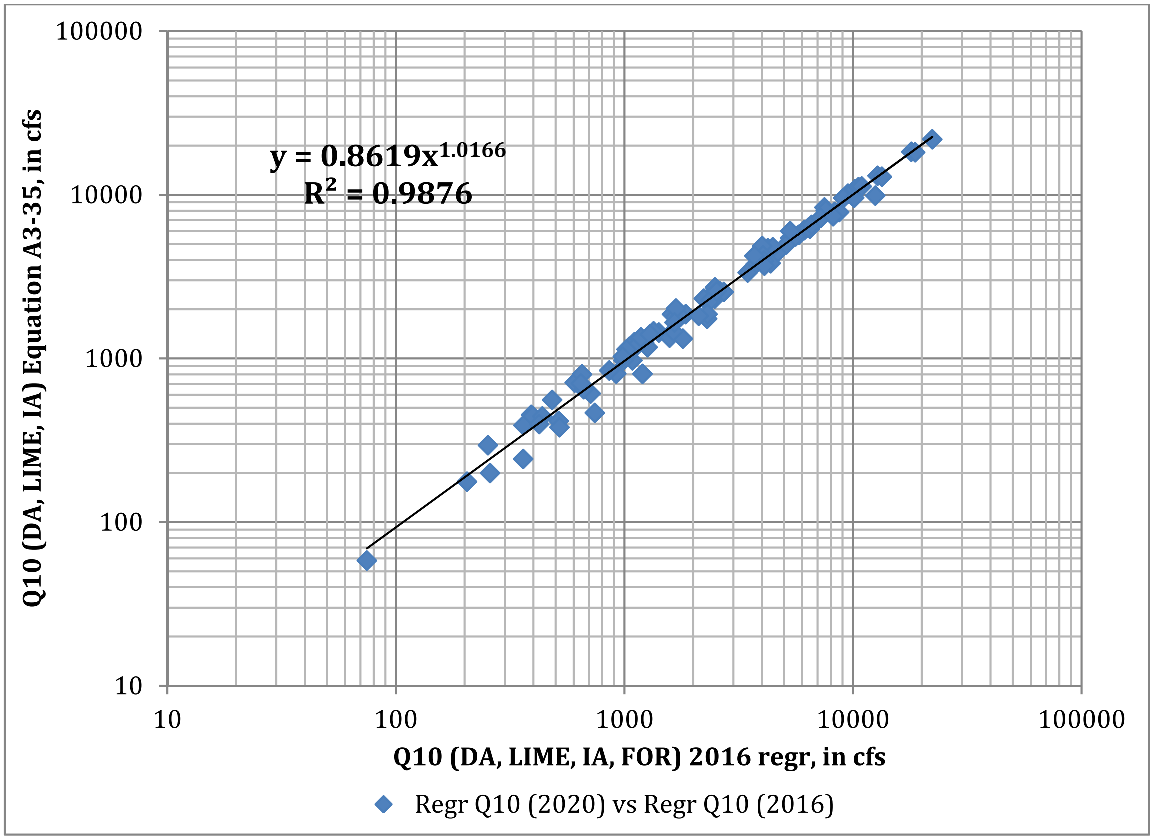


Figure A3-32: Comparison of 10-year flood discharges from Equation A3-35 to estimates from the published 2016 equation for the Piedmont-Blue Ridge Region

The adjustments to the flood frequency estimate for the 13 small rural watersheds have accomplished the intended objective of giving smaller regression equations for the small watersheds. Calibration of the TR-20 for small rural watersheds should be facilitated by use of the revised Equations A3-31 to A3-40.

Development of Regression Equations in the Appalachian Plateau Region

For the Appalachian Plateau, based on 24 stations, the two most significant watershed characteristics are drainage area (DA) in square miles and land (watershed) slope (LSLOPE) in feet per foot. In the Thomas and Moglen (2016) analysis, land slope was based on the legacy DEM data in GISHydro2000 prior to the 2016 analysis. The default DEM data available in GISHydro in 2020 is dated May 2018. To be consistent in application of the equations for the Appalachian Plateau Region, the equations were revised using the May 2018 DEM data to estimate land slope. As discussed earlier, the Youghiogheny River Tributary near Friendsville (03076505) gaging station was deleted from the analysis as an outlier. The annual flood peaks were very low for this 0.21-square-mile watershed and reasonable record extension was not possible.

LSLOPE is only statistically significant at the 5-percent level up to the 5-year flood but was retained in the equations for the larger floods for consistency. As with the Piedmont-Blue Ridge Region analysis, all variables were converted to logarithms, and a multiple linear regression analysis was performed using SAS. The equations for the 1.25- to 500-year flood discharges were then converted to exponential form for easier use and are presented below with the associated standard error and equivalent years of record:

Equation	**Standard error (%)**	**Eq. years**	
$Q_{1\_25} = 79.4\ DA^{0.840}\ LSLOPE^{0.397}$	29.2	1.3	(A3-41)
$Q_{1\_5} = 92.4\ DA^{0.831}\ LSLOPE^{0.348}$	21.8	4.4	(A3-42)
$Q_{2} = 115.2\ DA^{0.825}\ LSLOPE^{0.333}$	19.9	7.5	(A3-43)
$Q_{5} = 183.4\ DA^{0.813}\ LSLOPE^{0.306}$	20.7	11	(A3-44)
$Q_{10} = 221.2\ DA^{0.808}\ LSLOPE^{0.248}$	24.9	12	(A3-45)
$Q_{25} = 317.6\ DA^{0.803}\ LSLOPE^{0.261}$	28.7	13	(A3-46)
$Q_{50} = 397.6\ DA^{0.803}\ LSLOPE^{0.263}$	33.6	13	(A3-47)
$Q_{100} = 474.5\ DA^{0.799}\ LSLOPE^{0.244}$	38.3	12	(A3-48)
$Q_{200} = 559.4\ DA^{0.795}\ LSLOPE^{0.227}$	44.0	11	(A3-49)
$Q_{500} = 664.0\ DA^{0.790}\ LSLOPE^{0.183}$	51.3	10	(A3-50)

The standard error of estimate, expressed in percent, is the standard deviation of the residuals about the regression equation. It is a measure of the agreement between the regression estimates and the gaging station data used in the analysis. The equivalent years of record are defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of estimate for the regression equations. Equivalent years of record are used to weight the regression estimate with the gaging station estimate, as described in Chapter Two of this report. The computation of the equivalent years of record is described in Attachment WM-4.

Regression analyses were also performed by including the Appalachian Plateau stations in an analysis with the Piedmont-Blue Ridge stations and using a qualitative variable to account for differences in the Appalachian Plateau Region (total of 120 stations). The regression equations, based on 120 stations, had a significant bias for under-predicting flood discharges for the larger watersheds in the Appalachian Plateau Region. Therefore, Equations A3-41 to A3-50, based on a separate Appalachian Plateau Region, were considered more reasonable.

Rationale for the Regression Equations in Appalachian Plateau Region

For Equations A3-41 to A3-50, the drainage area exponent decreases with drainage area, the same trend observed for the Piedmont-Blue Ridge Region. For the larger storms, the rainfall intensity tends to vary across the watershed so that all parts of the watershed do not contribute equally to runoff. The drainage area exponents are larger than for the Piedmont-Blue Ridge Region, implying that the storms are more uniform or tend to cover more of the watershed. The Piedmont-Blue Ridge Region is more susceptible to the more intense storms from hurricane events. The land slope exponent decreases with the recurrence interval, implying the slope of the watershed becomes less critical to the runoff process as the flood magnitudes increase.

Channel slope is also significant at the 10-percent level for many recurrence interval floods, being the third most significant variable after drainage area and land slope. However, using land slope rather than channel slope results in lower standard errors for the regression equations. Figure A3-33 shows the correlations between the logarithms of selected watershed characteristics for the 24 stations in the Appalachian Plateau Region. Some significant correlations are as follows:

- Channel slope (lchansl) and drainage area (lda) have a correlation of -0.73;
- Land slope (lslope) and drainage area (lda) have a correlation of 0.58; and
- Forest cover (lfor) and impervious area (lia) have a correlation of -0.58.

Land slope and drainage area have a lower correlation than channel slope and drainage area, so land slope is explaining more variability than channel slope in a regression equation including drainage area. Forest cover and impervious area are not statistically significant, because forest cover does not exhibit much variability at the gaged watersheds in the Appalachian Plateau Region and impervious area has a very limited range (from 0 to 4.2 percent).

Pearson Correlation Coefficients, N = 24
Prob > | r | under H_0: $\rho = 0$

	lda	lia	lfor	lslope	lchansl
lda	1.00000	0.46560	-0.27400	0.58168	-0.72999
		0.0219	0.1951	0.0029	< 0.0001
lia	0.46560	1.00000	-0.57959	0.26817	-0.28976
	0.0219		0.0030	0.2052	0.1696
lfor	-0.27400	-0.57959	1.00000	0.08779	0.36722
	0.1951	0.0030		0.6833	0.0775
lslope	0.58168	0.26817	0.08779	1.00000	-0.15633
	0.0029	0.2052	0.6833		0.4657
lchansl	-0.72999	-0.28976	0.36722	-0.15633	1.00000
	< 0.0001	0.1696	0.0775	0.4657	

Figure A3-33: Correlation matrix for selected watershed characteristics for the 24 stations in the Appalachian Plateau Region

Watershed shape was also evaluated as a possible explanatory variable in the Appalachian Plateau Region. Watershed shape was defined as channel length squared divided by drainage area, essentially a measure of the length of the watershed divided by the width of the watershed. The watershed shape factor was not statistically significant.

The sums of A and B soils and C and D soils were also evaluated. The sum of C and D soils does not vary much across the gaging stations in the Appalachian Plateau Region and was not statistically significant. The sum of A and B soils was statistically significant for recurrence intervals of 10 years and less and reduced the standard error somewhat from the equations using drainage area and land slope. However, for recurrence intervals of 25 years and greater, the sum of A and B soils was not significant, and the standard errors were higher than the equations using drainage area and land slope. The latter variables were judged to be the two best variables for predicting flood discharges in the Appalachian Plateau.

Equations A3-41 to A3-50 are applicable to rural watershed for the following ranges of the explanatory variables:

- Drainage area ranging from 0.52 to 294.14 square miles, and
- Land slope ranging from 0.06400 to 0.25265 ft/ft.

Evaluation of Appalachian Plateau Region Equations

Figure A3-34 compares the 100-year discharges from Equation A3-48 to the gaging station estimates. The fitted trend line is close to the equal discharge line implying the regression estimates are reasonably unbiased.

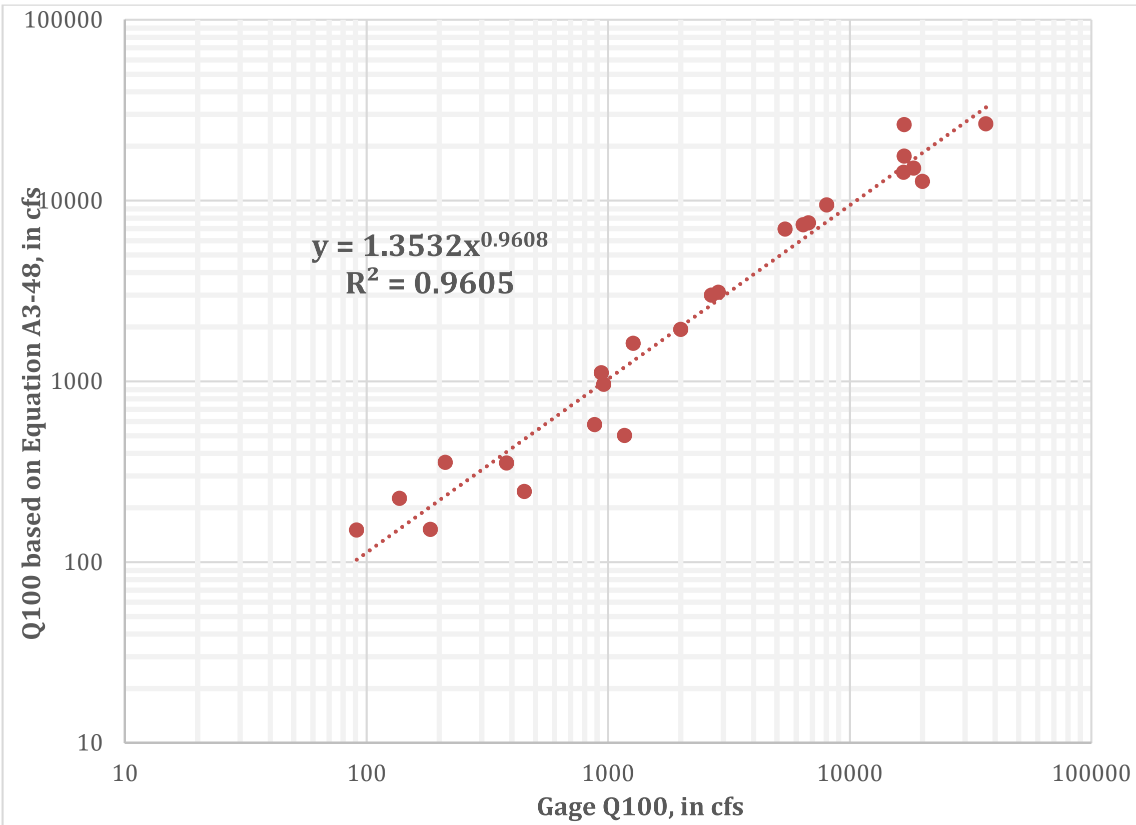


Figure A3-34: Comparison of the estimated 100-year discharges from Equation A3-48 to the gaging station estimates for 24 gaging stations in the Appalachian Plateau Region

Figure A3-35 is a comparison of 100-year discharges from Equation A3-48 based on land slope from the May 2018 DEM data and the published 2016 equations based on land slope from the legacy DEM data. The two estimates of the 100-year discharge are nearly identical as the fitted trend line is close to the equal discharge line (slope and intercept close to 1.0).

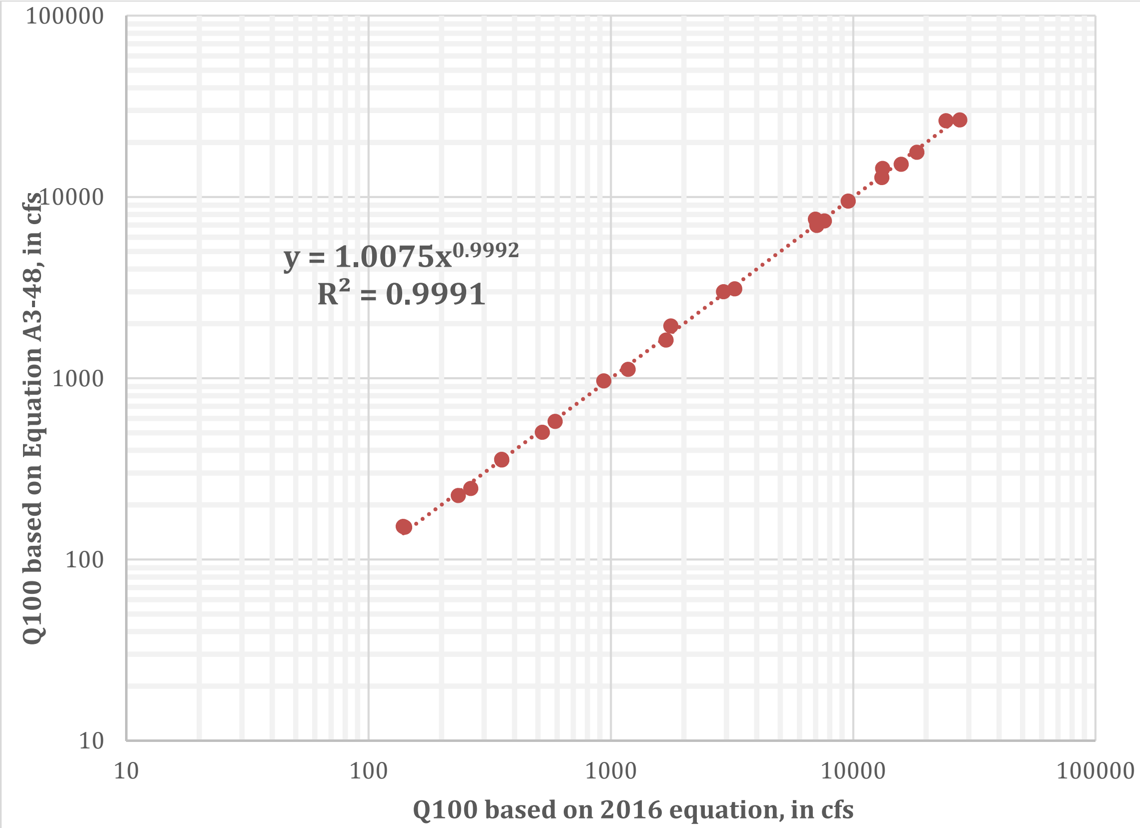


Figure A3-35: Comparison of estimated 100-year discharge from Equation A3-48 to estimated values from 2016 published equations for 24 gaging stations in the Appalachian Plateau Region

Summary

The regression equations for estimating the 1.25-, 1.5-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year flood discharges were updated for the combined Piedmont-Blue Ridge Region and the Appalachian Plateau Region in western Maryland. A new regional skew analysis was performed, and flood frequency curves were updated and revised for 133 stations, including 55 stations that were discontinued prior to 1999, 52 stations with additional data since 1999 (additional 13 years of record), and 26 new stations with at least 10 years of record. Most of the new stations are urban watersheds in Baltimore County or the City of Baltimore.

Eleven stations were identified as outliers in the regression analysis and two stations were combined with nearby stations, resulting in 120 stations being used in the regression analysis: 96 stations in the Piedmont-Blue Ridge Region and 24 stations in the Appalachian Plateau Region. The final regression equations for the Piedmont-Blue Ridge Region were based on drainage area in square miles and the percentages of limestone, and impervious area. These were the most statistically significant explanatory variables across all recurrence intervals. With the addition of the new stations in Baltimore County and the City of Baltimore, there are now 37 stations with impervious area greater than 10 percent (only 32 urban stations were used in equations), based on the Maryland Office of Planning generalized land use data. The urban regression equations documented in the September 2010 Hydrology Panel report are only applicable to the Piedmont Region and were based on just 16 stations. Equations A3-31 to A3-40 are now applicable to urban watersheds in the Piedmont and Blue Ridge Regions.

The final regression equations for the Appalachian Plateau (Equations A3-41 to A3-50) are based on drainage area in square miles and land slope in feet per foot, the same explanatory variables used in the 2006 analysis. Comparisons of the new and previous equations indicate little difference for the 100-year flood discharges.

The regression equations documented in this report are based on updated annual peak data through the 2012 water year where the data are available. This is an additional 13 years of record at many of the gaging stations including several major floods that occurred since 1999. In addition, 26 new stations (mostly urban stations) were used in the regression analysis. The number of urban gaging stations used in the regression analysis doubled from 16 to 32 stations for the current analysis. The regression equations (Equations A3-31 to A3-40) for the Piedmont-Blue Ridge Region are applicable to both rural and urban watersheds. The regression equations for the Appalachian Plateau Region (Equations A3-41 to A3-50) are only applicable to rural watersheds and give essentially the same estimates of the T-year discharges as the previous equations but are based on additional data.

Attachment WM-1. T-year flood discharges (QT) for the 120 stations used in the regression analysis for the Piedmont-Blue Ridge Region and the Appalachian Plateau Region.

Station Number	Q1.25 (cfs)	Q1.50 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
01495000	1790	2250	2890	4850	6450	8860	10950	13300	16000	20000
01495500	1320	1440	1650	2440	3230	4650	6100	7990	10400	14800
01496000	1010	1220	1540	2530	3400	4760	6010	7480	9220	12000
01496080	125	212	280	491	668	935	1170	1430	1730	2190
01496200	617	808	1090	2120	3110	4820	6490	8590	11200	15600
01577940	92	118	156	293	427	663	899	1200	1580	2240
01578500	2490	3280	4480	8920	13400	21200	29200	39500	52500	75300
01578800	272	340	432	700	909	1210	1460	1730	2030	2460
01579000	441	591	816	1600	2330	3540	4700	6090	7780	10500
01580000	2430	2950	3660	5700	7290	9580	11500	13600	15900	19400
01580200	2890	3580	4550	7610	10200	14200	17800	21900	26700	34200
01581500	756	962	1250	2140	2870	3960	4910	5980	7180	9000
01581700	1270	1830	2600	4800	6360	8340	9790	11200	12600	14300
01581752	276	365	502	1010	1530	2440	3370	4560	6090	8740
01581810	686	897	1220	2360	3470	5410	7320	9720	12700	17900
01581870	531	690	930	1810	2670	4190	5700	7630	10100	14300
01581940	36	53	83	225	495	730	1140	1700	2500	4100
01581960	491	618	794	1320	1740	2370	2900	3490	4150	5140
01582000	1500	1820	2270	3570	4600	6100	7380	8790	10400	12700
01582510	112	178	288	715	1140	1840	2500	3280	4190	5620
01583100	524	636	796	1310	1760	2460	3110	3880	4780	6230
01583495	52	77	116	249	367	547	705	881	1080	1370
01583500	1240	1630	2210	4330	6420	10100	13800	18500	24400	34800
01583580	45	68	107	268	443	768	1110	1550	2110	3100
01583979	500	625	789	1260	1610	2110	2520	2960	3430	4100
01584050	310	441	645	1400	2150	3440	4690	6240	8140	11300
01584500	1460	1930	2610	4790	6630	9440	11900	14700	17900	22700
01585090	704	844	1020	1480	1790	2200	2520	2840	3170	3620
01585095	320	340	405	680	980	1500	2050	2700	3600	5000
01585100	1140	1370	1690	2670	3490	4740	5840	7100	8550	10800
01585104	337	415	521	838	1090	1470	1790	2140	2540	3140
01585200	421	559	749	1300	1720	2310	2770	3270	3780	4519
01585225	134	142	156	210	260	333	400	475	560	680
01585230	1400	1680	2040	3030	3760	4760	5560	6410	7320	8630
01585300	788	982	1250	2070	2740	3750	4630	5620	6740	8450
01585400	188	237	316	633	984	1680	2450	3530	5030	7930

Station Number	Q1.25 (cfs)	Q1.50 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
01585500	117	166	243	538	836	1370	1900	2570	3410	4860
01586000	1520	1850	2360	4100	5750	8560	11300	14700	19000	26300
01586610	711	928	1240	2230	3070	4360	5510	6810	8310	10600
01587000	2270	2840	3660	6360	8770	12600	16300	20600	25700	34000
01587050	68	91	126	255	384	615	849	1150	1530	2200
01587500	1520	1990	2720	5510	8380	13600	19100	26200	35500	52100
01588000	332	463	674	1520	2440	4160	6000	8440	11700	17500
01589000	6120	7920	10500	18800	26000	37300	47400	59100	72700	93900
01589100	465	540	645	986	1280	1760	2190	2710	3320	4320
01589180	58	66	85	175	265	430	630	880	1200	1800
01589197	495	548	636	995	1380	2110	2900	3980	5440	8230
01589200	147	190	262	596	1020	1970	3160	5010	7850	14000
01589240	599	787	1080	2210	3400	5600	7910	11000	15000	22300
01589300	1310	1580	2000	3640	5360	8610	12100	16800	23200	35100
01589330	1260	1490	1830	2980	4040	5820	7540	9670	12300	16700
01589352	4730	5920	7580	12900	17400	24400	30700	38000	46500	59700
01589440	636	830	1150	2500	4080	7320	11100	16500	24200	39700
01589464	420	538	703	1200	1610	2210	2730	3310	3950	4910
01591000	768	1050	1510	3310	5230	8840	12700	17700	24400	36500
01591400	669	846	1100	1960	2720	3960	5110	6490	8140	10800
01591700	652	895	1260	2530	3700	5610	7390	9510	12000	16000
01593350	94	130	185	382	572	896	1210	1600	2070	2850
01594000	2090	2660	3500	6370	9000	13400	17500	22500	28600	38600
01594930	253	308	381	581	729	934	1100	1270	1460	1730
01594936	69	92	127	258	389	624	861	1170	1550	2230
01594950	76	98	130	242	345	517	681	880	1120	1530
01596005	20	39	50	82	107	144	177	212	252	312
01596500	1060	1250	1520	2380	3100	4220	5240	6420	7800	10000
01597000	305	378	485	844	1170	1720	2230	2860	3630	4900
01598000	2180	2710	3450	5880	8030	11500	14600	18400	22900	30100
01599000	1270	1520	1880	2970	3890	5310	6580	8040	9730	12400
01601500	4140	4900	6040	10100	14000	20900	27800	36600	47800	67500
01609000	2490	3120	3970	6510	8540	11500	14000	16800	19900	24500
01609500	190	223	267	398	502	657	789	938	1110	1360
01610105	41	46	54	74	88	107	121	137	153	176
01610150	219	283	375	666	912	1290	1620	2000	2440	3110
01610155	2180	2880	3860	6930	9460	13200	16400	20000	24100	30000
01612500	315	399	518	896	1220	1720	2160	2680	3280	4210
01613150	155	189	236	376	489	656	800	960	1140	1410

Station Number	Q1.25 (cfs)	Q1.50 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
01613160	60	74	94	151	197	263	319	380	448	550
01614500	5380	6340	7620	11400	14400	18700	22400	26600	31200	38200
01619000	986	1210	1540	2570	3460	4880	6160	7670	9450	12300
01619475	11	15	21	44	68	109	152	207	276	398
01619500	1580	2020	2620	4520	6100	8520	10600	13100	15800	20100
01637000	268	387	584	1410	2320	4080	5980	8530	11900	18100
01637500	1470	1900	2510	4540	6320	9140	11700	14700	18300	23900
01637600	141	192	274	600	949	1610	2310	3240	4480	6730
01639000	6690	7580	8740	12000	14400	17800	20600	23600	26900	31700
01639140	1310	1550	1890	2940	3820	5180	6390	7800	9430	12000
01639500	2250	2700	3360	5560	7550	10900	14000	17800	22400	30100
01640000	228	306	424	857	1280	2020	2760	3680	4830	6800
01640500	179	255	378	876	1410	2420	3490	4910	6760	10100
01640700	102	136	190	396	609	1000	1410	1950	2670	3920
01640965	59	80	115	251	392	653	925	1280	1740	2570
01640970	146	213	325	796	1320	2340	3440	4920	6900	10500
01641000	482	624	821	1400	1860	2520	3060	3650	4300	5230
01641500	71	100	148	346	568	1000	1480	2130	3020	4670
01642000	13000	14800	16900	22600	26500	31700	35700	39900	44300	50400
01642400	232	314	440	891	1320	2070	2780	3670	4760	6580
01642500	1600	1960	2480	4150	5600	7920	10000	12600	15500	20300
01643000	13900	15900	18600	26600	33000	42400	50400	59300	69300	84500
01643395	46	68	105	266	449	811	1210	1750	2490	3850
01643500	1500	1900	2520	4780	7060	11200	15400	20900	28000	40600
01644371	87	106	134	241	347	538	734	990	1320	1920
01644375	93	128	184	411	660	1140	1660	2360	3310	5060
01644380	45	88	175	530	830	1320	1800	2300	2900	3850
01644420	53	70	97	189	275	419	557	725	928	1260
01644600	1720	2100	2600	4400	5900	8400	10800	13600	17400	23100
01645000	2340	3010	4050	7980	12000	19400	27100	37300	50500	74400
01645200	341	454	622	1210	1760	2680	3560	4620	5910	8040
01646550	492	657	887	1570	2110	2860	3480	4140	4850	5860
01647720	520	660	850	1700	2600	4350	7500	9200	13500	27000
01650050	313	368	470	910	1370	2250	3150	4250	5700	8300
01650085	40	53	79	200	351	681	1080	1680	2400	4000
01650190	94	128	181	380	582	945	1310	1790	2390	3440
01650500	829	1000	1280	2320	3400	5400	7530	10400	14200	21200
01651000	2580	3190	4050	6870	9350	13300	17000	21300	26400	34700
03075450	20	23	28	41	51	68	74	91	105	140

Station Number	Q1.25 (cfs)	Q1.50 (cfs)	Q2 (cfs)	Q5 (cfs)	Q10 (cfs)	Q25 (cfs)	Q50 (cfs)	Q100 (cfs)	Q200 (cfs)	Q500 (cfs)
03075500	2910	3490	4280	6660	8580	11400	13900	16700	19800	24600
03075600	18	23	30	54	75	111	144	184	232	310
03076500	4570	5360	6350	8920	10700	13100	14900	16800	18700	21400
03076600	1150	1370	1640	2040	2340	3600	4800	5400	5800	6400
03077700	18	25	36	78	145	220	320	450	640	1000
03078000	1500	1730	2040	3000	3690	4780	5710	6750	7930	9720

Attachment WM-2. Watershed characteristics used in the regression analysis for the 96 gaging stations in the Piedmont-Blue Ridge Region.

Station number	Years of record	Drainage area (sq mi)	Limestone (percent)	Impervious area (percent)	Forest cover (percent)
01495000	80	53.36	0	2.5	35.4
01495500	12	26.46	0	2.5	30.9
01496000	37	24.87	0	1.9	22.8
01496080	10	1.75	0	1.5	94.3
01496200	27	9.00	0	1	14.8
01577940	16	0.67	0	1.6	28
01578500	19	191.66	0	1.9	33.6
01578800	10	1.25	0	2.5	15.3
01579000	22	5.08	0	2.9	18.9
01580000	86	94.31	0	1	35.8
01580200	11	127.16	0	1.2	34.7
01581500	38	8.79	0	12.9	22.3
01581700	45	34.64	0	8.1	27.1
01581752	11	2.47	0	42.9	5.2
01581810	12	27.46	2	4.9	25.7
01581870	13	15.76	0	7.8	19.8
01581940	10	0.77	0	2.5	74.1
01581960	13	9.66	0	4.8	35.4
01582000	69	53.70	0	1.3	41
01582510	14	1.39	0	2.4	31.2
01583100	23	12.45	0	3.4	31.2
01583495	10	0.23	0	0	27.5
01583500	68	60.31	0	1.5	34
01583580	26	1.49	0	8.4	64.5
01583979	11	2.10	0	40.2	12.7
01584050	37	9.31	0	5.7	18.5
01584500	72	36.04	0	3.5	28.2
01585090	18	2.58	0	44	11.7
01585095	17	1.36	0	42.9	5.6
01585100	40	7.56	0	37.7	18.6
01585104	13	2.44	0	22.5	28.6
01585200	46	2.31	0	42.1	4.1
01585225	16	0.14	0	41.1	0.5
01585230	16	3.50	0	45.4	1.8
01585300	29	4.52	0	25.3	29.9
01585400	29	1.94	0	36.8	21.4

Station number	Years of record	Drainage area (sq mi)	Limestone (percent)	Impervious area (percent)	Forest cover (percent)
01585500	64	3.26	0	4.2	19.5
01586000	67	55.48	3.1	5.4	23
01586610	30	28.01	0.1	4.9	31.7
01587000	24	164.23	1.74	4.6	31.5
01587050	11	0.49	0	10	5.9
01587500	32	64.26	0	4	31.4
01588000	43	11.40	0	4.6	20.5
01589000	23	284.71	0	4.7	33.3
01589100	47	2.47	0	33.8	24.5
01589180	14	0.31	0	42	15.8
01589197	14	4.09	0	37.7	11.8
01589200	17	4.89	0	14.6	26.5
01589240	12	19.27	0	16.6	35.1
01589300	34	32.59	0	19.5	30.7
01589330	31	5.52	0	41.1	8.4
01589352	14	63.57	0	41.3	16.5
01589440	47	25.21	0	11.4	35.9
01589464	9	2.26	0	41.7	1.4
01591000	68	34.95	0	1.4	33.3
01591400	46	22.86	0	4.3	25.3
01591700	34	27.31	0	8.9	32.7
01593350	11	1.06	0	34.8	5.4
01594000	59	98.25	0	11	28.6
01614500	85	502.44	41.5	1.6	32.6
01619000	27	93.90	64.6	3.9	56.9
01619475	11	0.11	81.72	0	9.7
01619500	85	280.89	75.6	4.8	24.8
01637000	30	8.76	0	0.8	54.8
01637500	65	67.33	0	0.8	46.6
01637600	11	2.32	0	1.5	37.6
01639000	72	172.7	1.3	0.8	13.1
01639140	12	31.07	2.4	3.7	13.6
01639500	65	102.98	1.1	1.8	22
01640000	31	8.11	76.53	6.9	19.5
01640500	53	6.10	0	0.5	80.8
01640700	11	1.12	0	0	4.7
01640965	13	2.19	0	0	96
01640970	10	3.91	0	1.2	76.7

Station number	Years of record	Drainage area (sq mi)	Limestone (percent)	Impervious area (percent)	Forest cover (percent)
01641000	43	18.69	16.23	1.8	77.3
01641500	39	7.30	0	0	100
01642000	35	665.1	14.14	1.7	28
01642400	10	2.67	0	0.1	6.8
01642500	49	82.37	0	1.3	26.4
01643000	84	816.45	12.3	2.4	27
01643395	9	1.18	0	1.5	86.4
01643500	62	62.94	0	2	38.3
01644371	9	0.42	0	28	23.5
01644375	9	1.29	0	53.5	8.6
01644380	9	0.81	0	1.5	42.5
01644420	10	0.28	0	0	15.2
01644600	12	53.89	0	23.1	27.2
01645000	48	102.19	0	11.6	27.2
01645200	30	3.70	0	26.2	13.6
01646550	40	4.09	0	32.4	5.2
01647720	11	9.68	0	9.9	23.2
01650050	10	2.51	0	5.1	33.6
01650085	10	0.35	0	3.8	66.2
01650190	10	0.49	0	5.4	4.4
01650500	75	21.23	0	11.6	26.3
01651000	47	49.43	0	25.1	19.7

Attachment WM-3. Watershed characteristics used in the regression analysis for the 24 gaging stations in the Appalachian Plateau Region.

Station Number	Years of Record	Drainage area (sq mi)	Land Slope (ft/ft)
01594930	26	8.23	0.155
01594936	28	1.91	0.144
01594950	25	2.36	0.130
01596005	14	1.43	0.099
01596500	64	48.53	0.203
01597000	33	16.75	0.194
01598000	24	115.87	0.227
01599000	82	72.74	0.164
01601500	83	247.03	0.209
01609000	33	149.45	0.202
01609500	25	5.00	0.166
01610105	15	0.65	0.160
01610150	18	10.27	0.115
01610155	24	102.71	0.184
01612500	17	17.28	0.143
01613150	22	4.60	0.113
01613160	12	1.24	0.129
03075450	12	0.55	0.066
03075500	72	134.16	0.115
03075600	22	0.52	0.071
03076500	89	294.14	0.112
03076600	48	49.07	0.168
03077700	12	1.07	0.085
03078000	65	63.77	0.101

Attachment WM-4. Computation of the Equivalent Years of Record for Regression Equations for the Piedmont-Blue Ridge Region and the Appalachian Plateau Region in Maryland.

Computational Procedure

The variance (standard error squared (SE^2)) of the x-year flood at a gaging station is estimated as

$$SE_x^2 = (S^2/N) * R_x^2 \qquad \text{(A3-51)}$$

where S is the standard deviation of the logarithms (log units) of the annual peak discharges at the gaging station, N is the actual record length in years and R_x is a function of recurrence interval x and skew (G) at the gaging station. The standard error increases as the recurrence interval increases, given the same record length.

In Equation A3-51, the standard error of the x-year flood at a gaging station is inversely related to record length N and directly related to the variability of annual peak flows represented by S (standard deviation) and G (skew). If the standard error of the x-year flood is interchanged with the standard error of estimate (SE) of the regression equation, then Equation A3-51 can be used to estimate the years of record needed to obtain that standard error of estimate. Rearranging Equation A1 and solving for N gives Equation A3-52 below.

The equivalent years of record of the regression estimate is defined as the number of years of actual streamflow record required at a site to achieve an accuracy equivalent to the standard error of the regional regression equation. The equivalent years of record (N_r) of a regression equation is computed as follows (Hardison, 1971):

$$N_r = (S/SE)^2 * R^2 \qquad \text{(A3-52)}$$

where S is an estimate of the standard deviation of the logarithms of the annual peak discharges at the ungaged site, SE is the standard error of estimate of the regional regression estimates in logarithmic units, and R^2 is a function of recurrence interval and skew and is computed as (Stedinger and others, 1993):

$$R^2 = 1 + G * K_x + 0.5 * (1 + 0.75 * G^2) * K_x^2 \qquad \text{(A3-53)}$$

where G is an estimate of the average skew for a given hydrologic region, and K_x is the Pearson Type III frequency factor for the x-year flood and skew G.

Computational Details

The equivalent years of record are estimated for the regional regression equations and computations in Equations A3-52 and A3-53 require an estimate of the average standard deviation and average skew for all gaging stations in a given region. For the Piedmont-Blue Ridge Region, the average standard deviation (S) is 0.3070 log units and the average skew (G) is 0.48. For the Appalachian Plateau Region, the average standard deviation (S) is 0.2353 log units and the average skew (G) is 0.39. The lower standard deviation and skew in the Appalachian Plateau Region is indicative of less variability in the annual peak flows in this region.

For the Piedmont-Blue Ridge Region, the pertinent data are S=0.3070 log units and G=0.48 and:

Recurrence Interval (years)	K value	SE^2 (log units squared)	Equivalent years of record
1.25	-0.85624	0.04691	2.0
1.50		0.03953	(2.4) Estimated
2	-0.07972	0.03286	2.8
5	0.80991	0.02203	8.3
10	1.32181	0.01798	14
25	1.90425	0.01615	24
50	2.30094	0.01707	29
100	2.67165	0.01944	32
200	3.02262	0.02367	31
500	3.46270	0.03190	29

The equivalent years of record are estimated using Equations A3-52 and A3-53 using the above data.

For the Appalachian Plateau Region, the pertinent data are S=0.2353 log units and G=0.39 and:

Recurrence Interval (years)	K value	SE^2 (log units squared)	Equivalent years of record
1.25	-0.85500	0.01544	1.3
1.50		0.00875	(4.4) Estimated
2	-0.06485	0.00729	7.5
5	0.81712	0.00791	11
10	1.31597	0.01136	12
25	1.87730	0.01489	13
50	2.25628	0.02012	13
100	2.60827	0.02582	12
200	2.93974	0.03341	11
500	3.35346	0.04409	10

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APPENDIX 4
REGRESSION EQUATIONS FOR THE ESTIMATION OF BANKFULL CROSS-SECTION AREA, DEPTH AND WIDTH AS FUNCTIONS OF UPSTREAM DRAINAGE AREA

Background

One method of estimating the time of concentration of a watershed is to estimate the travel time through the hydraulic flow path. An estimation of the time required for a particle of water to travel through the channel network is one element in the hydraulic flow path approach. This channel travel time is usually estimated by computing the velocity with the Manning equation under bankfull conditions.

Often, it is not feasible to send a crew into the field to make the measurements needed to define the bankfull depth, width and area. When field surveys are not practical, the Panel recommends use of the regression equations that estimate the bankfull depth, area and width as functions of the upstream drainage area. The US Fish and Wildlife Service (FWS) and the Maryland Department of Transportation State Highway Administration, in cooperation with the US Geological Survey, developed the three sets of equations presented in this appendix.

A4-1. The FWS Equations

A4-1.1 Equations for Piedmont Hydrologic Region

Reference: McCandless, Tamara L., and Everett, Richard A., *Maryland Stream Survey: Bankfull Discharge and Channel Characteristics of Streams in the Piedmont Hydrologic Region,* US Fish and Wildlife Service, Chesapeake Bay Field Office, CBFO-S02-01, 2002

Measurements were made at 23 sites having drainage areas between 1.47 and 102.00 square miles. The equations are:

$$\text{Cross-sectional Area} = 17.42\ DA^{0.73}$$
$$\text{Width} = 14.78\ DA^{0.39}$$
$$\text{Depth} = 1.18\ DA^{0.34}$$

where DA is the upstream drainage area in square miles. Figure A4-1 [from McCandless and Everett (2002)] illustrates the quality of the agreements.

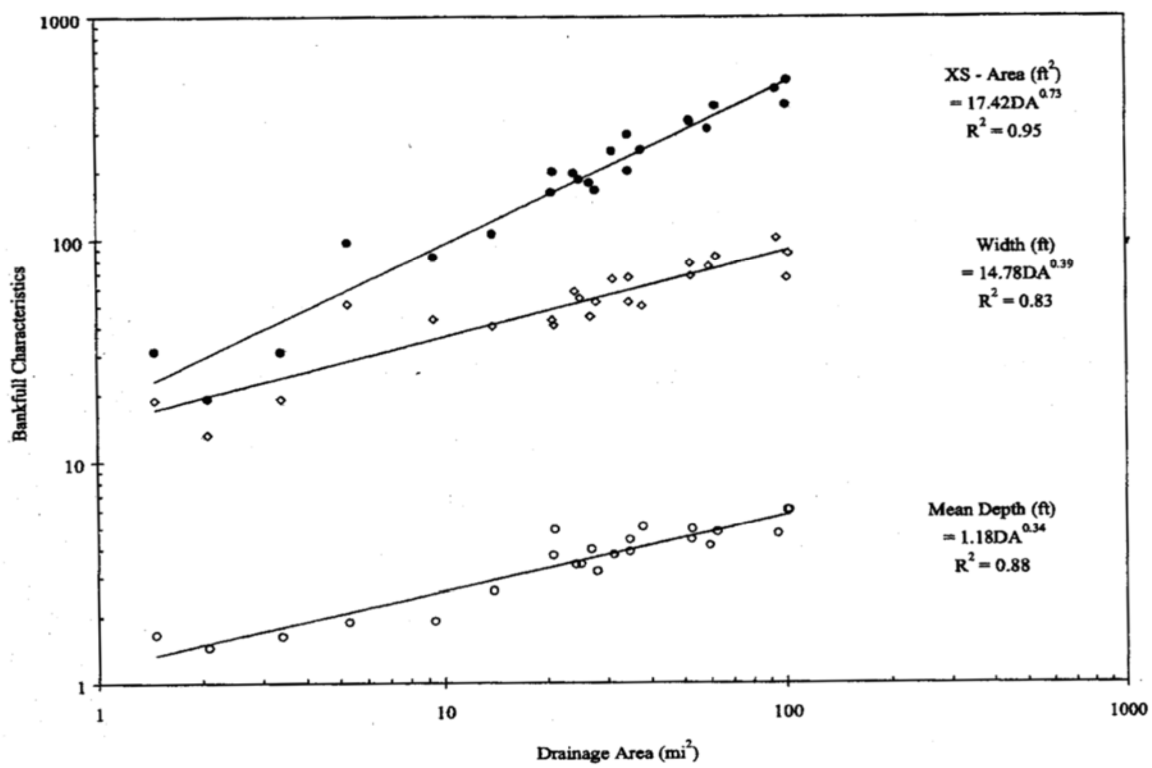


Figure A4-1: Bankfull channel dimensions as a function of drainage area for Maryland Piedmont survey sites (n = 23) (From McCandless and Everett, 2002)

A4-1.2 Equations for Allegheny Plateau and the Valley and Ridge Hydrologic Regions

Reference: McCandless, Tamara L., *Maryland Stream Survey: Bankfull Discharge and Channel Characteristics of Streams in the Allegheny Plateau and Valley and Ridge Hydrologic Region*, US Fish and Wildlife Service, Chesapeake Bay Field Office, CBFO-S03-01, 2003

Measurements were made at 14 sites having drainage areas between 0.2 and 73.1 square miles. The equations are:

$$\text{Cross-sectional Area} = 13.17\ DA^{0.75}$$
$$\text{Width} = 13.87\ DA^{0.44}$$
$$\text{Depth} = 0.95\ DA^{0.31}$$

where DA is the upstream drainage area in square miles. Figure A4-2 [from McCandless (2003)] illustrates the quality of the agreements.

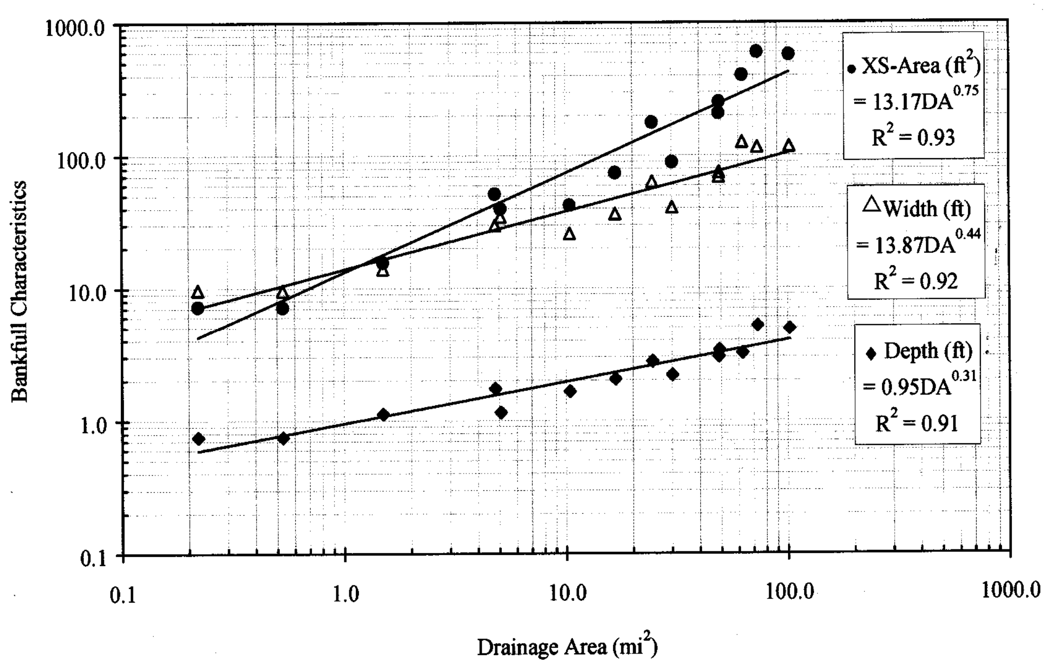


Figure A4-2: Bankfull channel dimensions as a function of drainage area for Appalachian Plateau / Valley & Ridge survey sites (n = 14) [from McCandless (2003)]

A4-1.3 Equations for the Coastal Plain Hydrologic Region

Reference: McCandless, Tamara L., *Maryland Stream Survey: Bankfull Discharge and Channel Characteristics of Streams in the Coastal Plain Hydrologic Region*, US Fish and Wildlife Service, Chesapeake Bay Field Office, CBFO-S03-02, 2003

Measurements were made at 14 sites having drainage areas between 0.3 and 113 square miles. The equations are:

$$\text{Cross-sectional Area} = 10.34\ DA^{0.70}$$
$$\text{Width} = 10.30\ DA^{0.38}$$
$$\text{Depth} = 1.01\ DA^{0.32}$$

where DA is the upstream drainage area in square miles. Figure A4-3 [from McCandless (2003)] illustrates the quality of the agreements.

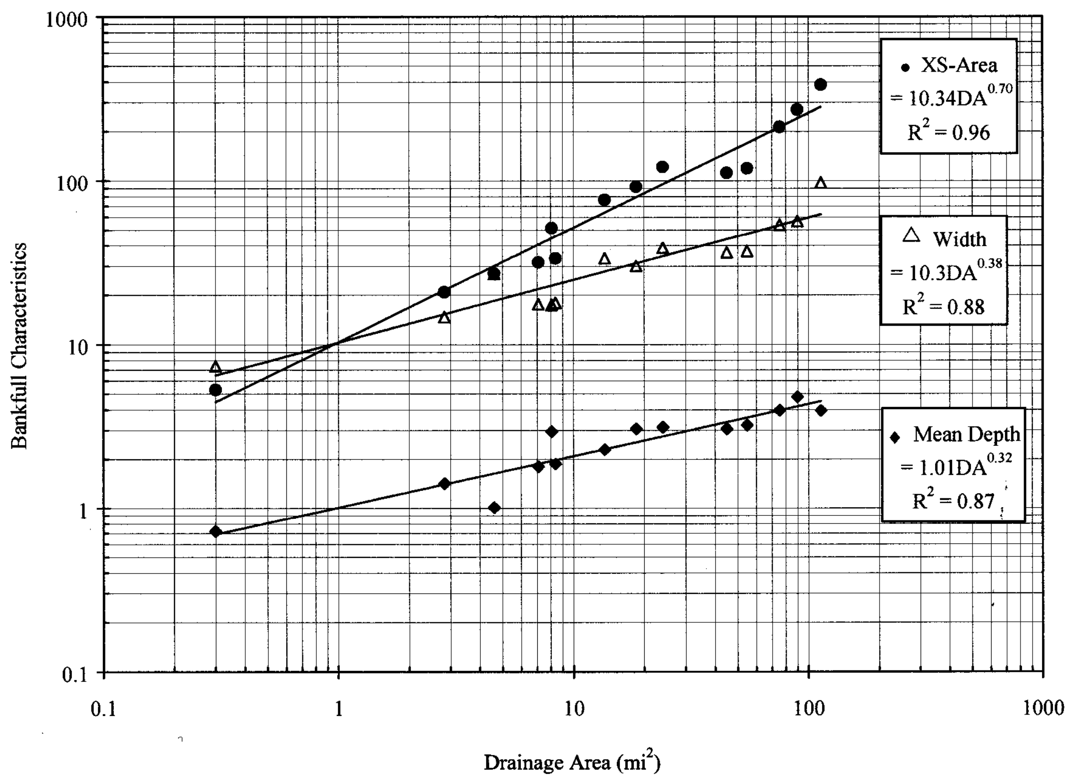


Figure A4-3: Bankfull channel dimensions as a function of drainage area for Coastal Plain survey sites (n = 14) [from McCandless (2003)]

A4-2 Manual Use of the FWS Equations

A4-2.1 Determining the Time of Concentration

The engineer will need to identify the channel portion of the longest flow path. The engineer should then determine the drainage area at the upstream and downstream extremes of the flow path. We will denote these areas as DA_u and DA_d, respectively. The geometric mean of these two values is determined as:

$$\overline{DA} = \exp\left[\frac{\ln(DA_u) + \ln(DA_d)}{2}\right] \qquad \text{(A4-1)}$$

The geometric mean of the upstream and downstream drainage areas is then substituted into the FWS channel geometry equations to determine a bankfull width and depth for this mean drainage area. The width and depth are then combined with the channel roughness and slope to determine a bankfull velocity. The channel length of the longest flow path is then divided by the bankfull velocity to determine the travel time associated with the channel portion of the time of concentration.

Example: Determine the channel portion of travel time for a 2000-foot long channel with a slope of 0.0015 ft/ft in the Maryland Piedmont physiographic province. The drainage area at the upstream end of the channel is 5.0 square miles. At the downstream end, the drainage area is 10.0 square miles. Use a channel roughness, n=0.05.

Solution: First, determine the geometric mean drainage area:

$$\overline{DA} = \exp\left[\frac{\ln(5) + \ln(10)}{2}\right] = 7.07\ \text{mi}^2$$

Using this value, the bankfull channel width and depth in the Piedmont are calculated:

$$w = 14.78(7.07)^{0.39} = 31.69\ \text{feet}$$

$$d = 1.18(7.07)^{0.34} = 2.29\ \text{feet}$$

Now use Manning's equation to determine the bankfull velocity, assuming a rectangular cross section:

$$v = \frac{1.49}{0.05}\left[\frac{31.69 \cdot 2.29}{2(2.29) + 31.69}\right]^{2/3} (0.0015)^{1/2} = 1.83\ \text{ft/s}$$

The channel portion of the travel time is then:

$$t_{channel} = \frac{l}{v} = \frac{2000}{1.83} = 1093 \text{ sec} = 18.2 \text{ minutes}$$

This travel time would be added to the overland and swale portions of the travel time along the longest flow path to determine the overall t_c value.

A4-2.2 Determining the Rating Curve for Reach Routing

As in the case of determining the time of concentration, t_c, the engineer will need to know the upstream and downstream drainage areas for the reach being studied. The engineer will additionally need the reach slope, roughness values for in-bank and out-of-bank flow, and cross-section geometry for the out-of-bank portion of the flow, presumably determined from a topographic map or digital terrain data. As in the t_c calculations, the engineer must determine the geometric mean drainage area and use this to determine the bankfull channel geometry – idealizing the channel as a rectangular section with bankfull width and depth determined from the FWS equations for the appropriate region using the geometric mean drainage area. (Note: Alternatively, the engineer may choose to simply use the drainage area from the location of the selected cross-section to determine the bankfull width and depth from the FWS equations.) This channel portion of the cross-section is then superimposed on the cross-section from the topographic map with the channel cross-section replacing the topographic map or digital terrain data measurements at the lowest observed elevation from the topographic map. That is, the topographic map is assumed to indicate only the top-of-bank elevation, so the rectangular cross-section is "carved" into the cross-section such that the channel incises a depth, *d*, into the topographically-derived cross-section.

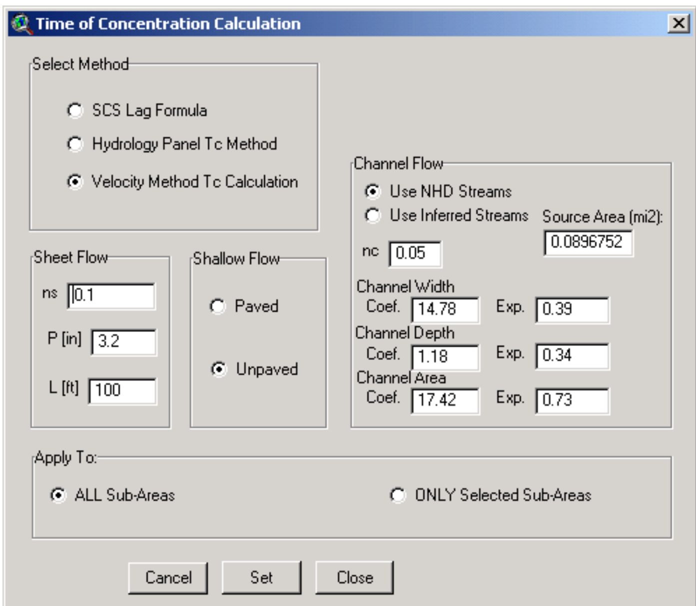
Once this cross-section is determined, the engineer need only choose an appropriate set of stages over which to apply Manning's equation to determine channel velocity and ultimately discharge. For each selected stage, the derived discharge and cross-sectional area ("End Area") should be recorded.

A4-3. Using the FWS Equations within GISHydro

The FWS channel geometry equations have an influence on two different elements of the WinTR-20 input file: the time of concentration calculation and the rating curve for reach routing. Additionally, the way these equations are to be used will likely differ depending on whether GISHydro is being used to generate the WinTR-20 input file, or whether the input file is being developed manually.

A4-3.1 Time of Concentration

GISHydro allows for the calculation of the time of concentration, t_c, using three different methods: the SCS lag equation, the Hydrology Panel equation, and the velocity method. The velocity method is the recommended method for t_c calculation. The time of concentration dialog box is shown below.



If the user selects the velocity method then the "Channel Flow" portion of the dialog directly reflects how the FWS equations' influence the t_c calculation. GISHydro detects the physiographic province(s) in which the watershed is located and performs an area-weighted calculation to determine the coefficients and exponents of the width, depth, and cross-sectional area channel geometry equations. (The coefficients shown in the illustrated dialog box correspond to the Piedmont province.) Once all parameters have been set, GISHydro proceeds in the calculation of velocity on a pixel-by-pixel basis all along the longest flow path. The channel portion of the longest flow path is indicated by either the minimum source area (the inferred streams option) or by the upstream extent of the 1:100,000 NHD (National Hydrography Dataset) produced by the USGS. Normal depth at bankfull conditions is assumed; thus, the local slope, channel roughness, and channel geometry may be used in Manning's equation to determine a velocity. Note that the channel geometry changes slightly on a pixel-by-pixel basis because the drainage area increases in a known way along the flow path. The local drainage area is used to determine the local channel bankfull width, depth, and area. The GIS determines the incremental flow length associated with each pixel and divides this incremental length by the local flow velocity to give an incremental travel time. Incremental travel times for all pixels along the longest flow path are summed to calculate the total travel time. The image below shows a small portion of the calculations along a longest flow path within the Piedmont region. The reader should note that the user does not need to specify the location of the longest flow path; it is determined internally by the GIS.

Attributes Of Longest Path Sub 0

Value	Count	Type	Mixed	Da	Slope	Width	Depth	Xarea	I_length	Tot_length	Vel	I_time	Tot_time
85	1	swale	No	2175	0.0100	-1.00	-1.00	-1.00	141.4	9577	1.60	0.025	1.502
86	1	swale	No	2185	0.0200	-1.00	-1.00	-1.00	141.4	9718	2.26	0.017	1.519
87	1	swale	No	2193	0.0200	-1.00	-1.00	-1.00	100.0	9818	2.26	0.012	1.531
88	1	swale	No	2196	0.0071	-1.00	-1.00	-1.00	100.0	9918	1.35	0.021	1.552
89	1	swale	No	2197	0.0200	-1.00	-1.00	-1.00	100.0	10018	2.26	0.012	1.564
90	1	swale	No	2198	0.0071	-1.00	-1.00	-1.00	141.4	10160	1.35	0.029	1.594
91	1	channel	No	7475	0.0071	21.71	1.65	35.79	141.4	10301	3.18	0.012	1.606
92	1	channel	No	7657	0.0200	21.92	1.66	36.42	100.0	10401	5.38	0.005	1.611
93	1	channel	No	7682	0.0071	21.95	1.67	36.51	141.4	10543	3.20	0.012	1.623
94	1	channel	No	7699	0.0400	21.96	1.67	36.57	141.4	10684	7.62	0.005	1.628
95	1	channel	No	7710	0.0400	21.98	1.67	36.61	141.4	10825	7.62	0.005	1.634
96	1	channel	No	7722	0.0300	21.99	1.67	36.65	100.0	10925	6.60	0.004	1.638
97	1	channel	No	7738	0.0071	22.01	1.67	36.70	141.4	11067	3.21	0.012	1.650
98	1	channel	No	7790	0.0071	22.07	1.67	36.88	141.4	11208	3.21	0.012	1.662

A4.3-2 Determining the Rating Curve for Reach Routing

GISHydro uses the FWS equation to develop the rating curve for each routing reach within the watershed. The user indicates the location of the cross-section within the GISHydro interface by drawing a line perpendicular to the flow path at a representative location along the routing reach. A cross-section editor dialog box appears as shown below. The GIS automatically determines the drainage area at the location of the cross-section. This area is used with the region-appropriate FWS equations to infer the in-bank portion of the channel geometry. The out-of-bank portion of the geometry is determined directly from the DEM. By combining the in-bank and out-of-bank portions of the cross-section and applying Manning's equation with the normal depth assumption at various depths spanning the likely range of flow conditions, GISHydro generates a stage-discharge-end area table which is used directly as input to WinTR-20

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APPENDIX 5
EXAMPLES OF CALIBRATION OF WINTR-20 TO THE REGIONAL REGRESSION EQUATIONS

OVERVIEW

This example illustrates how an existing land use condition WinTR-20 model and calibration window are created and how the model may be adjusted to that window.

The regression equations are developed from USGS gage data that represent land use conditions during the period of gage records. The calibrated WinTR-20 model uses existing land use and hydrograph timing that approximates the watershed characteristics reflected in the regional equations. The calibrated WinTR-20 model is then used to create a model for ultimate development of the watershed which is suitable for deriving design flows.

GISHydro is the primary tool used in this example. The GISHydro database contains all the watershed characteristics that are required to create a WinTR-20 model for existing conditions. It is used to develop WinTR-20 input parameters, calculate peak flows from the regression equations, and predict the calibration confidence limits from statistical standard errors.

PROJECT DESCRIPTION

Design flows for ultimate development of the watershed are required for the replacement of a State Highway bridge No. 1006200 on MD Route 140 in Frederick, Maryland. The study, report and computed discharges will be submitted to Maryland Department of Environment (MDE) for their review and approval as part of obtaining a waterway construction permit for the project.

Bridge No. 1006200 was built in 1932 and needs to be replaced due to its age and structural condition. The structure carries MD 140, Main Street, over Flat Run in Frederick County, Maryland (Figure A5-1).

MD 140 is classified as a Rural Minor Arterial by the Maryland Functional Classification System and the design storm for this roadway is the 50-year event. The hydrology study will develop discharges for the 2-, 10-, 25-, 50- and 100-year storm events. The study focuses on calibrating discharges to the 50-year and 100-year storm events since the 50-year is the design storm and the 100-year is the base flood used to analyze floodplain impacts.

Watershed Description

The 10.8 square mile watershed lies entirely in the Blue Ridge Region but is contained in both Maryland and Pennsylvania. The watershed is characterized by mostly cropland with some urban and forested land use. There are no stream gages in the watershed.

Study Description

The design flows will be based on a WinTR-20 hydrology model using ultimate development with the land use to be derived from zoning maps. This example develops and calibrates the existing condition WinTR-20 model to be within the Blue Ridge-Piedmont Region Regression Equation estimate and the upper 67 percent prediction limit (Tasker Limits).

Figure A5-1
Location of the MD 140 bridge site over Flat Run in Frederick County, MD

Step 1 – Delineate Watershed and Model Structure

The first task is to delineate the watershed and develop the structure of the model (i.e. main stem, tributary reaches, and sub areas). GISHydro is used to delineate the watershed (Figure A5-2). SSURGO soils and the NLUD 2001 land use databases are used in this analysis. The Maryland Office of Planning 2002 land use data cannot be used for this analysis because the drainage area extends outside of the Maryland boundary. The NLUD 2001 data is checked to ensure it adequately represents existing land use. The NLUD

Figure A5-2
Flat Run WWWaWWatershed Delineation

data are developed from satellite imagery and can sometimes overestimate the amount of tree cover.

Select the outlet to delineate the drainage area and then compute the basin statistics. The structure of the watershed model in now considered. This watershed has a semi-elongated shape and comprises one main stem which forms in the upper third of the watershed from three contributing tributaries. There are no structures on the main stem which would provide significant storage such as dams or railroad crossings. Design discharges are only needed at the bridge location at the watershed outlet. For these reasons, a single area watershed structure is first considered.

The NLUD 2001 land use data were checked using aerial photos. Several locations are visually investigated and appear to adequately represent the land use. Figure A5-3 shows the land use categories. The limits, appear to be reasonable.

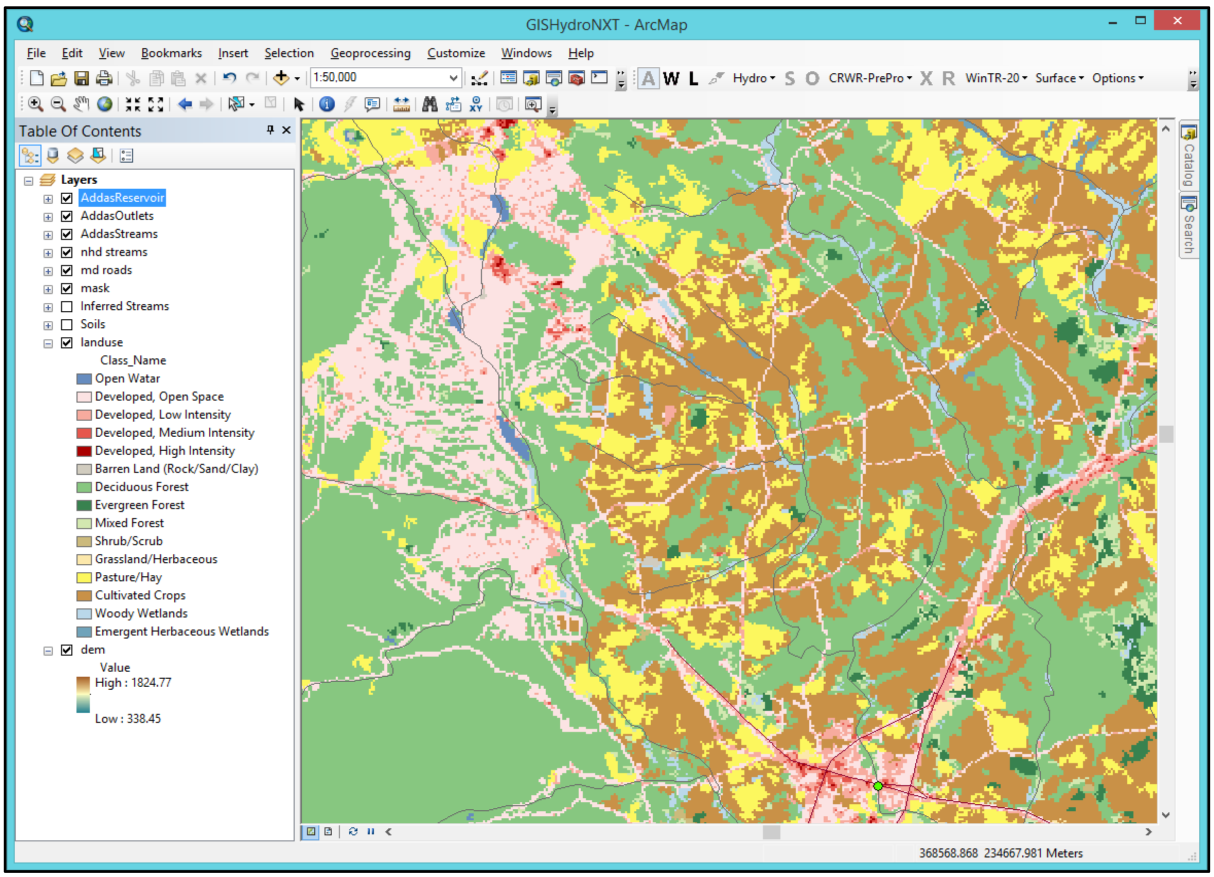


Figure A5-3
Distribution of land use data from the NLUD 2001 database

Step 2 – Compute the Fixed Region Regression Equations and Tasker Program Limits

Use GISHydro to compute the flood discharges and prediction limits for each frequency using the Blue Ridge-Piedmont Region regression equations. Figure A5-4 shows the single basin watershed, the main stem, and its tributaries from GISHydro.

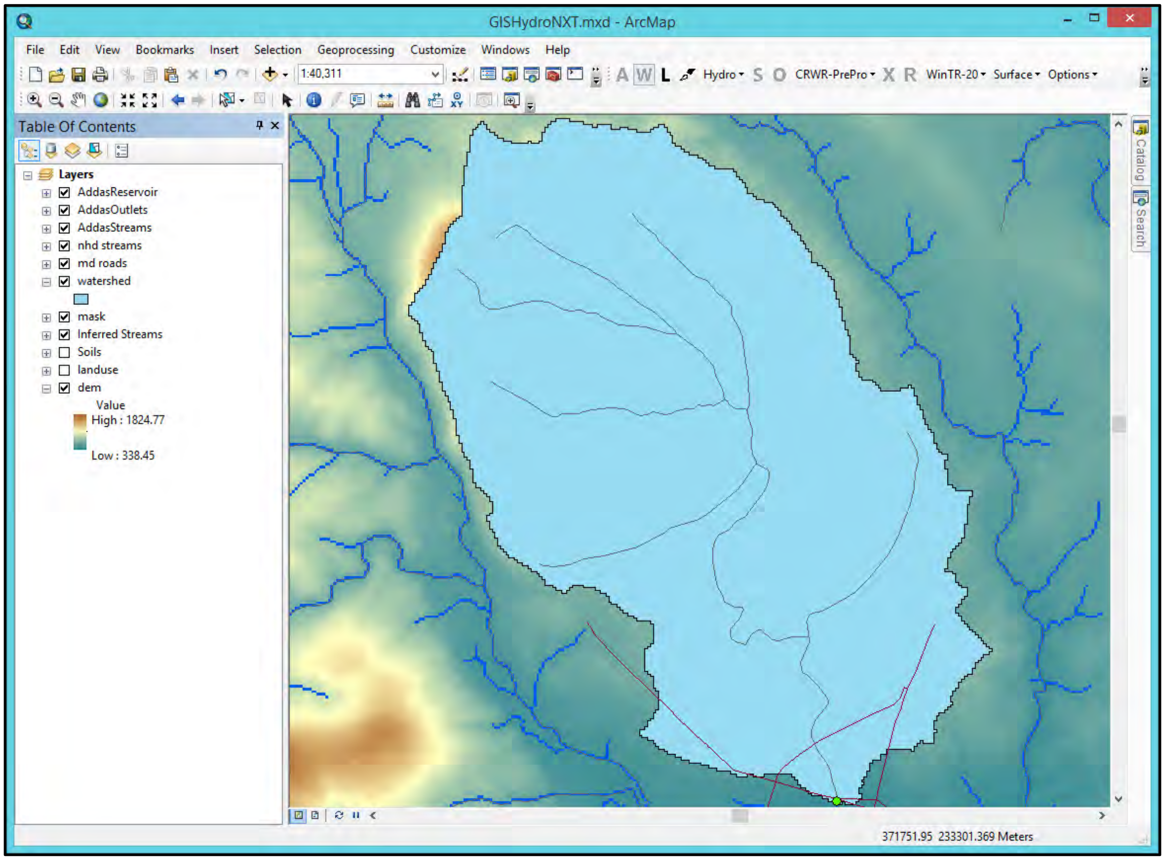


Figure A5-4
Flat Run watershed upstream of MD 140 in Frederick County, MD

Step 3 – Calculate the Time of Concentration

The time of concentration is calculated manually using the TR55 velocity method approach after estimating the location of the longest hydraulic flow path. The total time of concentration is the sum of sheet, shallow concentrated and channel flow segments.

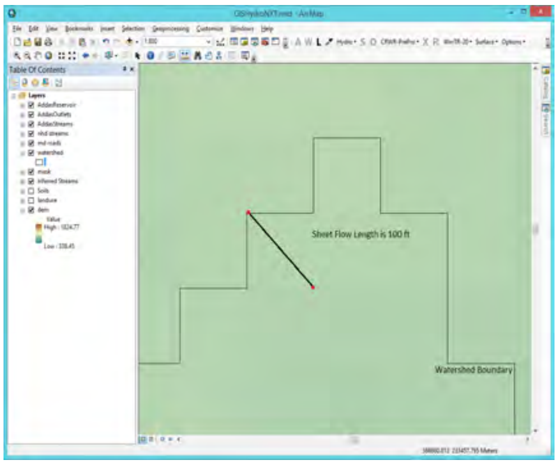


Figure A5-5
Flow lines for Sheet Flow

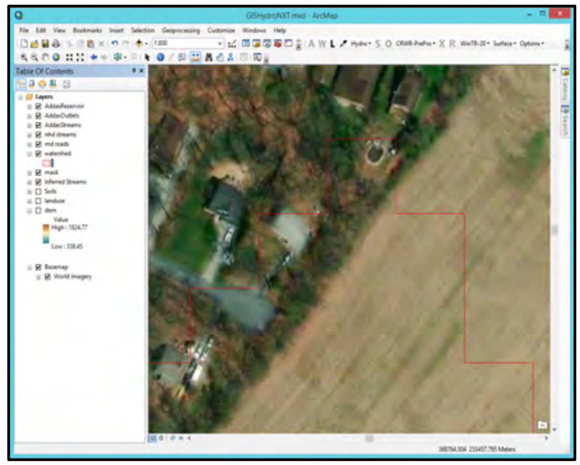


Figure A5-6
Aerial photograph showing
the Sheet Flow Reach

Sheet Flow

From the most hydraulically distant point, determine the length and slope of the flow lines. The elevation values are developed from the digital elevation model (DEM) included with GISHydro (Figure A5-5).

The surface cover for the sheet flow is determined from the aerial photograph which shows residential grass and light tree cover (Figure A5-6).

Computation of the Sheet Flow travel time is shown in calculation sheet A5-A.

Shallow Concentrated Flow

Obtain the slope and distance from the end of the sheet flow to the beginning of the channel as shown in Figure A5-7. Determine whether this is paved or unpaved. The beginning of the channel for this example appears to be a pond. This should be field verified and checked.

Computation of the Shallow Concentrated Flow travel time is given in calculation sheet A5-A.

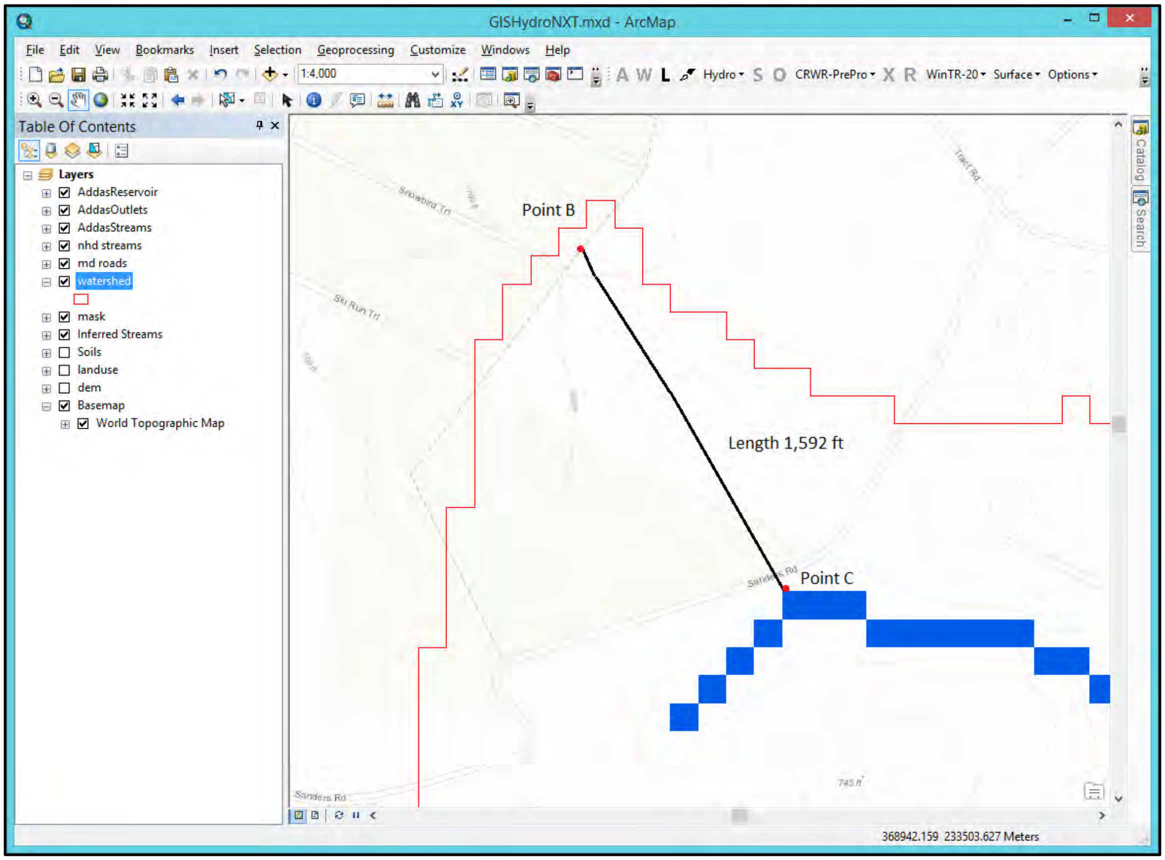


Figure A5-7
Location of the Shallow Concentrated Flow reach.

<u>Channel Flow</u>

The channel flow segment is broken into two reaches because of the difference in drainage areas and the enlargement of the channel between these reaches. The channel is significantly larger and deeper between Points D and E than between Points C and D (Figure A5-8). The U.S. Fish & Wildlife Service (USFWS) Report *Maryland Stream Survey: Bankfull Discharge and Channel Characteristics of Streams in the Alleghany Plateau and the Valley and Ridge Hydrologic Regions*, CBFO-S03-01, dated May 2003, is used to estimate the channel dimensions for this project. These equations can be found in Appendix 4 of the Hydrology Panel report. Two cross-sections are needed: between Points C and D and between Points D and E. The sections are located close to the midpoint of each reach. Their locations are shown in Figure A5-8. GISHydro is used to delineate the contributing area to the mid-point of each reach. The drainage area contributing to Reach C-D is 0.9 square miles and to Reach D-E is 7.4 square miles. Figure 13 of the USFWS report is used to estimate channel characteristics which are reported in the attached spreadsheet. The travel times used to calculate the Channel Flow time of concentration are also reported in calculation sheet A5-A.

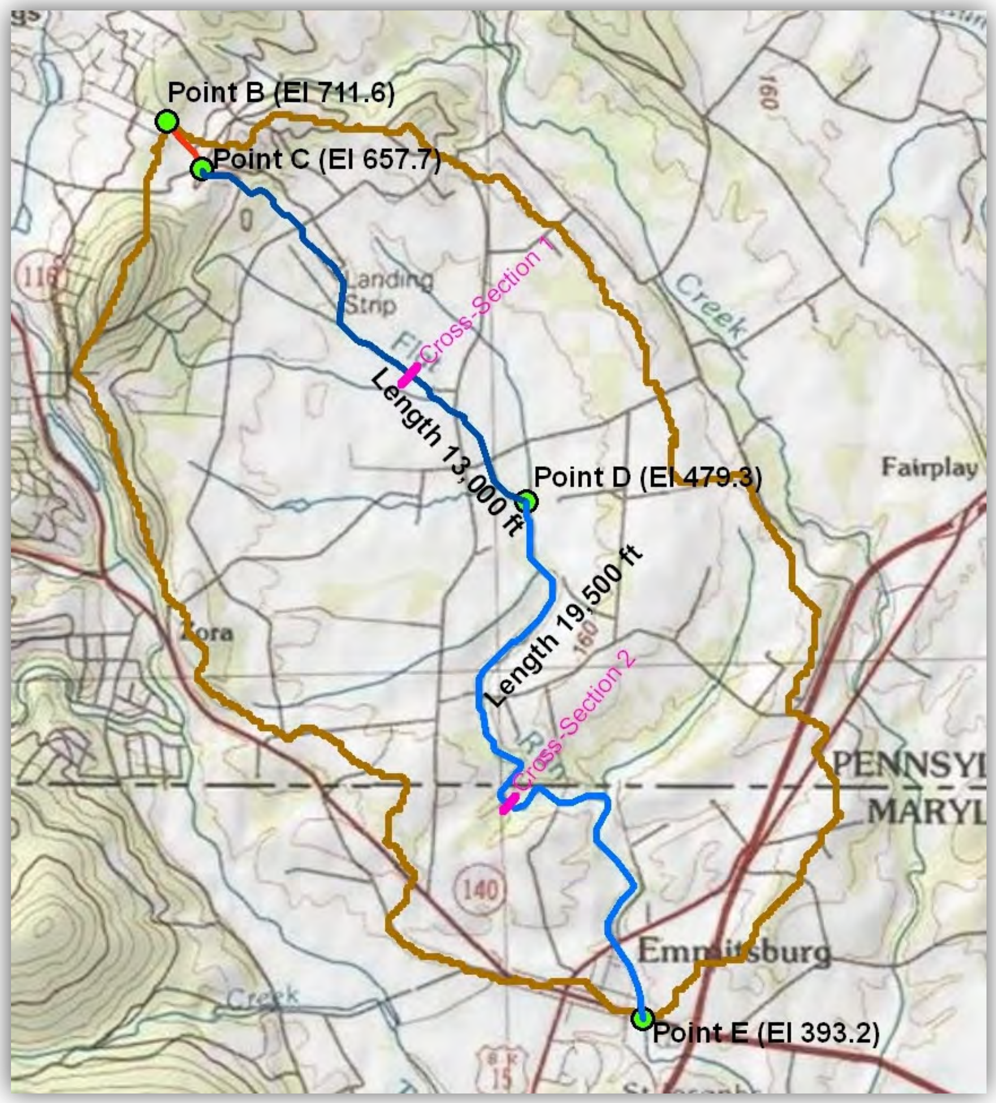


Figure A5-8
Location of channel reaches for determining the Channel Flow travel times

One of the values requiring the most judgment in estimating the time of concentration (Tc) is the selection of the Manning's Roughness Coefficient. In this calculation, the roughness coefficient must account for all losses including minor losses such as changes in channel cross-section, local obstructions, and gradient changes. The value should be larger than what may be appropriate for a straight uniform channel. The recommended 0.05 base value is used for this example.

Using these base values, a total time of concentration was computed as 3.65 hours. This value should be compared to the Appendix 6 regression equation estimate and the SCS Lag Equation estimate which are reported in the basin statistics file. These values are 5.95 hours and 4.85 hours, respectively. These values should reflect the upper limit of the time of concentration. The Appendix 6 regression equation is based mostly on smaller storm events and the SCS Lag equation relies on rural watershed data and tend to overestimate the time of concentration for more developed land use watersheds. It is expected that the segmented Tc calculation will yield lower Tc values.

Time of Concentration

Project: MD 140 over Flat Run	By: J. Knaub	Date: 12/22/10
Location: Frederick County	Checked:	Date:

Notes: Undivided Watershed. This calculation is performed for and example project to be included in the SHA/MDE hydrology panel report. No calibration has been performed.

Sheet Flow (Aplicable to T_c only)					
Segment I.D.	A-B				
1. Surface Description (See Table)...................	Grass/Light Woods				
2. Manning's roughness coefficient (See Table)...	0.3				
3. Flow Length, L (total L≤ 300 ft)...... ....ft	100				
4. Two-year 24-hr rainfall, P_2..............................in	3.15				
5.Land Slope, s...ft/ft	0.017	=(713.3-711.6)/100			
$T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\,s^{0.4}}$ compute T_t.......hr	0.31				0.31
Shallow Concentrated Flow					
Segment I.D.	B-C				
7. Surface Description (Paved or Unpaved)...........	Unpaved				
8. Flow Length, L..ft	1590				
9.Land Slope, s...ft/ft	0.034	=(711.6-657.7)/1590			
10.Average Velocity, V................................ft/s	3.0				
11. $T_t = \frac{L}{3600\ V}$ compute T_t.......hr	0.15				0.15
Channel Flow					
Segment I.D.	C-D	D-E			
12. Watershed area, a ....mi^2	0.9	7.4			
13. Cross sectional flow area, ...ft^2	12.2	59.1			
14. Width, ...ft	13.2	33.5			
15. Depth, ...ft	0.9	1.8			
13. Wetted perimeter, P_w..................................ft	15.1	37.0			
14. Hydraulic radius,r ...... $r = \frac{a}{P_w}$.....ft	0.8	1.6			
15.Channel Slope, s.....................................ft/ft	0.0137	0.0044			
16. Manning's roughness coefficient (See Table)...	0.05	0.05			
17 $v = \frac{1.49\,r^{\frac{2}{3}}\,s^{\frac{1}{2}}}{n}$ Compute V.......ft/s	3.0	2.7			
18. Flow Length, L..ft	13000	19500			
19. $T_t = \frac{L}{3600\ V}$ compute T_t.......hr	1.19	2.01			3.20
20. Watershed or Subarea T_c or T_t (add T_t in steps 6, 11, and 19)...........................Hr=					**3.65**

Calculation Sheet A5-A
Time of concentration computation using TR-55 method

Step 4 – Determine the Rainfall Data and Build the WinTR-20 Model

Use GISHydro to develop the rainfall depths for various storm duration and frequencies. The values for this example are shown in Figure A5-9. GISHydro develops these data from the NOAA Atlas 14 publication.

GISHydro builds the WinTR-20 model for each storm event. The time of concentration value is the one derived by the TR-55 velocity method shown above.

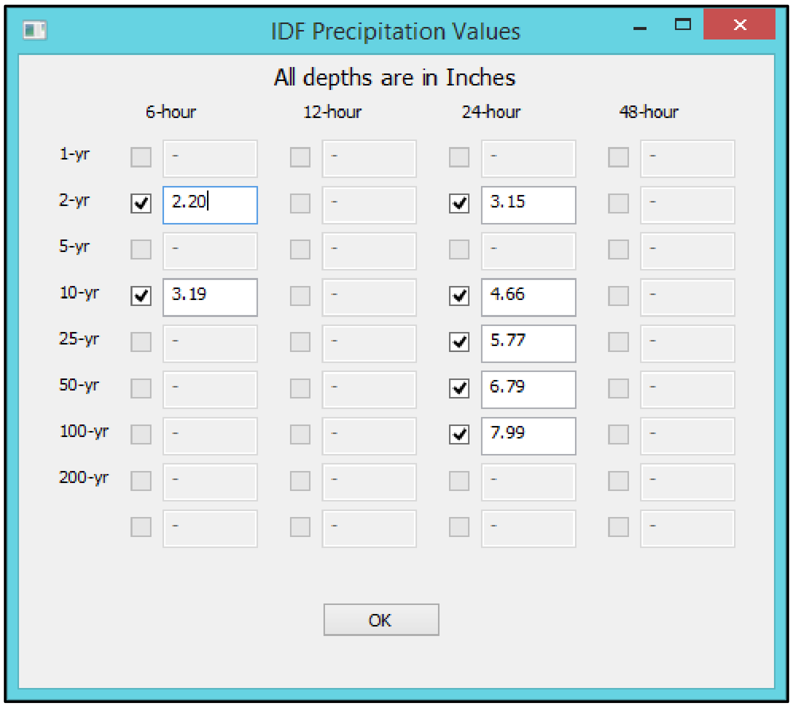


Figure A5-9
Rainfall depths from NOAA Atlas 14

Step 5 – Run WinTR-20

Open WinTR-20 and open the input file created by GISHydro. Save the file and run to compute discharges.

Step 6 – Evaluate Results

Table A5-1 and Figure A5-10 show the results of the WinTR-20 model as compared to the regression equation and upper 67-percent Tasker Limit.

Table A5-1
Regional Regression Values for Each Return Period for Uncalibrated Model

Return Period	Discharge			
	Fixed Region Eqn	Upper 67% Tasker Limit	WINTR20 (24 hr, Tc=3.65 hrs)	WINTR20 (6 hr, Tc=3.65)
2	844	1220	1334	803
10	2380	3190	2850	1891
25	3640	4830	4013	- - -
50	4860	6530	5089	- - -
100	6350	8690	6323	- - -

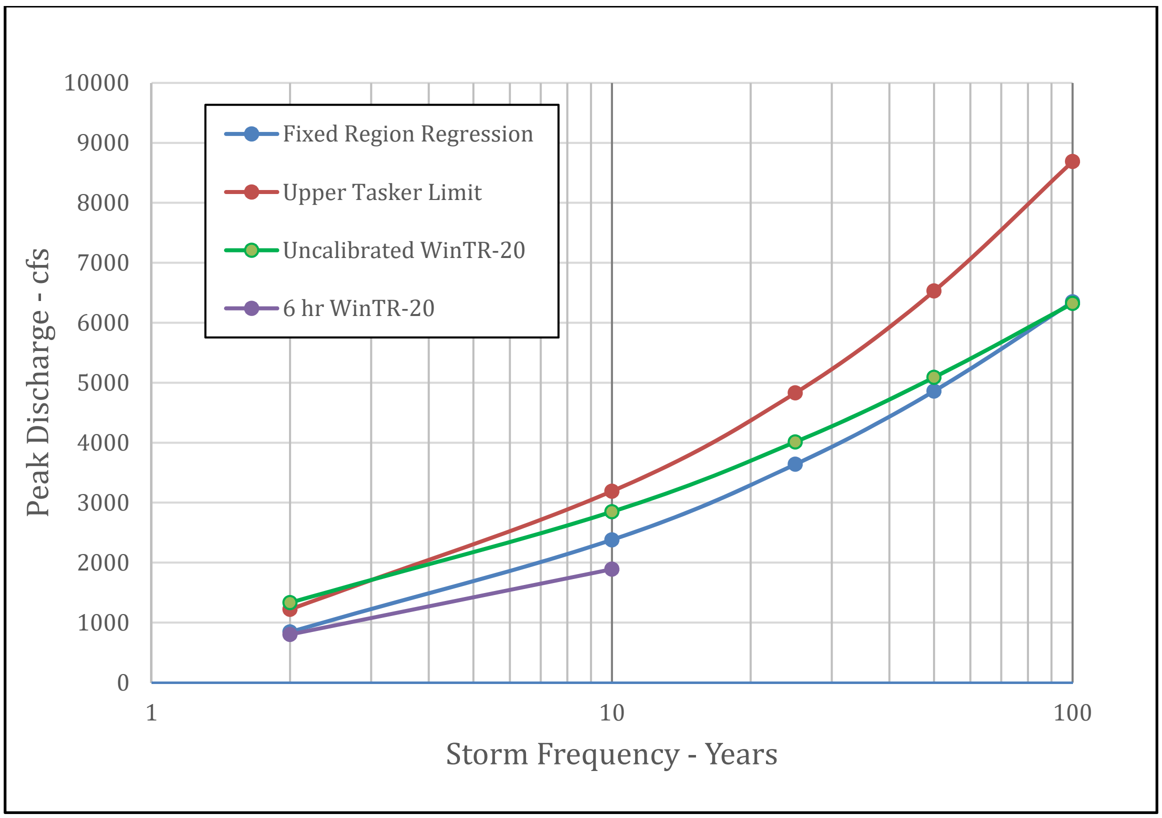


Figure A5-10
Comparison of Win TR-20 flood discharges without calibration to the Fixed Region regression estimates

Step 7 – Make Calibration Adjustments

Table A5-1 and Figure A5-10 show that the uncalibrated WinTR-20 results are nearly within the Fixed Region regression equation and the upper 67-percent limits. The 2 year, 24 hour storm model value is slightly higher than the Tasker Limit while the 100 year, 24 hour storm value is slightly lower than the regression value. It is also noted that the 6 hour storm analysis results are also below the regression values.

A calibration strategy could be to reduce the Time of Concentration (Tc) slightly to bring the 2 year, 6 hour value up to the regression value. This Tc calibration would also raise the 100 year, 24 hour value and bring it within the targeted limits. The Tc adjustment, however, should not cause the 10 year, 24 hour model value to exceed the Upper Tasker Limit.

Using *Chapter 4, Table 4.2* as a guide, we decided to reduce the channel n value by 20% from 0.050 to 0.040. Our justification, based on field reconnaissance, is that the channel in this lower reach is of regular geometry and has few obstructions. Since the n value used for Tc computations represents the bank full flow, the higher flood plain n values can be disregarded.

The resulting modified Tc computations are shown in Calculation Sheet A5-B below.

Time of Concentration				
Project: MD 140 over Flat Run		By: J. Knaub	Date: 12/22/10	
Location: Frederick County		Checked:	Date:	

Notes: Undivided Watershed. This calculation is performed for and example project to be included in the SHA/MDE hydrology panel report. **Reduce channel n by 20% (0.050 to 0.040)**

Sheet Flow (Aplicable to T_c only)

Segment I.D.	A-B		
1. Surface Description (See Table)........................	Grass/Light Woods		
2. Manning's roughness coefficient (See Table)...	0.3		
3. Flow Length, L (total L≤ 300 ft)...... ....ft	100		
4. Two-year 24-hr rainfall, P_2 ....in	3.15		
5.Land Slope, s...ft/ft	0.017	=(713.3-711.6)/100	
$T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{\;0.5}\,s^{0.4}}$ compute T_t......hr	0.31		0.31

Shallow Concentrated Flow

Segment I.D.	B-C		
7. Surface Description (Paved or Unpaved)...........	Unpaved		
8. Flow Length, L...ft	1590		
9.Land Slope, s...ft/ft	0.034	=(711.6-657.7)/1590	
10.Average Velocity, V.....................................ft/s	3.0		
11. $T_t = \frac{L}{3600\ V}$ compute T_t.......hr	0.15		0.15

Channel Flow

Segment I.D.	C-D	D-E		
12. Watershed area, a ....mi²	0.9	7.4		
13. Cross sectional flow area, ...ft²	12.2	59.1		
14. Width, ...ft	13.2	33.5		
15. Depth, ...ft	0.9	1.8		
13. Wetted perimeter, P_w...ft	15.1	37.0		
14. Hydraulic radius,r ...... $r = \frac{a}{P_w}$ ....ft	0.8	1.6		
15.Channel Slope, s.......................................ft/ft	0.0137	0.0044		
16. Manning's roughness coefficient (See Table)...	0.05	0.040		
17 $v = \frac{1.49\,r^{\frac{2}{3}}\,s^{\frac{1}{2}}}{n}$ Compute V.......ft/s	3.0	3.39		
18. Flow Length, L...ft	13000	19500		
19. $T_t = \frac{L}{3600\ V}$ compute T_t......hr	1.19	1.60		2.79
20. Watershed or Subarea T_c or T_t (add T_t in steps 6, 11, and 19)..............................Hr=				3.25

Calculation Sheet A5-B
Time of concentration computation adjustments

The WinTR-20 model is re-run using the Tc value of 3.25 hrs and the resulting discharges are shown in Table A5-2. The Tc adjustment has brought the 100 year value within the Tasker Limits. The 2-year value for the 24 hour storm is still too high. However, the 6 hour storm better reflects the event that would more likely cause the 2-year frequency peak flow so we can consider this value as the more appropriate. Experience in stream geomorphology has also shown that WinTR-20 for storms of 2 years and less tend to overestimate the flows for bank full conditions. Since the NRCS methods are based on rainfall-runoff data derived mostly from storms greater than the 2-year event. Therefore, in this case we have more confidence in selecting the 6 hour storm value of 872 cfs for the 2-year storm.

Table A5-2
Calibrated Model Results

Return Period	Discharge			
	Fixed Region Eqn	Upper 67% Tasker Limit	WINTR20 (24 hr, Tc=3.25 hrs)	WINTR20 (6 hr, Tc=3.25 hrs)
2	844	1220	1456	872
10	2380	3190	3113	2064
25	3640	4830	4370	- - -
50	4860	6530	5555	- - -
100	6350	8690	6882	- - -

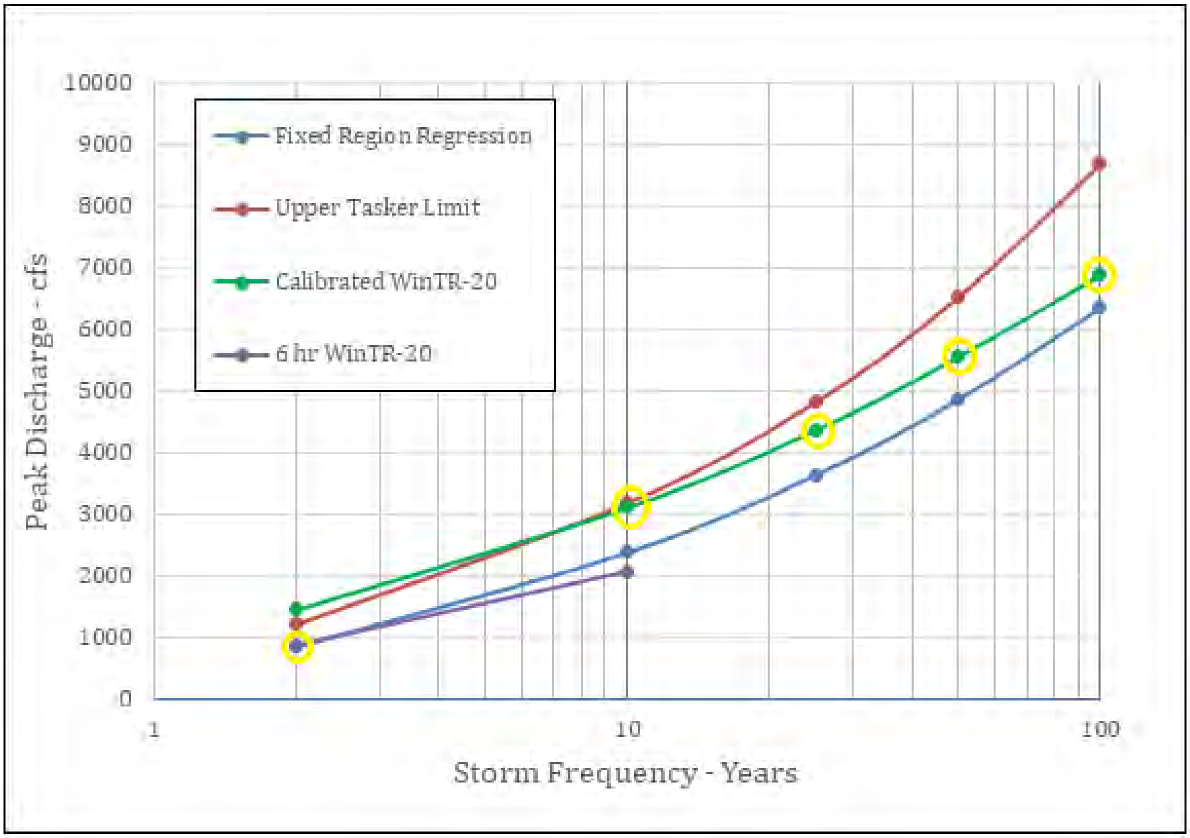


Figure A5-11
Final calibrated Win TR-20 flood discharges as compared to the Fixed Region regression estimates

Table A5-3
Final Calibrated Win TR-20 model (6- and 24-hour storm durations)

Return Period	Discharge		
	Fixed Region Eqn	Upper 67% Tasker Limit	WINTR20 (Tc=3.25 hrs)
2	844	1220	**872 (6 hr)**
10	2380	3190	**3113 (24 hr)**
25	3640	4830	**4370 (24 hr)**
50	4860	6530	**5555 (24 hr)**
100	6350	8690	**6882 (24 hr)**

Step 8 – Create the Ultimate Condition WinTR-20 Model

The final step to complete a study is to modify the calibrated existing condition WinTR-20 model to reflect the ultimate development condition. This example illustrates how to calibrate an existing condition WinTR-20 model. Chapter 4 of the Hydrology Panel report provides instructions on how to perform this final step. In this example we would simply use the Ultimate Development RCN value in the model to generate the 50- and 100-year design flows for the bridge.

It should be noted however, that for smaller watersheds under 1 or 2 square miles, the Tc path may also be modified by urban development. In that case, the Tc should be reevaluated and may be reduced by ultimate improvements to the flow paths.

GISHydro Watershed Land Use

GISHydro Release Version Date: July 17, 2019
Project Name: Example 1
Analysis Date: June 08, 2020

Landuse and Soil Distributions for:

Distribution of Landuse by Soil Group

Land Use	Acres on Indicated Soil Group A-Soil	B-Soil	C-Soil	D-Soil
Open Urban Land	0	230.85	265.1	62.27
Low Density Residential	0	73.17	163.24	14.9
Medium Density Residential	0	12.23	12.9	1.33
High Density Residential	0	0.44	0.67	0
Barren Land	0	2.22	3.11	4.67
Deciduous Forest	0	737.02	767.04	259.09
Evergreen Forest	0	23.57	40.92	8.67
Mixed Forest	0	36.25	52.26	10.9
Grassland	0	9.34	17.12	0
Pasture	0	254.86	649.61	114.98
Cropland	0	638.94	1902.14	444.35
Wetlands	0	37.36	47.81	21.35
Wetlands	0	8.45	5.12	1.56
Total Area:	0	2064.7	3927.04	944.07

Distribution of Land Use and Curve Numbers Used

Land Use	Acres	Percent	A	B	C	D
Open Urban Land	558.22	8.05	39	61	74	80
Low Density Residential	251.31	3.62	54	70	80	85
Medium Density Residential	26.46	0.38	61	75	83	87
High Density Residential	1.11	0.02	77	85	90	92
Barren Land	10.0	0.14	77	86	91	94
Deciduous Forest	1763.15	25.42	30	55	70	77
Evergreen Forest	73.16	1.05	30	55	70	77
Mixed Forest	99.41	1.43	30	55	70	77
Grassland	26.46	0.38	39	61	74	80
Pasture	1019.45	14.7	39	61	74	80
Cropland	2985.43	43.04	67	78	85	89
Wetlands	106.52	1.54	100	100	100	100
Wetlands	15.13	0.22	100	100	100	100

GISHydro Basin Statistics

```
GISHydro Release Version Date:    July 17, 2019
Project Name:                     Example 1
Analysis Date:                    June 08, 2020
Data Selected:
     DEM Coverage:                NED DEM 201805
     Land Use Coverage:           NLCD 2001
     Soil Coverage:               SSURGO 2000's
     Hydrologic Condition:        Good
     Impose NHD stream Locations: Yes
     Outlet Easting:              372636 m (MD Stateplane, NAD 1983)
     Outlet Northing:             226074 m (MD Stateplane, NAD 1983)
Findings:
     Outlet Location:             Blue Ridge and Great Valley
     Outlet State:                Maryland
     Drainage Area                10.87 square miles
       -Blue Ridge and Great Valley 100.00 percent of area
```

```
Channel Slope:              29.9 feet/mile (0.00567 feet/feet)
Land Slope:                 0.054 feet/feet
Urban Area (percent):       4.0
Impervious Area (percent):  2.0

           ******************************************************
               Watershed is within 1km of underlying limestone
               geology.  You should consider sensitivity
               of discharges to percent limestone calculated.
           ******************************************************

Time of Concentration:      5.95 hours [W.O. Thomas, Jr. Equation]
Time of Concentration:      4.85 hours  [From SCS Lag Equation * 1.67]
Longest Flow Path:          7.25 miles
Basin Relief:               139.25 feet
Average CN:                 75.4
Forest Cover (percent):     27.9
Storage (percent):          1.8
Limestone (percent):        0.0
Selected Soils Data Statistics Percent:
   A Soils:                 0.0
   B Soils:                 29.7
   C Soils:                 56.4
   D Soils:                 13.6
SSURGO Soils Data Statistics Percent (used in Regression Equations):
   A Soils:                 0.0
   B Soils:                 29.7
   C Soils:                 56.4
   D Soils:                 13.6
2-Year,24-hour Prec.:       3.15 inches
Mean Annual Prec.:          44.70 inches
```

Thomas Discharges and Prediction (Tasker) Limits

```
GISHydro Release Version Date: July 17, 2019
Project Name:                  Example 1
Analysis Date:                 June 08, 2020
Thomas Version:                2016

Geographic Province(s):
       -Blue Ridge and Great Valley 100.00 percent of area

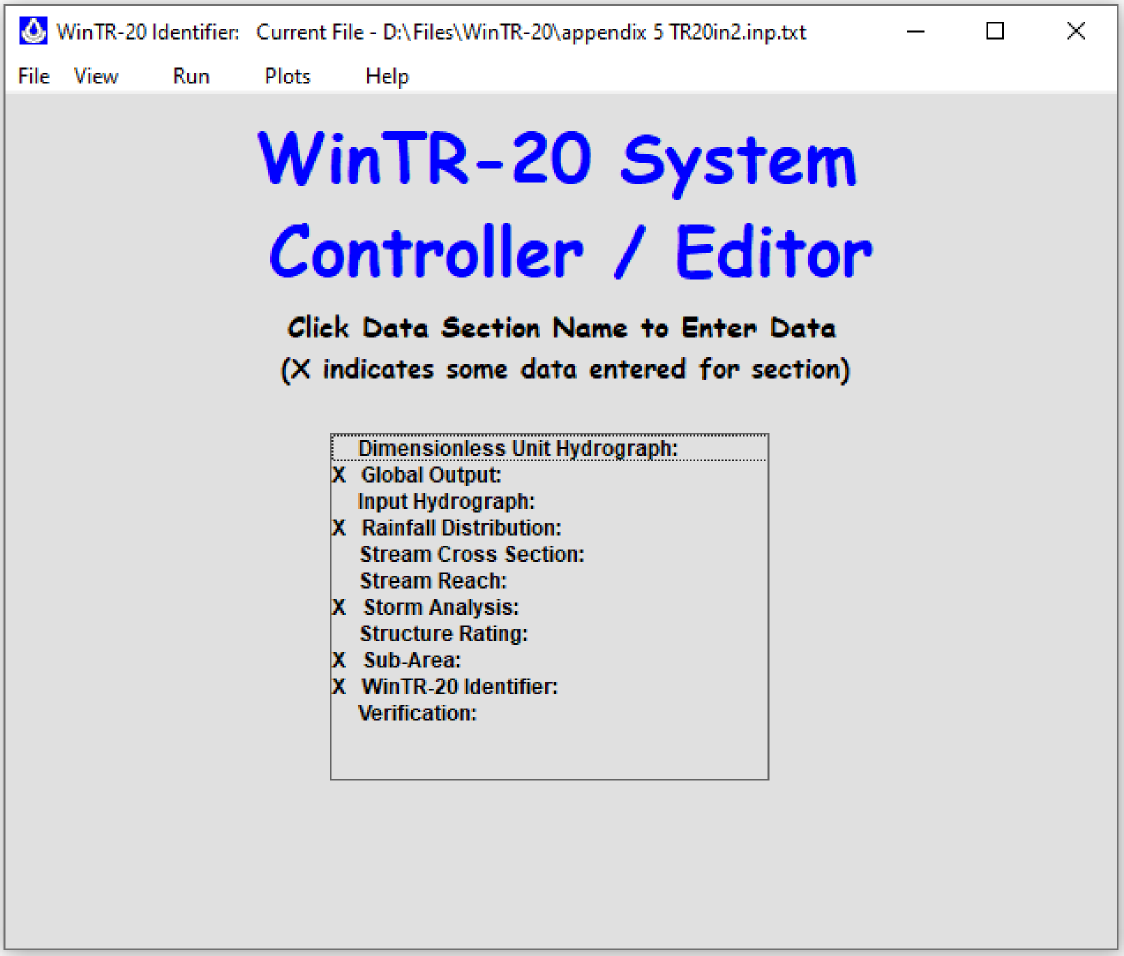
Q(1.25):   467 cfs
Q(1.50):   618 cfs
Q(2):      844 cfs
Q(5):      1630 cfs
Q(10):     2380 cfs
Q(25):     3640 cfs
Q(50):     4860 cfs
Q(100):    6350 cfs
Q(200):    8660 cfs
Q(500):    12000 cfs
```

Area Weighted Prediction Intervals (from Tasker)

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
1.25	351	622	305	716	232	941	202	1080
1.5	474	806	416	919	322	1190	284	1350
2	660	1080	585	1220	462	1540	411	1730
5	1320	2020	1190	2240	976	2730	883	3020
10	1960	2900	1780	3190	1480	3850	1340	4230
25	3010	4400	2740	4830	2290	5790	2090	6330
50	3990	5920	3620	6530	3000	7880	2730	8660
100	5150	7840	4650	8690	3800	10600	3440	11700
200	6860	10900	6110	12300	4890	15400	4370	17200
500	9120	15700	7980	18000	6160	23300	5410	26500

NOAA Atlas 14 Precipitation Data

```
GISHydro Release Version Date:    July 17, 2019
Project Name:                     Example 1
Data Selected:
     Outlet Easting:              372636.431849 m (MD Stateplane, NAD 1983)
     Outlet Northing:             226074.627438 m (MD Stateplane, NAD 1983)
Precipitation Frequency-Duration Depths:
     2-year, 06-hour: 2.2 inches
     2-year, 24-hour: 3.15 inches
     10-year, 06-hour: 3.19 inches
     10-year, 24-hour: 4.66 inches
     25-year, 24-hour: 5.77 inches
     50-year, 24-hour: 6.79 inches
     100-year, 24-hour: 7.99 inches
```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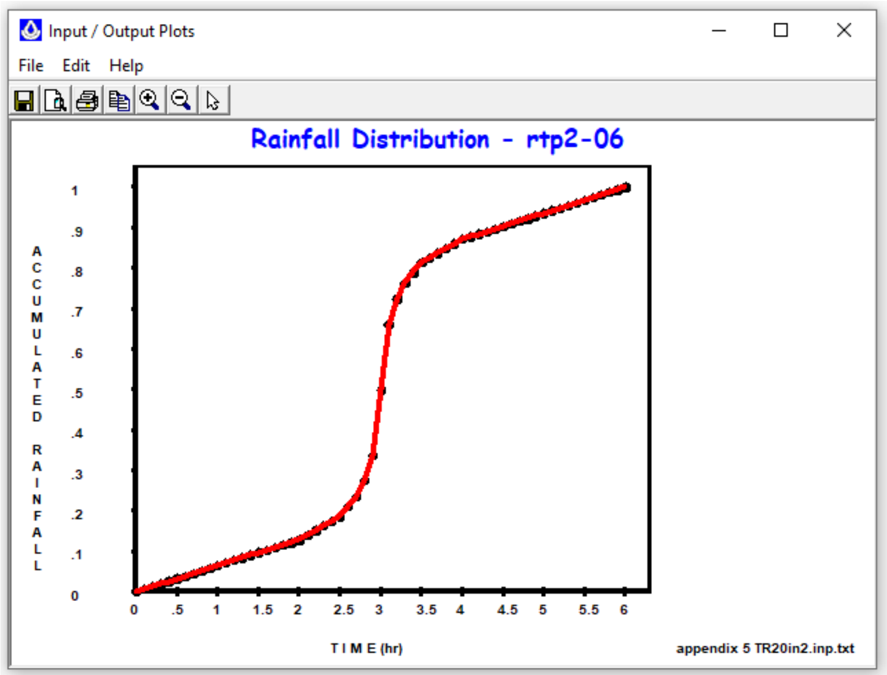
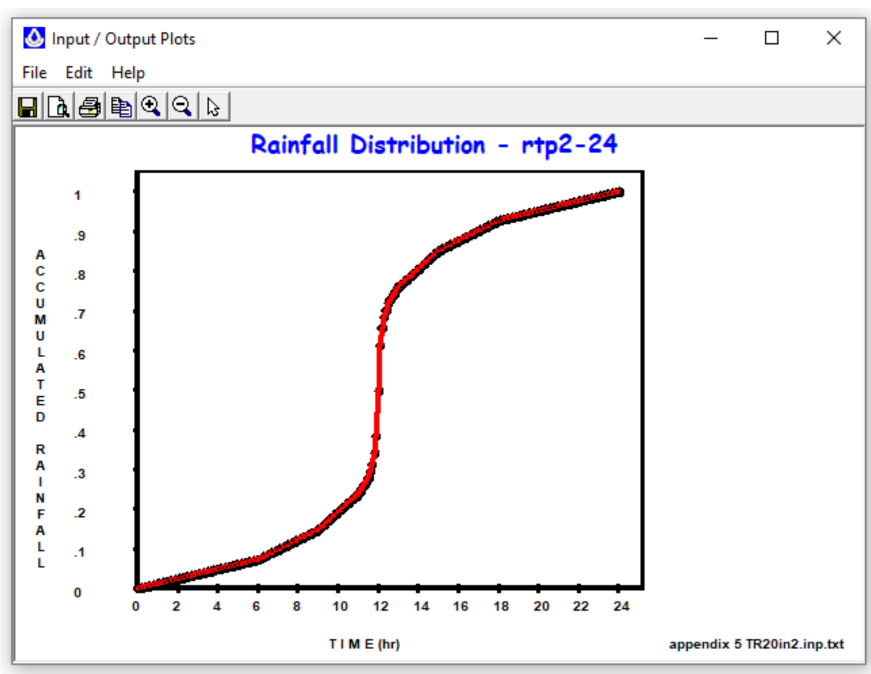
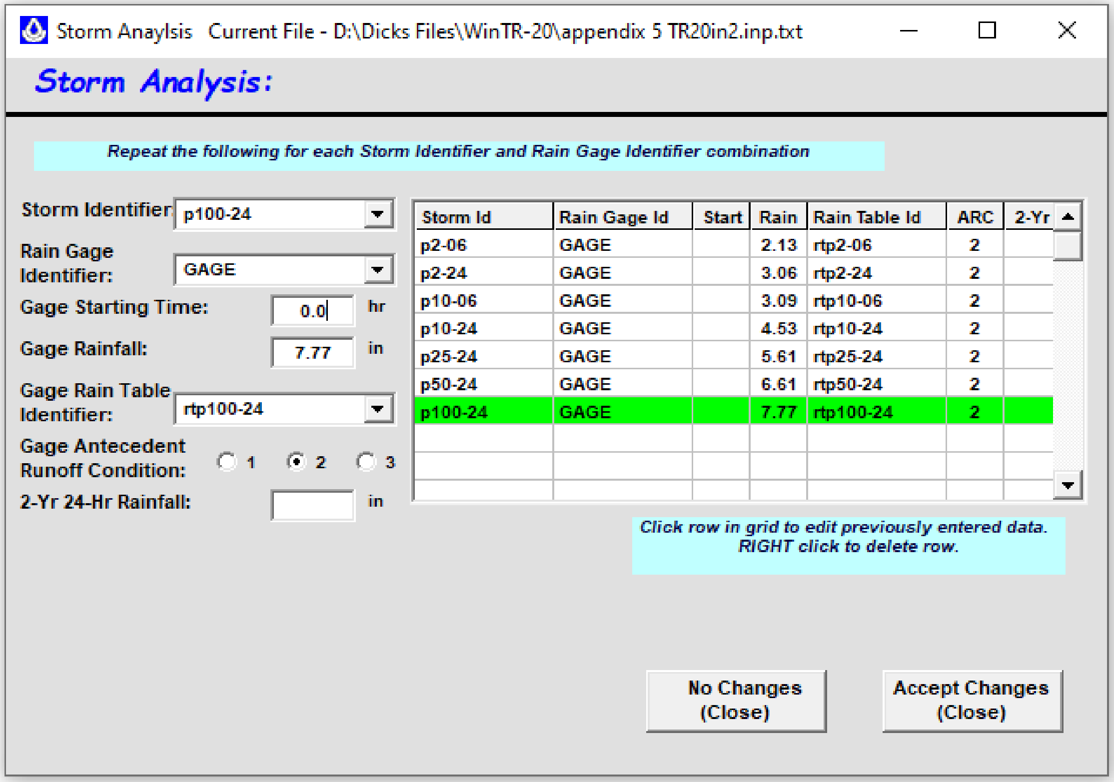
View
Run
Plots
Help
WinTR-20 System
Controller / Editor
Click Data Section Name to Enter Data
(X indicates some data entered for section)
Dimensionless Unit Hydrograph:
X Global Output:
Input Hydrograph:
X Rainfall Distribution:
Stream Cross Section:
Stream Reach:
X Storm Analysis:
Structure Rating:
X Sub-Area:
X WinTR-20 Identifier:
Verification:

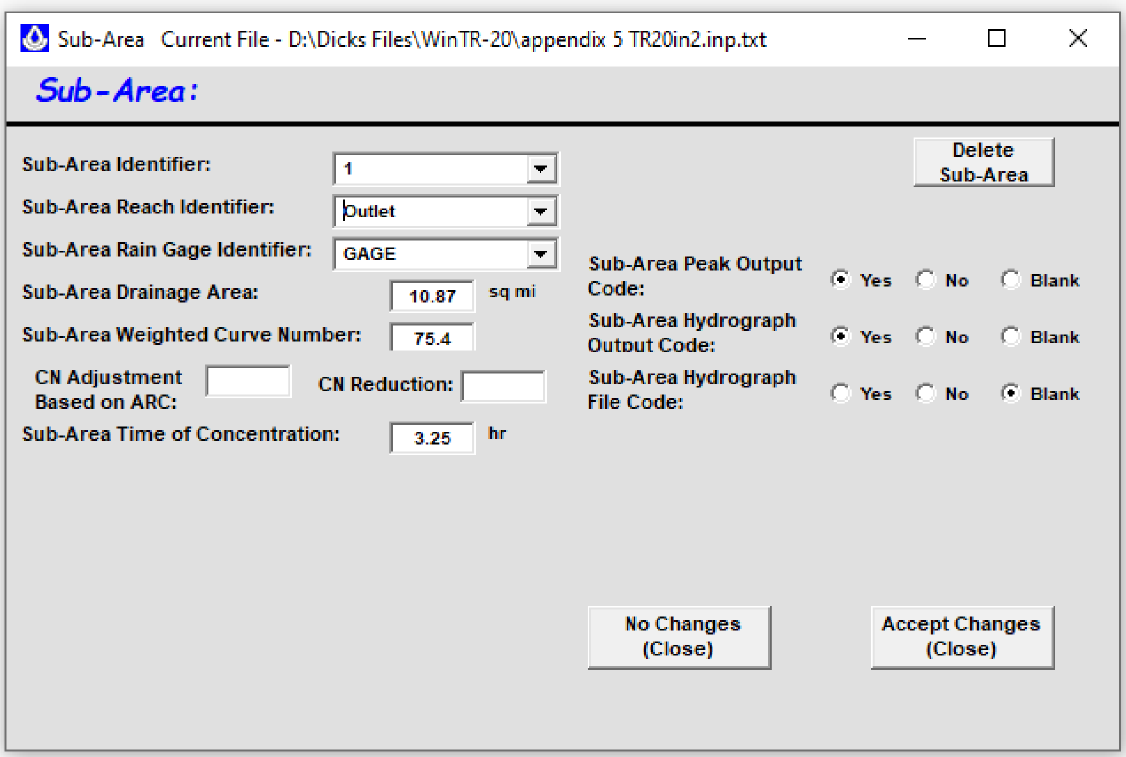
Example 6-hour Rainfall Distribution



Example 24-hour Rainfall Distribution




Storm Anaylsis Current File - D:\Dicks Files\WinTR-20\appendix 5 TR20in2.inp.txt
Storm Analysis:
Repeat the following for each Storm Identifier and Rain Gage Identifier combination
Storm Identifier p100-24
Rain Gage Identifier: GAGE
Gage Starting Time: 0.0 hr
Gage Rainfall: 7.77 in
Gage Rain Table Identifier: rtp100-24
Gage Antecedent Runoff Condition: 1 2 3
2-Yr 24-Hr Rainfall: in
Storm Id Rain Gage Id Start Rain Rain Table Id ARC 2-Yr
p2-06 GAGE 2.13 rtp2-06 2
p2-24 GAGE 3.06 rtp2-24 2
p10-06 GAGE 3.09 rtp10-06 2
p10-24 GAGE 4.53 rtp10-24 2
p25-24 GAGE 5.61 rtp25-24 2
p50-24 GAGE 6.61 rtp50-24 2
p100-24 GAGE 7.77 rtp100-24 2
Click row in grid to edit previously entered data.
RIGHT click to delete row.
No Changes (Close)
Accept Changes (Close)


Sub-Area Current File - D:\Dicks Files\WinTR-20\appendix 5 TR20in2.inp.txt
Sub-Area:
Delete Sub-Area
Sub-Area Identifier: 1
Sub-Area Reach Identifier: Outlet
Sub-Area Rain Gage Identifier: GAGE
Sub-Area Drainage Area: 10.87 sq mi
Sub-Area Weighted Curve Number: 75.4
CN Adjustment Based on ARC:
CN Reduction:
Sub-Area Time of Concentration: 3.25 hr
Sub-Area Peak Output Code: Yes No Blank
Sub-Area Hydrograph Output Code: Yes No Blank
Sub-Area Hydrograph File Code: Yes No Blank
No Changes (Close)
Accept Changes (Close)

WinTR-20 Output

```
WinTR-20 Printed Page File      Beginning of Input Data List      D:\Files\WinTR-20\appendix
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WinTR-20: version 3.20                  0         0         1.0       0
GISHydroNXT - [folder: E://temp/20200608_102815_My_Project/WinTR20]

SUB-AREA:
1         Outlet    GAGE      10.87     75.4      3.25      YY
STORM ANALYSIS:
p2-06         GAGE                2.13      rtp2-06   2
p2-24         GAGE                3.06      rtp2-24   2                               p10-
06        GAGE                3.09      rtp10-06  2                               p10-24
GAGE                4.53      rtp10-24  2                               p25-24        GAGE
5.61      rtp25-24  2                             p50-24        GAGE                6.61
rtp50-24  2                             p100-24       GAGE                7.77
rtp100-24 2

RAINFALL DISTRIBUTION:
6 Hour Storm       0.1
0.0000    0.0065    0.0130    0.0195    0.0260                                    0.0325
0.0390    0.0455    0.0520    0.0585                                    0.0650    0.0715
0.0780    0.0845    0.0910                                    0.0975    0.1037    0.1099
0.1160    0.1222                                    0.1284    0.1400    0.1515    0.1631
0.1747                                    0.1863    0.2111    0.2360    0.2756    0.3381
0.5000    0.6619    0.7244    0.7640    0.7889                                    0.8137
0.8253    0.8369    0.8485    0.8600                                    0.8716    0.8778
0.8840    0.8901    0.8963                                    0.9025    0.9090    0.9155
0.9220    0.9285                                    0.9350    0.9415    0.9480    0.9545
0.9610                                    0.9675    0.9740    0.9805    0.9870    0.9935
1.0000

24 Hour Storm      0.1
0.0000    0.0012    0.0024    0.0037    0.0049                                    0.0061
0.0073    0.0085    0.0097    0.0110                                    0.0122    0.0134
0.0146    0.0158    0.0170                                    0.0183    0.0195    0.0207
0.0219    0.0231                                    0.0244    0.0256    0.0268    0.0280
0.0292                                    0.0304    0.0317    0.0329    0.0341    0.0353
0.0365    0.0377    0.0390    0.0402    0.0414                                    0.0426
0.0438    0.0450    0.0463    0.0475                                    0.0487    0.0499
0.0511    0.0524    0.0536                                    0.0548    0.0560    0.0572
0.0584    0.0597                                    0.0609    0.0621    0.0633    0.0645
0.0657                                    0.0670    0.0682    0.0694    0.0706    0.0718
0.0731    0.0756    0.0781    0.0806    0.0831                                    0.0856
0.0882    0.0907    0.0932    0.0957                                    0.0982    0.1007
0.1033    0.1058    0.1083                                    0.1108    0.1133    0.1158
0.1184    0.1209                                    0.1234    0.1259    0.1284    0.1309
0.1335                                    0.1360    0.1385    0.1410    0.1435    0.1460
0.1486    0.1531    0.1577    0.1623    0.1668                                    0.1714
0.1760    0.1805    0.1851    0.1897                                    0.1943    0.1988
0.2034    0.2080    0.2125                                    0.2171    0.2214    0.2258
0.2301    0.2344                                    0.2388    0.2469    0.2551    0.2632
0.2714                                    0.2795    0.2970    0.3144    0.3423    0.3862
0.5000    0.6138    0.6577    0.6856    0.7030                                    0.7205
0.7286    0.7368    0.7449    0.7531                                    0.7612    0.7656
0.7699    0.7742    0.7786                                    0.7829    0.7875    0.7920
0.7966    0.8012                                    0.8057    0.8103    0.8149    0.8195
0.8240
0.8286    0.8332    0.8377    0.8423    0.8469                                    0.8514
0.8540    0.8565    0.8590    0.8615                                    0.8640    0.8665
0.8691    0.8716    0.8741                                    0.8766    0.8791    0.8816
0.8842    0.8867                                    0.8892    0.8917    0.8942    0.8967
0.8993                                    0.9018    0.9043    0.9068    0.9093    0.9118
0.9144    0.9169    0.9194    0.9219    0.9244                                    0.9269
0.9282    0.9294    0.9306    0.9318                                    0.9330    0.9343
0.9355    0.9367    0.9379                                    0.9391    0.9403    0.9416
```

```
0.9428    0.9440                                  0.9452    0.9464    0.9476    0.9489
0.9501                                  0.9513    0.9525    0.9537    0.9550    0.9562
0.9574    0.9586    0.9598    0.9610    0.9623                                  0.9635
0.9647    0.9659    0.9671    0.9683                                  0.9696    0.9708
0.9720    0.9732    0.9744                                  0.9756    0.9769    0.9781
0.9793    0.9805                                  0.9817    0.9830    0.9842    0.9854
0.9866                                  0.9878    0.9890    0.9903    0.9915    0.9927
0.9939    0.9951    0.9963    0.9976    0.9988                                  1.0000


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WinTR-20 Version 3.20                                          06/30/2020   6:53

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Area or      Drainage                ----------- Peak Flow by Storm -----------
Reach         Area                  p2-06      p2-24     p10-06     p10-24     p25-24
Identifier   (sq mi)                (cfs)      (cfs)      (cfs)      (cfs)      (cfs)

1             10.870                871.7     1456.3     2064.0     3113.2     4370.0      OUTLET
10.870                 871.7     1456.3     2064.0     3113.2     4370.0

Area or      Drainage                ----------- Peak Flow by Storm -----------
Reach         Area                  p50-24    p100-24
Identifier   (sq mi)                (cfs)      (cfs)      (cfs)      (cfs)      (cfs)

1             10.870               5555.0     6882.3                                         OUTLET
10.870                5555.0     6882.3
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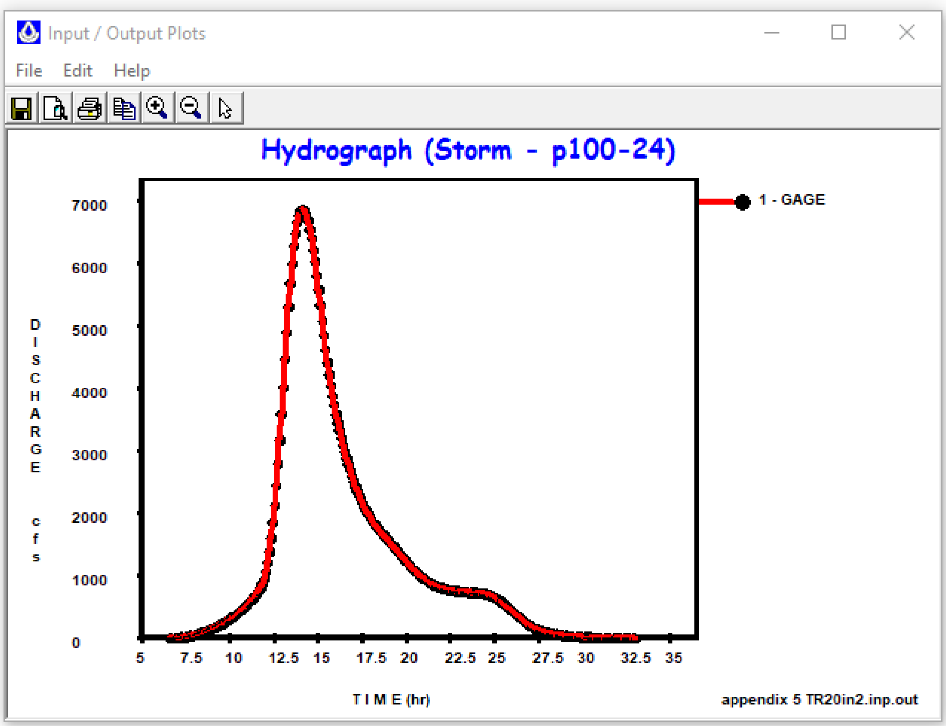

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APPENDIX 6
REGRESSION EQUATIONS FOR ESTIMATING THE TIME OF CONCENTRATION

REGRESSION EQUATION FOR ESTIMATING THE TIME OF CONCENTRATION

Time of concentration (T_c) can be defined from an observed rainfall hyetograph and the resulting discharge hydrograph. T_c is estimated as the time between the end of rainfall excess and the first inflection point on the recession of the runoff hydrograph. The T_c values were computed from rainfall-runoff data compiled by the Dillow (1998) as part of a flood hydrograph study for the Maryland State Highway Administration.

Dillow (1998) compiled data for 278 rainfall-runoff events at 81 gaging stations in Maryland. Not all of the 278 events were suitable in defining T_c for our study. For some rainfall-runoff events, it was not possible to detect an inflection point on the recession of the hydrograph. On average, about three events were used in determining the average T_c for a watershed. For three gaging stations, there were no rainfall-runoff events suitable for determining T_c. Therefore, data for 78 gaging stations are used in developing a regression equation for estimating T_c for ungaged watersheds. The average T_c values and watershed characteristics are given in Table A6.1 (at the end of this chapter).

Stepwise regression analysis was used to relate the average T_c value at 78 gaging stations to the watershed characteristics given in Table A6.1. The watershed characteristics used in this analysis were taken from Dillow (1998). Some of the watershed characteristics that are highly correlated with T_c are also highly correlated with each other. For example, drainage area has a correlation coefficient of 0.98 with channel length. Since these two variables are highly correlated, both variables are not significant in the regression analysis because they are essentially explaining the same variation in T_c. The regression equation based on channel length has a slightly lower standard error than the one with drainage area and so channel length is used in the final equation. Channel length also is a better predictor of travel time for a variety of watershed shapes.

Using Dillow's approach (1998), qualitative variables are used in the regression analysis to identify gaging stations in different hydrologic regions in Maryland. Dillow (1998) determined that there are three hydrologic regions for estimating flood hydrographs for Maryland streams: Appalachian Plateau, Piedmont and Coastal Plain. These same regions are assumed applicable in the T_c analysis and are shown in Figure A6.1. The qualitative-variable approach is superior to defining different regression equations for each geographic region because there are only 10 gaging stations in the Appalachian Plateau.

The qualitative variables AP and CP are used in the regression equation to account for variability in T_c not explained by the available explanatory variables. In Table A6.1, a CP value of 1 specifies the watershed is in the Coastal Plain Region, an AP value of 1 specifies the watershed is in the Appalachian Plateau and zero values for both CP and AP specify the watershed is in the Piedmont Region. The T_c values for watersheds in the Appalachian Plateau and Coastal Plains are larger than watersheds in the Piedmont for a given set of watershed characteristics (see Figure 4.2 in Chapter 4). The qualitative variables also account for regional differences in T_c related to watershed characteristics not available for analysis. Both AP and CP are highly significant in the regression analysis.

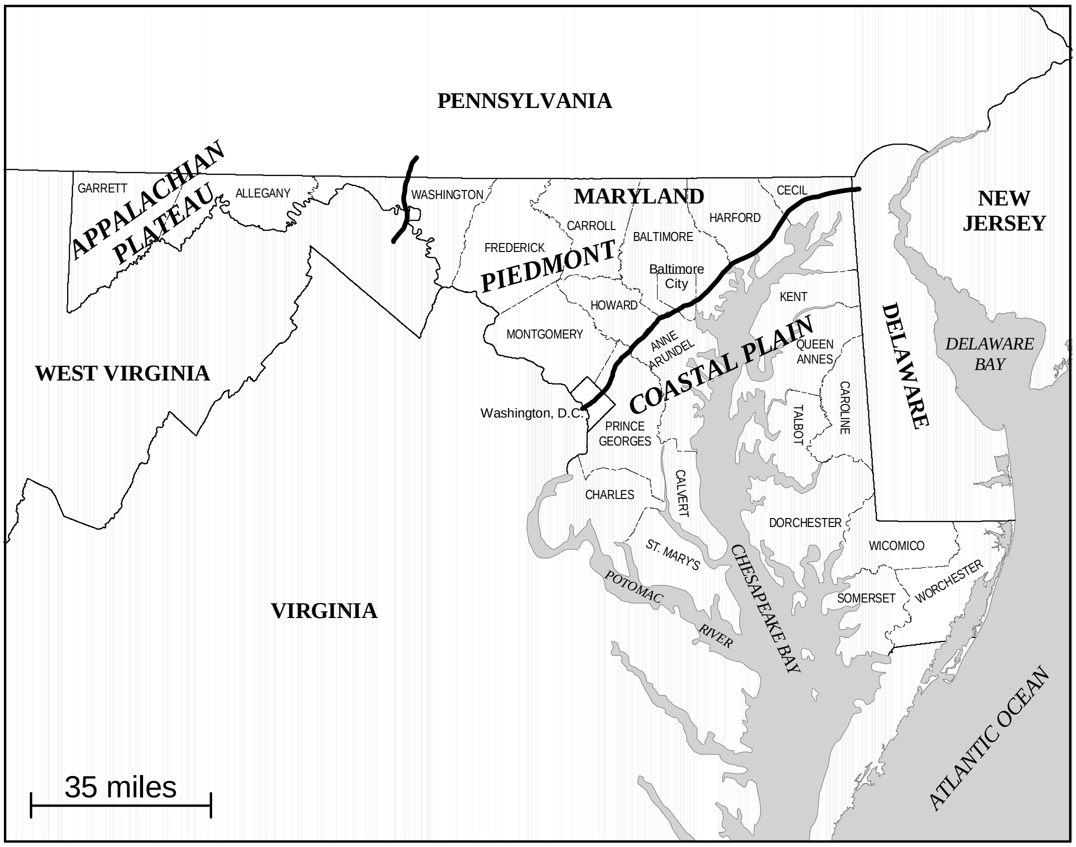

Figure A6-1: Hydrologic regions in Maryland used in developing a regression equation for estimating the time of concentration for ungaged watersheds

There is considerable variation in hydrology from the Coastal Plains of Maryland to the mountainous Appalachian Plateau. Therefore, several watershed characteristics are statistically significant in predicting T_c. In Equation A6-1, all explanatory variables are significant at the 5 percent level of significance. The coefficient of determination (R^2) is 0.888 percent implying the equation is explaining 88.8 percent of the variation in the observed value of T_c. The standard error of estimate is 30.0 percent.

$$T_c = 0.133\,(CL^{.475})\,(SL^{-.187})\,(101\text{-}FOR)^{-.144}\,(101\text{-}IA)^{.861}\,(ST+1)^{.154}\,(10^{.194AP})\,(10^{.366CP}) \qquad \text{(A6-1)}$$

where

T_c = time of concentration in hours,
CL = channel length in miles,

SL = channel slope in feet per mile,
FOR = forest cover in percentage of the watershed,
IA = impervious area in percentage of the watershed,
ST = lakes and ponds in percentage of the watershed,
AP = 1 if the watershed is in the Appalachian Plateau, 0 otherwise,
CP = 1 if the watershed is in the Coastal Plain, 0 otherwise,
AP and CP = 0 for watersheds in the Piedmont Region.

Equation A6-1 was computed by transforming the T_c values and watershed characteristics to logarithms and then fitting a linear regression model to the transformed data. This transformation is somewhat standard in hydrologic analyses since the logarithmic transformation tends to stabilize the variance of the residuals, normalize the distribution of the residuals about the regression equation and linearize the equation.

The percentages of forest cover (FOR), impervious area (IA) and storage (ST) can be zero for a given watershed. Therefore, it is necessary to add constants to these variables prior to the logarithmic transformation or to subtract these variables from a constant to avoid taking the logarithm of zero. For our analysis, subtracting the percentages from 101 provided more reasonable estimates of the regression coefficients and slightly reduced the standard error of the regression equation.

Equation A6-1 can be used to estimate T_c for rural and urban watersheds in Maryland. The percentage of impervious area (IA) is a measure of the urbanization or development in the watershed. In addition, urban watersheds would have a reduced amount of forest cover.

The T_c values in Table A6.1 are generally longer than computed by SCS (1986) procedures for a given watershed area. One possible hypothesis is that this is related to size of the flood events used to determine T_c. In general, the recurrence intervals of peak discharges were less than a 2-yr event. There were only about 30 events across the 78 gaging stations where the peak discharge of the runoff hydrograph was a 5-yr event or greater. An evaluation of the T_c values as a function of recurrence interval revealed that the T_c values did not vary with recurrence interval in any consistent pattern. In some instances, the larger flood events had smaller T_c values and at other stations the converse was true. Therefore, it is not conclusive that the use of larger flood events would result in smaller T_c values

A comparison was also made between estimates of T_c computed from Equation A6-1 and procedures in SCS (1986) based on travel time. The travel times shown in Table A6-2 were computed by MDOT SHA personnel at selected bridge sites as a combination of overland flow, shallow concentrated flow and channel flow (SCS, 1986). In Table A6-2, the sites in the Eastern and Western Coastal Plain Regions were identified but both regions are treated as coastal plain regions in applying Equation A6-1 (i.e., CP=1). The times of concentration in Table A6-2 are plotted versus drainage area in Figure A6-2.

Table A6-2: A comparison of time of concentration (T_c) estimated from Equation A6-1 based on watershed characteristics to T_c values based on travel time

Site or Location	Hydrologic Region	DA (mi^2)	Travel time Tc (hrs)	Regression Tc (hrs)
MD 165 over West Branch	Piedmont	6.66	3	3.4
MD 17 over Middle Creek	Piedmont	25.7	5.2	5.9
MD 7 over Mill Creek	Piedmont	5.01	4.3	4.4
US 1 over Little Gunpowder Falls	Piedmont	43.73	9	7.9
MD 109 over Little Monocacy River	Piedmont	3.16	1.5	2.7
MD 136 over Deer Creek	Piedmont	143.5	19.9	14.2
MD 97 over Meadow Branch	Piedmont	1.3	0.92	2
ICC over Upper Rock Creek	Piedmont	3.7	2.1	3
MD 25 over Trib to Blackrock Run	Piedmont	0.86	1.83	1.8
MD 25 over Jones Falls	Piedmont	25.2	3.14	4.92
MD 32 over Middle Patuxent River	Piedmont	14.28	3.24	4.2
MD 136 over Falling Branch	Piedmont	4.32	3.62	3.88
MD 140 over Branch of Cattail Creek	Piedmont	0.6	1.53	1.9
MD 140 over Gwynns Falls	Piedmont	9.38	2.8	3.1
MD 191 over Bulls Run	Piedmont	3.17	1.43	1.8
MD 222 over Rock Run	Piedmont	3.48	1.62	3.1
MD 273 over Little Northeast Creek	Piedmont	2	1.59	3
MD 478 over Branch of Potomac River	Piedmont	0.9	1.02	1.9
MD 496 over Big Pipe Creek	Piedmont	12.3	2.18	3.8
US 1 over Winters Run	Piedmont	34.6	6.1	6.8
MD 481 over Blockston Branch	Eastern CP	6.26	8.7	10.8
US 113 over Middle Branch	Eastern CP	3.24	7.2	9.1
US 113 over Church Branch	Eastern CP	6.05	10.6	11
US 113 over Carey Branch	Eastern CP	1.61	5.7	6
US 113 over Birch Branch	Eastern CP	6.64	7.6	11
US 50 in Queen Anne's County	Eastern CP	5.8	6.9	11.9
US 50 in Queen Anne's County	Eastern CP	2.5	4.4	8.4
MD 298 over Branch of Fairlee Lake	Eastern CP	0.5	3.1	5.3
MD 404 over Norwich Creek	Eastern CP	11.8	8.4	16
US 50 over Trib to Otter Pond Branch	Eastern CP	0.94	8.41	17.4
MD 5 over St. Mary's River	Western CP	26.1	6.71	16.6
MD 210 over Henson Creek	Western CP	21.4	7.8	10.9
MD 225 over Branch of Mattawoman	Western CP	80.3	20.47	28
MD 648 over Branch of Cattail Creek	Western CP	2.2	2.35	5.3
MD 47 over North Branch	A Plateau	3.77	2	4.8
MD 47 over North Branch	A Plateau	3.3	1.4	4
MD 47 over North Branch	A Plateau	12.26	2.79	6.1
US 50 over Trib to Youghiogheny River	A Plateau	0.63	2.52	3.1
US 219 over Trib to Youghiogheny River	A Plateau	0.4	1.8	4.2

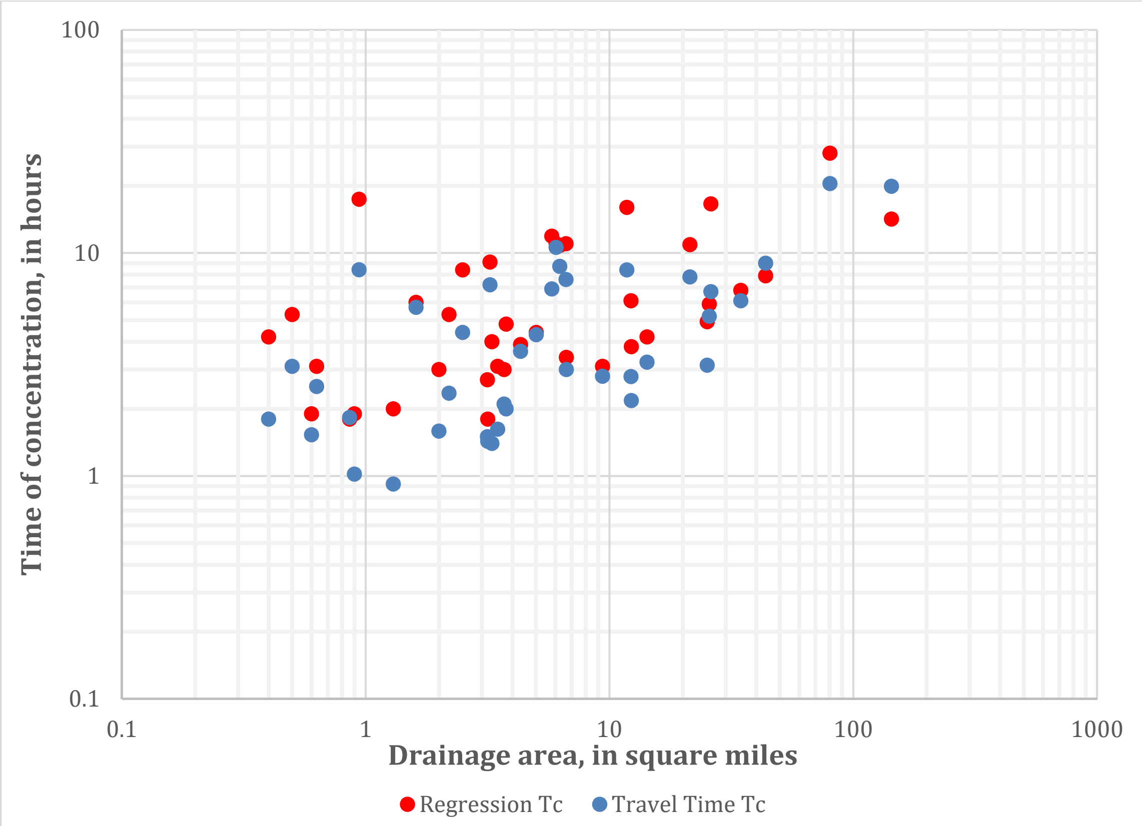

Figure A6-2: Comparison of time of concentration based on Equation A6-1 and the travel time method

There is close agreement for T_c estimates for several of the sites shown in Table A6-2 and Figure A6-2, especially for the larger watersheds. When there are significant differences, the values based are travel times (also known as the segmental approach) are less than those from the regression equation. Based on this limited comparison, it appears that Equation A6-1 can be used to determine realistic bounds on T_c estimated by the travel time or segmental approach.

There are 39 observations in Table A6-2 and Figure A6-2, of which 20 observations are for the Piedmont Region (includes the Blue Ridge Region). The times of concentrations for the Piedmont Region are plotted in Figure A6-3 with trend lines in order to clarify the differences in travel time and regression estimates of T_c.

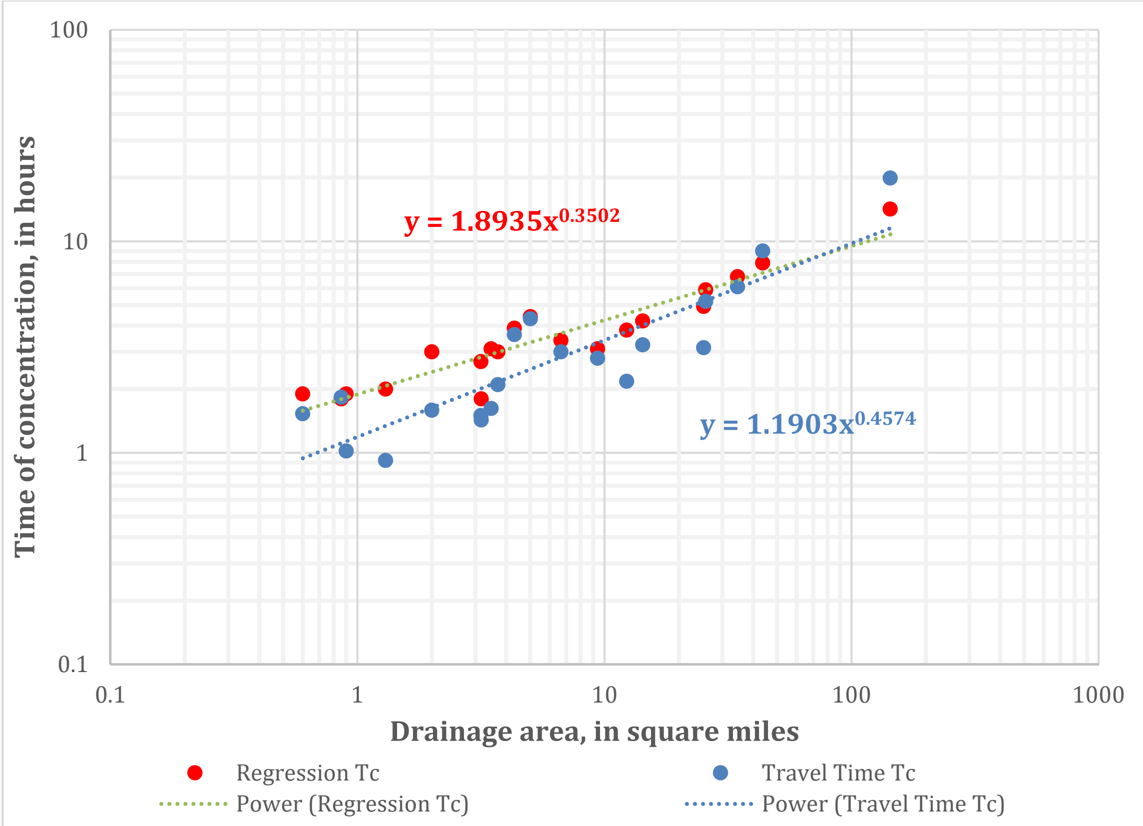

Figure A6-3: Comparison of time of concentration based on Equation A6-1 and the travel time method for 20 bridge sites in the Piedmont Region

The trend lines in Figure A6-3 illustrate that Equation A6-1 gives higher estimates of Tc for a given drainage area size than the travel time method. Just based on the data in Table A6-2, Equation A6-1 tends to overpredict the travel time T_c estimates by a larger amount than in the Piedmont Region.

Any regression equation, such as Equation A6-1, should only be used at ungaged watersheds that have watershed characteristics within the range of those used to develop the equation. The upper and lower limits for the watershed characteristics are given in Table A6-3 for each hydrologic region to define the applicability of Equation A6-1. Therefore, Equation A6-1 should not be used for watersheds having characteristics outside the limits of those shown in Table A6-3.

Table A6-3: Upper and lower limits for watershed characteristics for the time of concentration regression equation for each hydrologic region

Region	Variable	Upper limit	Lower limit
Appalachian Plateau	Drainage area (mi^2)	295	1.6
Appalachian Plateau	Channel length (mi)	40.8	2.1
Appalachian Plateau	Channel slope (ft/mi)	195	6.1
Appalachian Plateau	Storage (%)	3.2	0.0
Appalachian Plateau	Forest cover (%)	89	54
Appalachian Plateau	Impervious area (%)	1.25	0.0
Piedmont	Drainage area (mi^2)	494	2.1
Piedmont	Channel length (mi)	70	2.2
Piedmont	Channel slope (ft/mi)	336	11
Piedmont	Storage (%)	1.16	0.0
Piedmont	Forest cover (%)	92	2.0
Piedmont	Impervious area (%)	41	0.0
Coastal Plain	Drainage area (mi^2)	113	2.0
Coastal Plain	Channel length (mi)	18.3	2.0
Coastal Plain	Channel slope (ft/mi)	41.8	1.5
Coastal Plain	Storage (%)	26.0	0.0
Coastal Plain	Forest cover (%)	79	5.0
Coastal Plain	Impervious area (%)	35	0.0

In summary, Equation A6-1 is based on estimates of T_c computed from rainfall-runoff events at 78 gaging stations in Maryland. The computed values of T_c tend to be larger than similar estimates based on SCS (1986) procedures. However, Equation A6-1 can be used to evaluate the reasonableness of T_c estimates from SCS (1986) procedures.

Table A6-1: Watershed characteristics and times of concentration for rural and urban watersheds used in developing the regression equations.

STANO is the station number
DA is the drainage area in square miles
SL is the channel slope in feet per mile
CL is channel length in miles
SIN is the channel sinuosity determined by dividing channel length by basin length
BL is the basin length in miles
ST is the percentage area of the drainage area covered by lakes, ponds and swamps
SH is the basin shape defined as channel length squared divided by drainage area
FOR is forest cover in percentage of the drainage area
IA is impervious area expressed as percentage of the drainage area
BDF is the basin development factor
LT is the lagtime in hours
AP = 1 if the watershed is in the Appalachian Plateau, CP = 1 if the watershed is in the Coastal Plains, CP and AP = 0 implies the watershed is in the Piedmont Region
T_c is the time of concentration in hours

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	T_c
01594930	8.23	26.4	4.4	1.14	3.86	0.000	1.81	86	0.00	0	7.50	1	0	6.38
01594934	1.55	161.9	2.1	1.07	1.95	0.000	2.45	82	0.00	0	6.43	1	0	4.00
01594936	1.91	130.9	2.7	1.16	2.33	0.000	2.84	87	0.00	0	6.62	1	0	6.00
01594950	2.30	194.6	2.7	1.24	2.18	0.000	2.07	89	0.00	0	6.74	1	0	5.00
01595000	73.0	30.5	16.5	1.30	12.70	0.186	2.21	78	0.49	0	12.27	1	0	11.50
01596500	49.1	65.1	19.0	1.41	13.44	0.066	3.68	80	0.06	0	13.97	1	0	9.75
03075500	134.	6.09	19.3	1.59	12.12	0.493	1.10	54	0.88	0	22.57	1	0	23.50
03076500	295.	22.2	40.8	1.45	28.11	3.180	2.68	66	0.24	0	25.10	1	0	29.25
03076600	48.9	65.6	15.3	1.89	8.11	0.000	1.35	62	1.25	0	16.47	1	0	11.25
03078000	62.5	28.2	19.5	1.61	12.13	1.005	2.35	75	0.66	0	16.88	1	0	19.58
01614500	494.	11.2	69.5	2.44	28.45	0.101	1.64	37	1.43	0	25.42	0	0	26.33
01617800	18.9	23.8	9.4	1.08	8.69	0.000	4.00	2	2.32	0	15.53	0	0	.
01619500	281.	10.8	49.9	1.55	32.26	0.123	3.70	30	2.67	0	24.66	0	0	27.12
01637500	66.9	47.5	23.3	1.50	15.50	0.000	3.59	38	1.01	0	8.98	0	0	7.62
01639000	173.	18.9	30.8	1.92	16.05	0.114	1.49	20	0.69	0	15.91	0	0	17.25
01639375	41.3	75.4	12.2	1.40	8.70	0.207	1.83	70	0.87	0	3.47	0	0	5.00

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	T_c
01639500	102.	13.5	26.9	1.43	18.75	0.000	3.45	14	0.13	0	11.80	0	0	8.50
01640965	2.14	336.4	2.2	1.12	1.96	0.000	1.80	92	0.00	0	1.78	0	0	1.88
01641000	18.4	145.2	9.7	1.57	6.18	0.373	2.08	80	1.93	1	5.11	0	0	5.44
01483700	31.9	4.66	12.3	1.38	8.89	11.927	2.48	21	4.46	2	27.41	0	1	32.92
01484000	13.6	6.26	5.9	1.01	5.87	0.626	2.53	34	0.33	0	21.04	0	1	20.85
01484500	5.24	4.87	4.4	1.19	3.70	0.000	2.61	39	3.24	0	12.82	0	1	14.88
01484548	13.6	4.39	7.9	1.22	6.48	26.055	3.09	33	1.13	0	24.28	0	1	31.75
01485000	60.5	1.49	14.6	1.18	12.42	18.396	2.55	25	0.08	0	28.58	0	1	37.00
01485500	44.9	3.56	12.2	1.11	10.98	1.326	2.69	79	0.30	0	37.21	0	1	41.75
01487000	75.4	3.23	13.7	1.20	11.44	0.000	1.74	40	0.85	0	20.80	0	1	23.25
01488500	44.8	2.65	11.7	1.17	10.00	0.000	2.23	39	0.14	0	12.99	0	1	15.08
01489000	8.50	7.65	5.3	1.46	3.64	0.000	1.87	24	0.00	0	5.78	0	1	8.44
01491000	113.	3.01	18.3	1.36	13.41	6.910	1.59	38	0.66	0	31.57	0	1	36.88
01493000	19.7	6.06	9.7	1.09	8.89	8.777	3.54	20	0.35	0	26.10	0	1	22.25
01493500	12.7	9.15	5.9	1.10	5.38	0.199	2.28	5	0.25	0	13.35	0	1	16.38
01483200	3.85	15.8	3.5	1.04	3.37	1.298	2.95	45	0.38	0	7.37	0	1	11.67
01484100	2.83	7.12	2.5	1.07	2.33	0.000	1.92	43	0.00	0	14.54	0	1	15.50
01486000	4.80	5.47	4.1	.	.	0.000	.	57	0.00	0	.	0	1	10.50
01590500	6.92	19.8	4.7	1.14	4.12	0.000	2.45	65	1.87	0	10.90	0	1	11.94
01594526	89.7	8.2	16.1	1.18	13.60	0.037	2.06	30	7.84	4	23.16	0	1	36.38
01594670	9.38	16.9	5.2	1.30	3.99	0.000	1.70	70	3.85	0	9.17	0	1	12.33
01653600	39.5	16.1	14.4	1.64	8.79	0.176	1.96	38	8.25	2	17.29	0	1	29.05
01660920	79.9	10.6	16.6	1.15	14.48	5.051	2.62	56	3.60	0	26.17	0	1	31.25
01661050	18.5	12.4	7.2	1.22	5.92	0.000	1.89	56	3.09	0	14.26	0	1	16.38
01594710	3.26	41.8	2.9	1.08	2.68	0.000	2.20	52	9.24	0	3.86	0	1	5.08
01661500	24.0	12.9	8.0	1.28	6.25	0.000	1.63	78	2.46	0	15.78	0	1	13.75
01583600	20.9	52.0	8.2	1.43	5.72	0.309	1.57	29	18.6	4	5.63	0	0	4.25
01585100	7.61	48.2	6.0	1.12	5.38	0.000	3.80	28	27.5	7	2.11	0	0	2.75
01585200	2.13	72.7	2.2	1.12	1.97	0.000	1.82	7	33.0	8	1.02	0	0	1.38

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	T_c
01585300	4.46	54.7	4.6	1.25	3.68	0.558	3.04	28	23.6	6	2.06	0	0	2.38
01585400	1.97	27.1	2.0	1.22	1.64	0.000	1.37	24	35.1	2	2.33	0	1	3.25
01589100	2.47	87.1	3.2	1.22	2.62	0.000	2.78	19	37.0	4	1.67	0	0	2.17
01589300	32.5	21.0	13.7	1.37	9.99	0.000	3.07	31	18.6	4	3.95	0	0	3.38
01589330	5.52	52.1	3.2	1.12	2.86	0.000	1.48	4	40.8	12	2.26	0	0	2.83
01589500	4.97	24.8	4.4	1.17	3.75	0.000	2.83	44	21.9	3	8.19	0	1	.
01589512	8.24	23.5	5.9	1.17	5.04	1.092	3.08	31	30.8	3	6.72	0	1	7.75
01593500	38.0	15.8	15.5	1.40	11.04	0.623	3.21	23	18.7	6	7.48	0	0	10.58
01645200	3.70	67.4	2.7	1.16	2.33	0.000	1.47	14	28.0	6	1.91	0	0	2.75
01649500	72.8	27.2	15.3	1.33	11.54	0.192	1.83	33	22.0	5	8.85	0	0	8.50
01651000	49.4	19.7	19.1	1.36	14.05	0.047	4.00	19	22.0	6	6.45	0	0	6.58
01495000	52.6	17.9	22.2	1.41	15.80	0.053	4.75	14	1.92	0	9.87	0	0	8.88
01496200	9.03	29.0	5.9	1.36	4.33	0.000	2.08	4	0.00	0	4.38	0	0	5.81
01580000	94.4	17.7	27.3	1.52	17.92	0.039	3.40	27	0.42	0	7.29	0	0	7.50
01581657	4.16	74.2	3.7	1.19	3.12	0.000	2.34	33	5.25	0	4.08	0	0	3.83
01581658	5.22	56.1	4.8	1.28	3.74	0.000	2.68	31	4.78	0	4.38	0	0	4.92
01581700	34.8	30.0	15.8	1.60	9.89	0.000	2.81	21	2.37	2	4.68	0	0	3.50
01582000	52.9	33.8	15.0	1.35	11.14	0.015	2.35	32	0.91	0	6.84	0	0	6.62
01583100	12.3	50.9	7.8	1.08	7.25	0.092	4.27	26	0.29	0	5.77	0	0	4.50
01583500	59.8	24.5	15.9	1.40	11.36	0.064	2.16	22	0.16	0	8.20	0	0	8.08
01584050	9.40	70.0	4.8	1.11	4.32	0.000	1.99	13	1.00	0	3.05	0	0	3.00
01585105	2.65	65.2	3.6	1.14	3.16	0.000	3.77	16	5.22	0	3.86	0	0	4.00
01585500	3.29	56.0	3.5	1.11	3.14	1.165	3.00	21	0.45	0	3.08	0	0	3.12
01586000	56.6	28.5	14.6	1.38	10.61	0.069	1.99	19	1.77	0	8.56	0	0	9.75
01586210	14.0	44.0	8.1	1.38	5.86	0.000	2.45	19	1.77	0	4.39	0	0	4.00
01586610	28.0	30.9	10.0	1.47	6.81	0.000	1.66	20	0.38	0	5.97	0	0	4.58
01589440	25.2	38.2	9.5	1.37	6.95	0.000	1.92	34	9.92	2	5.29	0	0	6.92
01591000	34.8	28.2	12.2	1.22	10.02	0.000	2.89	21	0.21	0	6.51	0	0	7.12
01591400	22.9	28.0	8.7	1.35	6.44	0.097	1.81	16	1.52	0	6.16	0	0	6.83

STANO	DA	SL	CL	SIN	BL	ST	SH	FOR	IA	BDF	LT	AP	CP	T_c
01591700	27.0	26.5	10.9	1.28	8.52	0.141	2.69	19	2.08	0	5.28	0	0	6.83
01593710	48.4	17.8	14.7	1.28	11.45	0.000	2.71	24	2.16	0	5.99	0	0	8.25
01594000	98.4	13.6	23.5	1.33	17.62	0.134	3.16	26	6.52	4	10.83	0	0	9.88
01641510	0.40	817.8	0.9	1.09	0.83	0.000	1.72	100	0.00	0	4.26	0	0	.
01643495	0.15	1000.	0.5	1.13	0.44	0.000	1.29	100	0.00	0	1.26	0	0	1.75
01643500	62.8	23.8	15.6	1.43	10.89	0.000	1.89	23	1.19	0	7.30	0	0	8.35
01645000	101.	14.0	21.2	1.56	13.61	0.120	1.83	25	3.15	4	10.88	0	0	4.31

APPENDIX 7
PARTIAL DURATION RAINFALL FREQUENCY DATA 6, 12, AND 24-HOUR TEMPORAL DISTRIBUTION

Development of the 24-hour storm distribution from NOAA Atlas 14 Data

Unique storm distributions are recommended for all locations and return periods when using NOAA Atlas 14 data.

The WinTR-20 will import a partial duration text file downloaded from the NOAA Atlas 14 web site and develop storm distributions for each return period from 1-year to 500-years. Even though the 1000-year return period is included in the data, the WinTR-20 is not programmed to accept it. The complete partial duration data for each location may be downloaded from the NOAA Atlas 14 web site, http://hdsc.nws.noaa.gov/ .

The user of WinTR-20 has the choice to use the original NOAA Atlas 14 data or smoothed data to develop the 24-hour storm distribution. In developing the rainfall-frequency data, NOAA treated each duration independently. In some cases, this causes irregularities in rainfall intensity between durations, which then creates irregularities in 24-hour storm distribution and resulting flood hydrograph. The smoothed storm distribution provides a more reasonable trend in rainfall intensities for the various durations and is recommended for use in the WinTR-20 analysis. A description of the smoothing process that is incorporated in WinTR-20 follows.

For example, for a location in Howard County, Maryland, the 100-year 2-hour rainfall is 3.86 inches, the 100-year 3-hour rainfall is 4.20, and the 100-year 6-hour rainfall is 5.39. Between 2 and 3 hours the rainfall intensity is 0.34 inches per hour ((4.20 – 3.86)/ 1). Between 3 and 6 hours the rainfall intensity is 0.4 inches per hour ((5.39 – 4.20) / 3). The data shows the intensity actually increasing as the duration increases. As the duration increases, rainfall intensity should decrease. The smoothing algorithm in the WinTR-20 will smooth data from 5-minutes to 1-hour and from 1-hour to 24-hours while keeping the 1-hour rainfall and 24-hour rainfall unchanged. In the Howard County example, the smoothed values are 4.01 inches for the 100-year 2-hour, 4.69 inches for the 100-year 3-hour, and 5.83 inches for the 100-year 6-hour rainfall. This will produce intensities of 0.68 inches per hour between 2 and 3 hours and 0.38 inches per hour between 3 and 6 hours. The complete smoothing table for the 100-year data follows.

Table A7-1: NOAA Atlas 14 data and smoothed data for location in Howard County, MD

Duration	5-min	10-min	15-min	30-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
Original rainfall Inches	0.72	1.14	1.44	2.21	3.04	3.86	4.20	5.39	7.00	8.47
Intensity In/hr	8.64	5.04	3.6	3.08	1.66	0.82	0.34	0.4	0.27	0.12
Smooth rainfall Inches	0.69	1.14	1.48	2.16	3.04	4.01	4.69	5.83	7.09	8.47
Intensity In/hr	8.28	5.37	4.17	2.7	1.75	0.97	0.68	0.38	0.21	0.12
Rainfall Difference	-0.03	0.0	0.04	-0.05	0.0	0.15	0.49	0.44	0.09	0.0

The durations from 5-minutes to 60-minutes are relatively smooth (small difference between original and smoothed rainfall values). The 3-hour and 6-hour rainfall values are increased to provide a smooth relationship of intensity and duration (when plotted on a log-log scale).

This section of Appendix 7 discusses in detail how the WinTR-20 generates 24-hour storm distributions based on NOAA Atlas 14 data (5-minutes through 24-hour duration). A spreadsheet was developed which automates the steps. This spread sheet will provide similar (though not exact) results when compared to the WinTR-20 program. The reason the results are not exact is that Fortran and Excel operate with different numbers of significant digits so rounding of numbers is a concern.

The procedure will be described using an example from a location in Howard County Maryland. The 100-year 24-hour storm distribution will be developed using the smoothed rainfall frequency data. The NOAA Atlas 14 data and the ratio of rainfall at each duration to the 24-hour rainfall are in the following table.

Table A7-2: NOAA 14 data and ratios for durations at a location in Howard County, MD

	5-min	10-min	15-min	30-min	60-min	2-hr	3-hr	6-hr	12-hr	24-hr
Rainfall inches	0.69	1.14	1.48	2.16	3.04	4.01	4.69	5.83	7.09	8.47
Ratio to 24-hour	0.081	0.135	0.175	0.255	0.359	0.473	0.554	0.688	0.837	1.000

A symmetrical nested preliminary distribution is developed based on the ratios from 10-minutes to 24-hours. The mid-point of the preliminary distribution is 50% of the cumulative rainfall at 12.0 hours. It is symmetrical about 12 hours and places each duration 50% before 12 hours and 50% after 12 hours. For example, the 60-minute duration rainfall ratio is 0.3589. At 11.5 hours, one-half of 0.3589 is subtracted from 0.5 to calculate the cumulative ratio at 11.5 hours of 0.32054.

The preliminary distribution from 0.0 to 12.0 hours is shown in the following table.

Table A7-3: Preliminary rainfall distribution from 1 hour to 12 hours

Time-hours	**0.0**	**6**	**9**	**10.5**	**11**	**11.5**	**11.75**	**11.875**	**11.9167**	**12.0**
Cum Ratio	0.0	0.08146	0.15584	0.22314	0.26328	0.32054	0.41623	0.4327	0.45927	0.5

Once this preliminary distribution is developed, the next step is to develop the distribution ratios at a time interval of 0.1 hour. The general concept is to interpolate the ratios between the points in the above table at an interval of 0.1 hour. The ratios at 6, 9, 10.5, 11.0 and 11.5 are preserved in the final distribution. Ratios for times of 0.1 to 11.7 hours are based on slightly curved line segments between the ratios at the points in the table above. The slight curvature insures a gradual increase of rainfall intensity from 0.0 to 11.7 hours. Values for 11.8 and 11.9 hours are linearly interpolated between ratios at 11.75, 11.875, and 11.9167 hours. After the distribution from 0.0 to 12.0 hours is developed the ratios from 12.1 to 24 are calculated by subtracting the ratio of the opposite value from 1.0. For example, the ratio at 12.1 equals 1.0 minus the ratio at 11.9 hours. The ratio at 12.2 hours is equal to 1.0 minus the ratio at 11.8. This continues all the way to the ends where at time 0.0 the ratio is 0.0 and at 24.0 hours the ratio is 1.0. The 5-minute rainfall ratio has not been considered yet. In order to include the 5-minute ratio, the ratio at 6-minutes (0.1 hour) is calculated as:

6-minute ratio = 5-minute ratio + 0.2 * (10-minute ratio – 5-minute ratio)

To incorporate this value into the 24-hour distribution, the 6-minute ratio is subtracted from the ratio at 12.1 hours to determine the ratio at 12.0 hours. This causes the ratio at 12.0 hours to be slightly less than 0.5.

Table A7-4: Complete 24-hour distribution table in WinTR-20 (5-column) format at 0.1-hour time increment

0.00000	0.00112	0.00225	0.00339	0.00454
0.00569	0.00685	0.00802	0.00920	0.01039
0.01158	0.01278	0.01400	0.01522	0.01644
0.01768	0.01892	0.02017	0.02143	0.02270
0.02398	0.02526	0.02655	0.02785	0.02916
0.03048	0.03181	0.03314	0.03448	0.03583
0.03719	0.03855	0.03991	0.04129	0.04267
0.04406	0.04546	0.04686	0.04828	0.04970
0.05113	0.05257	0.05402	0.05547	0.05694
0.05841	0.05989	0.06138	0.06287	0.06438
0.06589	0.06741	0.06894	0.07048	0.07202
0.07358	0.07514	0.07671	0.07828	0.07987
0.08146	0.08353	0.08562	0.08774	0.08989
0.09208	0.09429	0.09653	0.09880	0.10110
0.10343	0.10579	0.10818	0.11060	0.11305
0.11553	0.11801	0.12052	0.12306	0.12563
0.12822	0.13085	0.13351	0.13620	0.13892
0.14166	0.14444	0.14725	0.15008	0.15295
0.15584	0.15939	0.16307	0.16688	0.17083
0.17491	0.17913	0.18348	0.18797	0.19259
0.19734	0.20223	0.20726	0.21242	0.21771
0.22314	0.23037	0.23799	0.24602	0.25445
0.26328	0.27359	0.28447	0.29592	0.30795
0.32054	0.34028	0.36106	0.38855	0.42468
0.48323	0.57532	0.61145	0.63894	0.65972
0.67946	0.69205	0.70408	0.71553	0.72641
0.73672	0.74555	0.75398	0.76201	0.76963
0.77686	0.78229	0.78758	0.79274	0.79777
0.80266	0.80741	0.81203	0.81652	0.82087
0.82509	0.82917	0.83312	0.83693	0.84061
0.84416	0.84705	0.84992	0.85275	0.85556
0.85834	0.86108	0.86380	0.86649	0.86915
0.87178	0.87437	0.87694	0.87948	0.88199
0.88447	0.88695	0.88940	0.89182	0.89421
0.89657	0.89890	0.90120	0.90347	0.90571
0.90792	0.91011	0.91226	0.91438	0.91647
0.91854	0.92013	0.92172	0.92329	0.92486
0.92642	0.92798	0.92952	0.93106	0.93259
0.93411	0.93562	0.93713	0.93862	0.94011
0.94159	0.94306	0.94453	0.94598	0.94743
0.94887	0.95030	0.95172	0.95314	0.95454
0.95594	0.95733	0.95871	0.96009	0.96145
0.96281	0.96417	0.96552	0.96686	0.96819
0.96952	0.97084	0.97215	0.97345	0.97474
0.97602	0.97730	0.97857	0.97983	0.98108
0.98232	0.98356	0.98478	0.98600	0.98722
0.98842	0.98961	0.99080	0.99198	0.99315
0.99431	0.99546	0.99661	0.99775	0.99888
1.0				

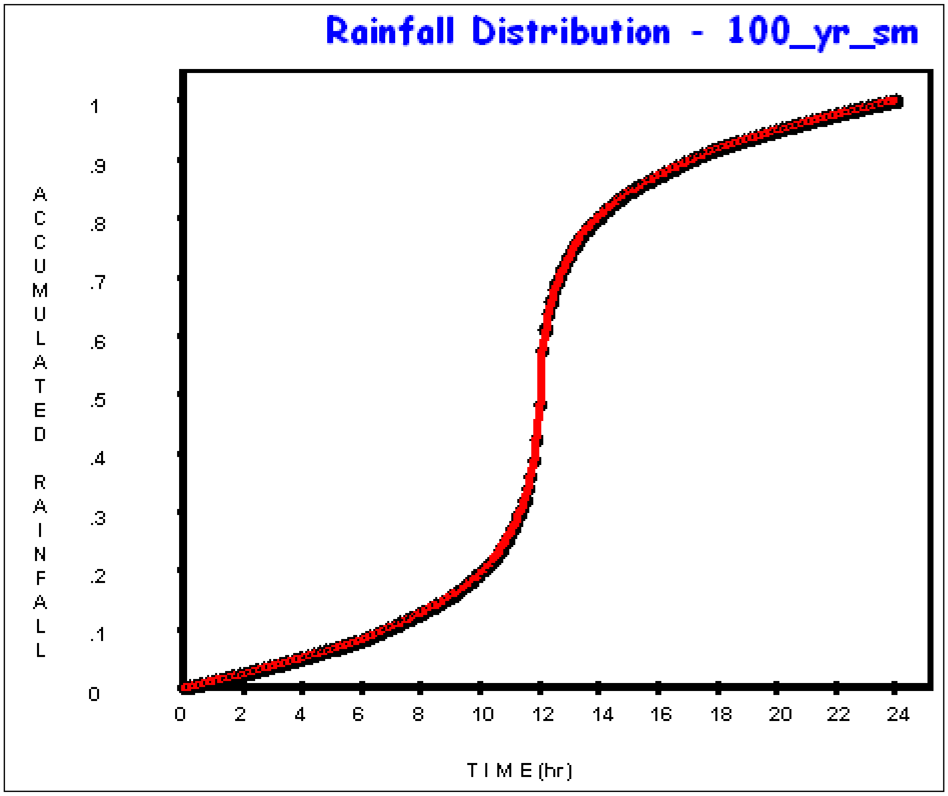

Figure A7-1: Plot of the final 100-year 24-hour storm distribution from WinTR-20

This procedure is used to develop storm distributions for return periods from 1-year to 500-years. Each distribution may be different because the ratios of the original NOAA Atlas 14 data may vary for each return period. The development of the 24-hour rainfall distribution is explained in NRCS NEH Part 630 Chapter 4 available from the NRCS Hydrology and Hydraulics web site.

Development of the 12-hour storm distribution from the 24-hour storm distribution

The 12-hour distribution is extracted from the 24-hour storm distribution developed in the previous section of Appendix 7. The 12-hour storm distribution represents the 12-hours in the 24-hour distribution from 6 hours to 18 hours.

In the example of the location in Howard County, Maryland described in the 24-hour storm section, the cumulative ratio at 6 hours is 0.08146. The cumulative rainfall ratio at 18 hours is 0.91854. The difference between these ratios is 0.83708. The 12-hour storm distribution cumulative rainfall must begin at 0.0 and end at 1.0, so to calculate the ratio at each time step of 0.1 hour, 0.08146 is subtracted from the cumulative rainfall ratio from the 24-hour storm and the result is divided by 0.83708 to obtain the cumulative ratio at that time step. Two time steps will be used in this example. The rest are computed in a similar way. At time 6.3 hours (0.3 hours in the 12-hour storm distribution), the 24-hour cumulative ratio is 0.08744. So,

Cumulative ratio at 0.3 hour = (0.08744 – 0.08146) / 0.83708 = 0.0075

Cumulative ratio at 3.0 hours = (0.15584 – 0.08146) / 0.83708 = 0.08886

The spreadsheet developed to calculate the 12-hour storm distribution automates this process. The WinTR-20 does not have the 12-hour distribution calculation included, so if the 12-hour storm distribution is desired, it should be developed through the spreadsheet and cut and pasted into the WinTR-20 input file using a text editor such as Notepad. A rainfall table header record with RAINFALL DISTRIBUTION: and a second record with an identifier (up to 10 characters) and a time interval in hours need to be placed before the table of numbers. At least one blank record needs to precede the RAINFALL DISTRIBUTION: record and follow the last line of table numbers.

Development of the 6-hour storm distribution from the 24-hour storm distribution

The 6-hour distribution is extracted from the 24-hour storm distribution developed in a previous section of Appendix 7. The 6-hour storm distribution represents the 6-hours in the 24-hour distribution from 9 hours to 15 hours.

In the example of the location in Howard County, Maryland described in the 24-hour storm section, the cumulative ratio at 9 hours is 0.15584. The cumulative rainfall ratio at 15 hours is 0.84416. The difference between these ratios is 0.68832. The 6-hour storm distribution cumulative rainfall must begin at 0.0 and end at 1.0, so to calculate the ratio at each time step of 0.1 hour, 0.15584 is subtracted from the cumulative rainfall ratio from the 24-hour storm and the result is divided by 0.68832 to obtain the cumulative ratio at that time step. Two time steps will be used in this example. The rest are computed in a similar way. At time 10.0 hours (1.0 hour in the 6-hour storm distribution), the 24-hour cumulative ratio is 0.19734. So,

Cumulative ratio at 1.0 hour = (0.19734 – 0.15584) / 0.68832 = 0.06029

Cumulative ratio at 3.0 hours = (0.48323 – 0.15584) / 0.68832 = 0.47564

The spreadsheet developed to calculate the 6-hour storm distribution automates this process. The WinTR-20 does not have the 6-hour distribution calculation included, so if the 6-hour storm distribution is desired, it should be developed through the spread sheet and cut and pasted into the WinTR-20 input file using a text editor such as Notepad. A rainfall table header record with RAINFALL DISTRIBUTION: and a second record with an identifier (up to 10 characters) and a time interval in hours need to be placed before the table of numbers. At least one blank record needs to precede the RAINFALL DISTRIBUTION: record and follow the last line of table numbers

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APPENDIX 8
HISTORICAL SUMMARY OF REGRESSION EQUATIONS TO PREDICT FLOOD FLOWS IN MARYLAND

Historical Summary of Regression Equations to Predict Flood Flows in Maryland

Regression equations have been developed over the years to estimate floods ranging from the 1.25-year event to as great as the 500-year event in the State of Maryland. Below is a summary of these regression equations from 1980 to the present.

Table A8-1: Metadata summary of regression equations documented in this appendix

Equation ID	Year Pub-lished	Last Year of Flood Obser-vation	Soils Data Source	Land Use Source (latest data)	Lime-stone Source (see notes)	Comment
Carpenter	1980	1977	unknown	USGS quadrangle maps	N/A	Three regions: Northern Region, Southern Region, Eastern Region
Dillow	1996	1990	Maryland State Department of Planning, 1973	USGS quadrangle maps	(1)	New regions defined: A, BR, P, WC, EC
Moglen et al. L-Moment	2006	1999	NRCS STATSGO	MOP 2002	(1)	
Moglen et al. ROI	2006	1999	NRCS STATSGO	MOP 2002	(1)	Uses 30 closest gages with predictors determined by ungaged outlet region
Moglen et al. Fixed Region	2006	1999	NRCS STATSGO	MOP 2002	(1)	All regions A, BR, P, WC, EC published in August 2006 Panel report
Thomas	2007	2006	NRCS SSURGO	MOP 2002	(1)	New EC only, published in September 2010 Panel report
Thomas	2009	2008	NRCS SSURGO	MOP 2002	(1)	New WC only, published in September 2010 Panel report
Maryland Hydrology Panel (3rd Edition, September 2010)	2010	1999	NRCS SSURGO	MOP 2002	(2)	BR and P combined and new P/BR rural developed P urban did not change

Equation ID	Year Pub-lished	Last Year of Flood Obser-vation	Soils Data Source	Land Use Source (latest data)	Lime-stone Source (see notes)	Comment
Thomas and Moglen	2016	2012	NRCS SSURGO	MOP 2010	(2)	New P/BR, A published in July 2016 anel report
Thomas and Sanchez-Claros	2019	2017	NRCS SSURGO (2018) Dominant Component	MOP 2010	(2)	Update for WC and EC Regions published in July 2020 Panel report
Maryland Hydrology Panel (6th Edition, 2023)	2023	2017	NRCS SSURGO (2021) Dominant Condition	MOP 2010	(2)	Updated for WC and EC Regions with new SSURGO soils data

(1) Composite from Berg (1980), Butts and Edmundson (1963), Cardwell (1968), Edwards (1978), Hubbard (1990), Jonas and Stose (1938).
(2) New Limestone layer developed by Berich/Knaub

The equations in this appendix refer to the five hydrologic regions of the state of Maryland, as shown in Figure A8-1. Starting with the Third Edition of the Hydrology Panel report in September 2010, the Blue Ridge and Great Valley Region was combined with the Piedmont Region to give just four hydrologic regions in Maryland.

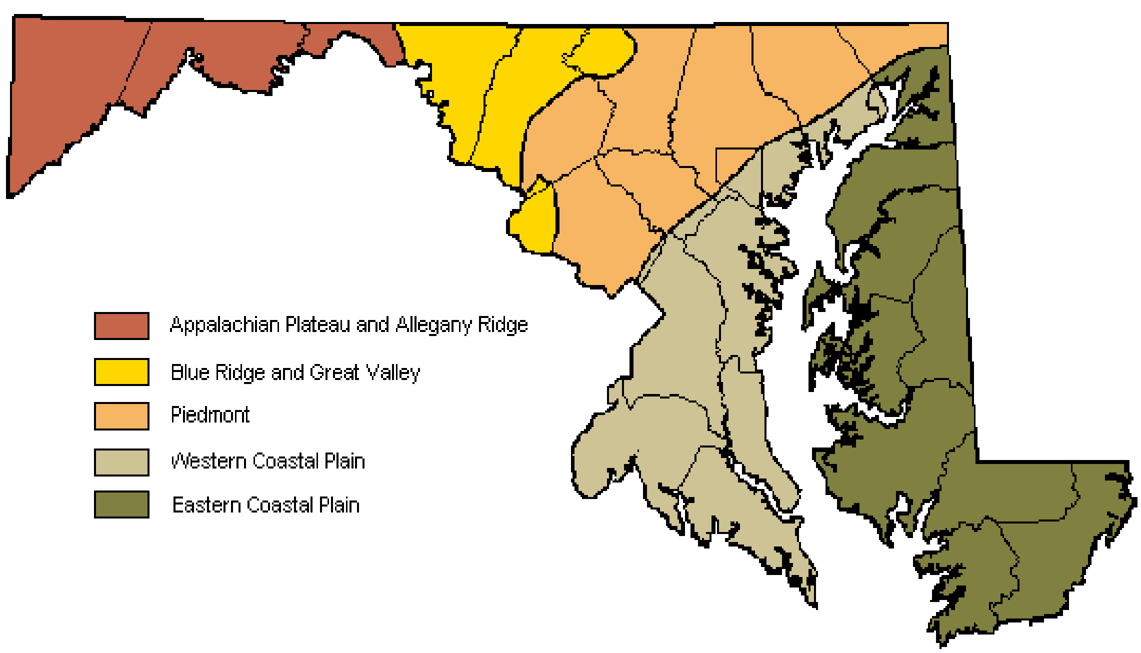

Figure A8-1. Hydrologic regions for Maryland as defined by Dillow (1996)

In the equations appearing below, the following predictor and criterion variable symbols are used:

- *BR*: basin relief, the average elevation of all area within a watershed above the outlet elevation in feet
- *DA*: drainage area in mi^2
- *FOR*: area of watershed covered by forest cover in percent
- *IA*: area of watershed that is impervious as determined using NRCS imperviousness coefficients and the Maryland Department of Planning land use data in percent
- *LIME*: area of watershed underlain by limestone geology in percent
- L_{SLOPE}: average land slope calculated on a pixel-by-pixel basis in ft/ft
- P_2: 2-year, 24 hour rainfall depth in inches
- Q_x: peak discharge for return period, *x* in ft^3/s
- *RCN*: the NRCS runoff curve number in $inches^{-1}$
- S_A: area of watershed in hydrologic soil group A in percent
- S_{CD}: area of watershed in hydrologic soil groups C and D in percent
- S_D: area of watershed in hydrologic soil group D in percent
- *SL*: main channel slope in ft/mile
- *ST*: area of watershed occupied by lakes, ponds, and swamps in percent
- *Anew*: area of watershed in hydrologic soil group A in percent (Dominant Component)
- *Acond*: area of watershed in hydrologic soil group A in percent (Dominant Condition)

At the top of each grouping of equations, the following headers are used:

EQ: equation number
SE: standard error of estimate in percent
EY: equivalent years of record

<u>Carpenter (1980) Equations</u>

Northern Region: (Appalachian/Blue Ridge/Piedmont) **EQ**

$Q_2 = 142\ DA^{0.745}\ (FOR + 10)^{-0.273}\ P_2^{0.669}$ (A8.1)

$Q_5 = 120\ DA^{0.731}\ (FOR + 10)^{-0.275}\ P_2^{1.358}$ (A8.2)

$Q_{10} = 106\ DA^{0.724}\ (FOR + 10)^{-0.286}\ P_2^{1.810}$ (A8.3)

$Q_{25} = 90.1\ DA^{0.717}\ (FOR + 10)^{-0.307}\ P_2^{2.376}$ (A8.4)

$Q_{50} = 78.5\ DA^{0.712}\ (FOR + 10)^{-0.323}\ P_2^{2.793}$ (A8.5)

$Q_{100} = 66.6\ DA^{0.708}\ (FOR + 10)^{-0.336}\ P_2^{3.212}$ (A8.6)

- Note: standard error varies from 39 to 49 percent

Southern Region (Western Coastal Plain) **EQ**

$Q_2 = 55.1DA^{0.672}$ (A8.7)

$Q_5 = 112DA^{0.670}$ (A8.8)

$Q_{10} = 172DA^{0.667}$ (A8.9)

$Q_{25} = 280DA^{0.666}$ (A8.10)

$Q_{50} = 394DA^{0.665}$ (A8.11)

$Q_{100} = 548DA^{0.662}$ (A8.12)

- Note: standard error varies from 52 to 86 percent

Eastern Region (Eastern Coastal Plain) **EQ**

$Q_2 = 28.6\ DA^{0.910}\ SL^{0.681}\ (ST + 10)^{-0.148}\ (FOR + 10)^{-0.647}\ (S_A + 10)^{-0.309}\ (S_D + 10)^{0.560}$ (A8.13)

$Q_5 = 119\ DA^{0.989}\ SL^{0.843}\ (ST + 10)^{-0.533}\ (FOR + 10)^{-0.731}\ (S_A + 10)^{-0.369}\ (S_D + 10)^{0.577}$ (A8.14)

$Q_{10} = 306\ DA^{1.016}\ SL^{0.911}\ (ST + 10)^{-0.820}\ (FOR + 10)^{-0.804}\ (S_A + 10)^{-0.367}\ (S_D + 10)^{0.624}$ (A8.15)

$Q_{25} = 936\ DA^{1.039}\ SL^{0.974}\ (ST + 10)^{-1.114}\ (FOR + 10)^{-0.868}\ (S_A + 10)^{-0.384}\ (S_D + 10)^{0.655}$ (A8.16)

$Q_{50} = 2120\ DA^{1.051}\ SL^{1.009}\ (ST + 10)^{-1.321}\ (FOR + 10)^{-0.916}\ (S_A + 10)^{-0.396}\ (S_D + 10)^{0.676}$ (A8.17)

$Q_{100} = 4800\ DA^{1.060}\ SL^{1.035}\ (ST + 10)^{-1.519}\ (FOR + 10)^{-0.963}\ (S_A + 10)^{-0.410}\ (S_D + 10)^{0.695}$ (A8.18)

- Note: standard error varies from 37 to 40 percent

Dillow (1996) Equations

Appalachian Plateaus and Allegheny Ridges region	**SE**	**EY**	**EQ**
$Q_2 = 106DA^{0.851}(FOR+10)^{-0.223}BR^{0.056}$	23	5	(A8.19)
$Q_5 = 109DA^{0.858}(FOR+10)^{-0.143}BR^{0.064}$	20	10	(A8.20)
$Q_{10} = 113DA^{0.859}(FOR+10)^{-0.106}BR^{0.072}$	19	14	(A8.21)
$Q_{25} = 118DA^{0.858}(FOR+10)^{-0.072}BR^{0.087}$	21	18	(A8.22)
$Q_{50} = 121DA^{0.858}(FOR+10)^{-0.051}BR^{0.099}$	22	20	(A8.23)
$Q_{100} = 124DA^{0.858}(FOR+10)^{-0.033}BR^{0.111}$	25	20	(A8.24)
$Q_{500} = 127DA^{0.859}(FOR+10)^{0.004}BR^{0.140}$	31	19	(A8.25)

Blue Ridge and Great Valley region	**SE**	**EY**	**EQ**
$Q_2 = 4{,}260DA^{0.774}(LIME+10)^{-0.549}BR^{-0.405}$	47	2	(A8.26)
$Q_5 = 6{,}670DA^{0.752}(LIME+10)^{-0.564}BR^{-0.354}$	41	4	(A8.27)
$Q_{10} = 8{,}740DA^{0.741}(LIME+10)^{-0.579}BR^{-0.326}$	37	7	(A8.28)
$Q_{25} = 12{,}000DA^{0.730}(LIME+10)^{-0.602}BR^{-0.295}$	35	12	(A8.29)
$Q_{50} = 15{,}100DA^{0.723}(LIME+10)^{-0.620}BR^{-0.276}$	34	15	(A8.30)
$Q_{100} = 18{,}900DA^{0.719}(LIME+10)^{-0.639}BR^{-0.261}$	34	18	(A8.31)
$Q_{500} = 31{,}800DA^{0.712}(LIME+10)^{-0.686}BR^{-0.241}$	37	23	(A8.32)

Piedmont region	**SE**	**EY**	**EQ**
$Q_2 = 451DA^{0.635}(FOR+10)^{-0.266}$	38	3	(A8.33)
$Q_5 = 839DA^{0.606}(FOR+10)^{-0.248}$	34	7	(A8.34)
$Q_{10} = 1{,}210DA^{0.589}(FOR+10)^{-0.242}$	33	10	(A8.35)
$Q_{25} = 1{,}820DA^{0.574}(FOR+10)^{-0.239}$	34	15	(A8.36)
$Q_{50} = 2{,}390DA^{0.565}(FOR+10)^{-0.240}$	36	17	(A8.37)
$Q_{100} = 3{,}060DA^{0.557}(FOR+10)^{-0.241}$	39	19	(A8.38)
$Q_{500} = 5{,}190DA^{0.543}(FOR+10)^{-0.245}$	48	20	(A8.39)

Western Coastal Plain region	**SE**	**EY**	**EQ**
$Q_2 = 1{,}410DA^{0.761}(\underline{FOR}+10)^{-0.782}$	50	2	(A8.40)
$Q_5 = 1{,}780DA^{0.769}(FOR+10)^{-0.687}$	46	4	(A8.41)
$Q_{10} = 1{,}910DA^{0.771}(FOR+10)^{-0.613}$	45	7	(A8.42)
$Q_{25} = 2{,}000DA^{0.772}(FOR+10)^{-0.519}$	46	10	(A8.43)
$Q_{50} = 2{,}060DA^{0.771}(FOR+10)^{-0.452}$	49	12	(A8.44)
$Q_{100} = 2{,}140DA^{0.770}(FOR+10)^{-0.391}$	52	13	(A8.45)
$Q_{500} = 2{,}380DA^{0.765}(FOR+10)^{-0.263}$	64	14	(A8.46)

Eastern Coastal Plain region	**SE**	**EY**	**EQ**
$Q_2 = 0.25\ DA^{0.591}(RCN-33)^{1.70}\ BR^{0.310}\ (FOR+10)^{-0.464}\ (ST+10)^{-0.148}$	42	2	(A8.47)
$Q_5 = 1.05\ DA^{0.595}(RCN-33)^{1.74}\ BR^{0.404}(FOR+10)^{-0.586}\ (ST+10)^{-0.498}$	40	5	(A8.48)
$Q_{10} = 3.24\ DA^{0.597}(RCN-33)^{1.71}\ BR^{0.436}\ (FOR+10)^{-0.667}(ST+10)^{-0.694}$	39	7	(A8.49)
$Q_{25} = 13.1\ DA^{0.597}(RCN-33)^{1.66}\ BR^{0.457}(FOR+10)^{-0.770}\ (ST+10)^{-0.892}$	37	12	(A8.50)
$Q_{50} = 35.0\ DA^{0.594}\ (RCN-33)^{1.62}\ BR^{0.465}\ (FOR+10)^{-0.847}\ (ST+10)^{-1.01}$	37	16	(A8.51)
$Q_{100} = 87.6\ DA^{0.589}\ (RCN-33)^{1.58}\ BR^{0.470}\ (FOR+10)^{-0.923}\ (ST+10)^{-1.11}$	36	19	(A8.52)
$Q_{500} = 627\ DA^{0.573}(RCN-33)^{1.49}\ BR^{0.478}(FOR+10)^{-1.10}\ (ST+10)^{-1.29}$	36	28	(A8.53)

Moglen et al. (2006) – Fixed Region Equations

Appalachian Plateaus Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 70.25\ DA^{0.837}\ L_{SLOPE}^{0.327}$	23.6	5.7	(A8.54)
$Q_{1.50} = 87.42\ DA^{0.837}\ L_{SLOPE}^{0.321}$	21.9	5.9	(A8.55)
$Q_{1.75} = 96.37\ DA^{0.836}\ L_{SLOPE}^{0.307}$	21.2	6.4	(A8.56)
$Q_{2} = 101.41\ DA^{0.834}\ L_{SLOPE}^{0.300}$	20.7	7.1	(A8.57)
$Q_{5} = 179.13\ DA^{0.826}\ L_{SLOPE}^{0.314}$	21.6	12	(A8.58)
$Q_{10} = 255.75\ DA^{0.821}\ L_{SLOPE}^{0.340}$	24.2	14	(A8.59)
$Q_{25} = 404.22\ DA^{0.812}\ L_{SLOPE}^{0.393}$	29.1	15	(A8.60)
$Q_{50} = 559.80\ DA^{0.806}\ L_{SLOPE}^{0.435}$	33.1	16	(A8.61)
$Q_{100} = 766.28\ DA^{0.799}\ L_{SLOPE}^{0.478}$	37.4	15	(A8.62)
$Q_{200} = 1046.9\ DA^{0.793}\ L_{SLOPE}^{0.525}$	41.8	15	(A8.63)
$Q_{500} = 1565.0\ DA^{0.784}\ L_{SLOPE}^{0.589}$	48.0	15	(A8.64)

Blue Ridge Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 57.39\ DA^{0.784}\ (LIME+1)^{-0.190}$	74.6	1.0	(A8.65)
$Q_{1.50} = 81.45\ DA^{0.764}\ (LIME+1)^{-0.193}$	67.1	1.1	(A8.66)
$Q_{1.75} = 96.33\ DA^{0.755}\ (LIME+1)^{-0.194}$	65.2	1.2	(A8.67)
$Q_{2} = 107.20\ DA^{0.750}\ (LIME+1)^{-0.194}$	64.0	1.3	(A8.68)
$Q_{5} = 221.28\ DA^{0.710}\ (LIME+1)^{-0.202}$	55.4	3.0	(A8.69)
$Q_{10} = 336.84\ DA^{0.687}\ (LIME+1)^{-0.207}$	52.5	4.9	(A8.70)
$Q_{25} = 545.62\ DA^{0.660}\ (LIME+1)^{-0.214}$	51.6	8.8	(A8.71)
$Q_{50} = 759.45\ DA^{0.641}\ (LIME+1)^{-0.219}$	52.5	9.7	(A8.72)
$Q_{100} = 1034.7\ DA^{0.624}\ (LIME+1)^{-0.224}$	54.4	11	(A8.73)
$Q_{200} = 1387.6\ DA^{0.608}\ (LIME+1)^{-0.229}$	57.4	13	(A8.74)
$Q_{500} = 2008.6\ DA^{0.587}\ (LIME+1)^{-0.235}$	62.3	13	(A8.75)

Piedmont Region: Rural	**SE**	**EY**	**EQ**
$Q_{1.25} = 202.9\ DA^{0.682}\ (FOR+1)^{-0.222}$	39.0	3.3	(A8.76)
$Q_{1.50} = 262.0\ DA^{0.683}\ (FOR+1)^{-0.217}$	33.8	3.8	(A8.77)
$Q_{1.75} = 308.9\ DA^{0.679}\ (FOR+1)^{-0.219}$	32.1	4.3	(A8.78)
$Q_{2} = 349.0\ DA^{0.674}\ (FOR+1)^{-0.224}$	31.3	4.8	(A8.79)
$Q_{5} = 673.8\ DA^{0.659}\ (FOR+1)^{-0.228}$	25.6	14	(A8.80)
$Q_{10} = 992.6\ DA^{0.649}\ (FOR+1)^{-0.230}$	24.3	23	(A8.81)
$Q_{25} = 1556\ DA^{0.635}\ (FOR+1)^{-0.231}$	25.3	33	(A8.82)
$Q_{50} = 2146\ DA^{0.624}\ (FOR+1)^{-0.235}$	27.5	37	(A8.83)
$Q_{100} = 2897\ DA^{0.613}\ (FOR+1)^{-0.238}$	30.6	37	(A8.84)
$Q_{200} = 3847\ DA^{0.603}\ (FOR+1)^{-0.239}$	34.2	37	(A8.85)
$Q_{500} = 5519\ DA^{0.589}\ (FOR+1)^{-0.242}$	39.7	35	(A8.86)

Piedmont Region: Urban	**SE**	**EY**	**EQ**
$Q_{1.25} = 17.85\ DA^{0.652}\ (IA+1)^{0.635}$	41.7	3.3	(A8.87)
$Q_{1.50} = 24.66\ DA^{0.648}\ (IA+1)^{0.631}$	36.9	3.8	(A8.88)
$Q_{1.75} = 30.82\ DA^{0.643}\ (IA+1)^{0.611}$	35.6	4.1	(A8.89)
$Q_{2} = 37.01\ DA^{0.635}\ (IA+1)^{0.588}$	35.1	4.5	(A8.90)
$Q_{5} = 94.76\ DA^{0.624}\ (IA+1)^{0.499}$	28.5	13	(A8.91)
$Q_{10} = 169.2\ DA^{0.622}\ (IA+1)^{0.435}$	26.2	24	(A8.92)
$Q_{25} = 341.0\ DA^{0.619}\ (IA+1)^{0.349}$	26.0	38	(A8.93)
$Q_{50} = 562.4\ DA^{0.619}\ (IA+1)^{0.284}$	27.7	44	(A8.94)
$Q_{100} = 898.3\ DA^{0.619}\ (IA+1)^{0.222}$	30.7	45	(A8.95)
$Q_{200} = 1413\ DA^{0.621}\ (IA+1)^{0.160}$	34.8	44	(A8.96)
$Q_{500} = 2529\ DA^{0.623}\ (IA+1)^{0.079}$	41.2	40	(A8.97)

Western Coastal Plain Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 18.62\ DA^{0.611}\ (IA+1)^{0.419}\ (S_D+1)^{0.165}$	38.9	3.2	(A8.98)
$Q_{1.50} = 21.97\ DA^{0.612}\ (IA+1)^{0.399}\ (S_D+1)^{0.226}$	36.3	3.2	(A8.99)
$Q_{1.75} = 24.42\ DA^{0.612}\ (IA+1)^{0.391}\ (S_D+1)^{0.246}$	35.6	3.4	(A8.100)
$Q_{2} = 26.32\ DA^{0.612}\ (IA+1)^{0.386}\ (S_D+1)^{0.256}$	35.4	3.7	(A8.101)
$Q_{5} = 42.64\ DA^{0.607}\ (IA+1)^{0.347}\ (S_D+1)^{0.340}$	36.3	6.8	(A8.102)
$Q_{10} = 58.04\ DA^{0.603}\ (IA+1)^{0.323}\ (S_D+1)^{0.382}$	40.6	8.4	(A8.103)
$Q_{25} = 86.25\ DA^{0.582}\ (IA+1)^{0.295}\ (S_D+1)^{0.421}$	48.9	9.3	(A8.104)
$Q_{50} = 111.50\ DA^{0.584}\ (IA+1)^{0.270}\ (S_D+1)^{0.457}$	54.7	9.9	(A8.105)
$Q_{100} = 143.56\ DA^{0.586}\ (IA+1)^{0.260}\ (S_D+1)^{0.469}$	65.7	9.0	(A8.106)
$Q_{200} = 185.15\ DA^{0.580}\ (IA+1)^{0.243}\ (S_D+1)^{0.488}$	75.5	8.7	(A8.107)
$Q_{500} = 256.02\ DA^{0.573}\ (IA+1)^{0.222}\ (S_D+1)^{0.510}$	89.8	8.3	(A8.108)

Eastern Coastal Plain Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 19.85\ DA^{0.796}\ BR^{0.066}\ (S_A+1)^{-0.106}$	34.2	4.5	(A8.109)
$Q_{1.50} = 20.48\ DA^{0.795}\ BR^{0.156}\ (S_A+1)^{-0.140}$	33.7	4.1	(A8.110)
$Q_{1.75} = 20.81\ DA^{0.799}\ BR^{0.197}\ (S_A+1)^{-0.146}$	34.2	4.1	(A8.111)
$Q_{2} = 20.95\ DA^{0.803}\ BR^{0.222}\ (S_A+1)^{-0.144}$	34.9	4.1	(A8.112)
$Q_{5} = 25.82\ DA^{0.793}\ BR^{0.368}\ (S_A+1)^{-0.190}$	36.9	6.8	(A8.113)
$Q_{10} = 31.17\ DA^{0.777}\ BR^{0.439}\ (S_A+1)^{-0.215}$	38.2	9.5	(A8.114)
$Q_{25} = 40.26\ DA^{0.751}\ BR^{0.511}\ (S_A+1)^{-0.242}$	40.0	13	(A8.115)
$Q_{50} = 50.00\ DA^{0.732}\ BR^{0.549}\ (S_A+1)^{-0.261}$	41.7	16	(A8.116)
$Q_{100} = 63.44\ DA^{0.711}\ BR^{0.576}\ (S_A+1)^{-0.279}$	44.0	18	(A8.117)
$Q_{200} = 79.81\ DA^{0.689}\ BR^{0.601}\ (S_A+1)^{-0.296}$	46.5	19	(A8.118)
$Q_{500} = 108.7\ DA^{0.660}\ BR^{0.628}\ (S_A+1)^{-0.316}$	50.8	21	(A8.119)

Moglen et al. (2006) – L-Moment Equations

Using codes published by Wallis and Hosking, (1998), five homogeneous regions over the State of Maryland were determined that corresponded well with the framework established previously by Dillow (1996). These regions are: Appalachian, Blue Ridge / Piedmont (<20 mi^2), Blue Ridge / Piedmont (> 20 mi^2), Western Coastal Plain, and Eastern Coastal Plain. These regions agree well with those established by Dillow with the difference being the merger of the Blue Ridge and Piedmont provinces but with this merged province divided based on whether the drainage area is greater or less than 20 mi^2.

The output from the L-moment methods is an equation for determining the L-mean and a set of quantiles that correspond to the various return periods one may wish to estimate a flood discharge for. This is basically an index flood procedure in which the discharge for any given return period is the product of the L-mean determined for that watershed (A8.nd region) and the quantile for the return period and region.

The equations for the L-means for each region are given below in equations 38-42. Table A8.2 provides the corresponding quantiles for each region.

Table A8-2: Quantiles for L-moment method by region and return period

Region	**Return Period**										
	1.25	**1.50**	**1.75**	**2**	**5**	**10**	**25**	**50**	**100**	**200**	**500**
Appalachian	0.55	0.67	0.76	0.83	1.31	1.70	2.32	2.89	3.55	4.34	5.61
Piedmont/Blue Ridge < 20 mi^2	0.36	0.49	0.60	0.69	1.34	1.96	3.03	4.12	5.52	7.34	10.60
Piedmont/Blue Ridge > 20 mi^2	0.48	0.61	0.70	0.78	1.32	1.80	2.58	3.33	4.26	5.4	7.32
Western Coastal Plain	0.40	0.53	0.62	0.70	1.30	1.88	2.90	3.95	5.33	7.14	10.43
Eastern Coastal Plain	0.45	0.60	0.71	0.80	1.38	1.86	2.60	3.26	4.03	4.93	6.37

EQ

For the Appalachian Region:

$$L_A = 18.4606 \cdot (DA)^{0.8234} \cdot (S_C + 10)^{0.3186} \qquad \text{(A8.120)}$$

For the Blue Ridge / Piedmont Region less than 20 mi^2:

$$L_{BR-P<20} = 1551.203 \cdot (DA)^{0.5202} \cdot (LIME + 10)^{-0.7158} \qquad \text{(A8.121)}$$

For the Blue Ridge / Piedmont Region more than 20 mi^2:

$$L_{BR-P>20} = 1035.085 \cdot (DA)^{0.6489} \cdot (LIME + 10)^{-0.6525} \qquad \text{(A8.122)}$$

For the Western Coastal Plain Region:

$$L_W = 0.0107 \cdot (BR)^{1.9} \cdot (S_D + 10)^{0.7446} \qquad \text{(A8.123)}$$

For the Eastern Coastal Plain Region:

$$L_E = 3.5742 \cdot (DA)^{0.6189} \cdot (BR)^{0.9183} \qquad \text{(A8.124)}$$

Moglen et al. (2006) – Region of Influence Equations

The concept behind the Region of Influence (ROI) method is to develop regression models based on the flood frequency information of the *n* most similar gaged sites to the ungaged watershed in question. Regression models are thus unique to every ungaged location. Similarity is asserted by examining such watershed properties as those that appeared in the earlier section on determining watershed properties. The ROI method uses as input the ungaged watershed properties and then determines a "distance" function that reflects the similarity of the ungaged site to all gaged sites in the database. The *n* gaged sites that are the most similar (i.e. have the smallest "distance") to the ungaged site are used to develop a multiple-predictor power law regression model for each return period from 2 to 500 years.

To determine the best predictors to use in this method, two-, three-, and four-parameter models were examined with the smallest standard errors associated with the higher parameter models. Testing of models was performed by treating the data from each gage as if it were an ungaged location and then using the remaining gages to predict the flood frequency distribution at this gage. Gages were grouped according to the physiographic provinces identified by Dillow (1996). The standard error within each physiographic province was then calculated using the Bulletin 17b discharges at each gage as the "observed" discharges and the ROI determined regression equations as the "predicted" discharges.

The original ROI code was obtained from Gary Tasker at the USGS for the State of North Carolina and was modified to work for the State of Maryland. The original code set *n* (the number of most similar gages used to develop a regression equation) at 30. A small analysis confirmed that $n = 30$ produced the smallest standard errors representing the tradeoff between gaining more information from larger sample sizes and having that information be of lower quality because it corresponds to less similar gages.

Table A8-3: Best predictors for Region of Influence Method – by geographic province

Physiographic Province	Predictor 1	Predictor 2	Predictor 3	Predictor 4
Appalachian	Drainage area	Basin relief	A soils	Forest cover (1985)
Blue Ridge*	Drainage area	Limestone	Forest cover (1985)	Impervious area (1985)
Piedmont*	Drainage area	Limestone	Forest cover (1985)	Impervious area (1985)
Western Coastal Plain	Drainage area	Land slope	Impervious area (1990)	D soils
Eastern Coastal Plain	Drainage area	Basin relief	A soils	Forest Cover (1985)

Model development was a lengthy undertaking, proceeding largely by a trial-and-error process. The models that were examined all included drainage area as a predictor by default. The remaining predictors were allowed to vary although it was expected that a blend of predictors reflecting structural properties of the watershed (e.g. relief, soil type) and dynamic properties of the watershed (e.g. forest cover, imperviousness) would ultimately produce the best regression models. Ultimately, after trying a great number of potential models, we found that that the most effective watershed predictors were drainage area, land slope, basin relief, percent imperviousness, and forest cover. Table 1 below shows the precise best predictive models found, grouped by physiographic province.

The reader will note that Table 1 indicates that the models that were ultimately found to produce the smallest standard errors also contained highly correlated predictors (e.g. [land slope and basin relief] or [impervious area and forest cover]). These correlated predictors resulted in regression models with irrational exponents (e.g. a negative exponent on land slope or basin relief). We subsequently searched for the best regression model with more independent predictors. The resulting best four predictor model was found to be dependent on drainage area, basin relief, percent imperviousness, and percent hydrologic soil group D. The exponents associated with this model were rational in sign and were rational in their trends with increasing return period (e.g. the exponent on percent imperviousness decreases as return period increases.) Although there is a small sacrifice in the magnitude of the standard errors during the calibration step using this new model, we feel the rationality of the exponents is ultimately more important when using these equations to make predictions at ungaged sites.

The asterisk (*) next to the Blue Ridge and Piedmont regions indicates a slight difference in treatment of the Region of Influence methods for these two regions with regards to the limestone predictor. The presence of an underlying limestone geology in these areas has been found by others to be significant for both low flows (Carpenter and Hayes, 1996) and for floods (Dillow, 1996). For the two indicated regions, the limestone predictor was handled as follows. If the percent limestone was greater than zero, then all four predictors were used. If no limestone was present then the initial set of calibrated models did not include limestone as a predictor.

A final wrinkle we added beyond the standard region of influence method was a test for rationality of all exponents in the calibrated regression equations. If an irrational exponent (e.g. a negative exponent on land slope or a positive exponent on forest cover) was determined for any predictor for any return period, that predictor was removed from the set of predictors and the region of influence method was repeated with the reduced predictor set. For the calibration of a set of equations for a given region, this process was repeated, starting with the four predictors indicated in Table 1 until all calibrated exponents were rational for all return periods.

Third Edition of the Maryland Hydrology Panel dated September 2010 Equations

The GIS representation of limestone up to the year 2010 had been a digitized and georeferenced representation of the limestone geology layer published in Dillow's (1996) report. The Maryland Hydrology Panel was aware of other limestone geology extending into the Maryland Piedmont region that was not represented in this layer. Further, limestone had not previously been examined as a potential predictor variable in the Piedmont region. The Maryland Hydrology Panel undertook an extensive analysis to determine if the limestone representation used for predicting flood behavior could be revised resulting in improved predictive capabilities of the resulting regression equations.

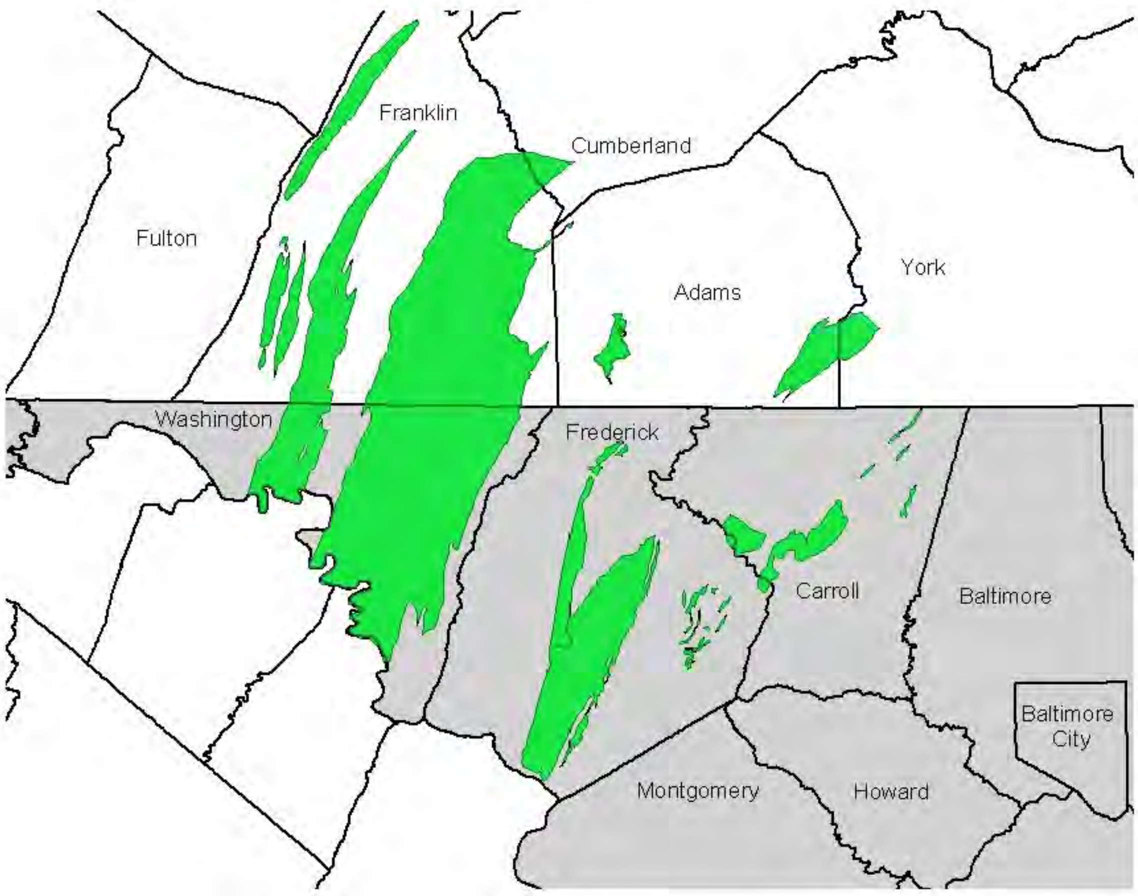

Figure A8-2: Green areas are underlain by limestone geology used in 2010 and more recent equations

<u>Third Edition of the Maryland Hydrology Panel dated September 2010 Equations</u> (continued)

Piedmont/Blue Ridge Region: Rural	**SE**	**EY**	**EQ**
$Q_{1.25} = 287.1\ DA^{0.774}\ (LIME{+}1)^{-0.118}\ (FOR{+}1)^{-0.418}$	42.1	2.8	(A8.125)
$Q_{1.50} = 327.3\ DA^{0.758}\ (LIME{+}1)^{-0.121}\ (FOR{+}1)^{-0.358}$	37.6	3.1	(A8.126)
$Q_{2} = 396.9\ DA^{0.743}\ (LIME{+}1)^{-0.124}\ (FOR{+}1)^{-0.332}$	35.6	3.7	(A8.127)
$Q_{5} = 592.5\ DA^{0.705}\ (LIME{+}1)^{-0.133}\ (FOR{+}1)^{-0.237}$	31.4	9.0	(A8.128)
$Q_{10} = 751.1\ DA^{0.682}\ (LIME{+}1)^{-0.138}\ (FOR{+}1)^{-0.183}$	30.9	14	(A8.129)
$Q_{25} = 996.0\ DA^{0.655}\ (LIME{+}1)^{-0.145}\ (FOR{+}1)^{-0.122}$	32.2	20	(A8.130)
$Q_{50} = 1{,}218.8\ DA^{0.635}\ (LIME{+}1)^{-0.150}\ (FOR{+}1)^{-0.082}$	34.5	23	(A8.131)
$Q_{100} = 1{,}471.1\ DA^{0.617}\ (LIME{+}1)^{-0.154}\ (FOR{+}1)^{-0.045}$	37.5	24	(A8.132)
$Q_{200} = 1{,}760.7\ DA^{0.600}\ (LIME{+}1)^{-0.159}\ (FOR{+}1)^{-0.009}$	41.0	25	(A8.133)
$Q_{500} = 2{,}215.4\ DA^{0.577}\ (LIME{+}1)^{-0.165}\ (FOR{+}1)^{0.035}$	46.3	25	(A8.134)

Piedmont Region: Urban	**SE**	**EY**	**EQ**
$Q_{1.25} = 17.85\ DA^{0.652}\ (IA{+}1)^{0.635}$	41.7	3.3	(A8.135)
$Q_{1.50} = 24.66\ DA^{0.648}\ (IA{+}1)^{0.631}$	36.9	3.8	(A8.136)
$Q_{2} = 37.01\ DA^{0.635}\ (IA{+}1)^{0.588}$	35.1	4.5	(A8.137)
$Q_{5} = 94.76\ DA^{0.624}\ (IA{+}1)^{0.499}$	28.5	13	(A8.137)
$Q_{10} = 169.2\ DA^{0.622}\ (IA{+}1)^{0.435}$	26.2	24	(A8.139)
$Q_{25} = 341.0\ DA^{0.619}\ (IA{+}1)^{0.349}$	26.0	38	(A8.140)
$Q_{50} = 562.4\ DA^{0.619}\ (IA{+}1)^{0.284}$	27.7	44	(A8.141)
$Q_{100} = 898.3\ DA^{0.619}\ (IA{+}1)^{0.222}$	30.7	45	(A8.142)
$Q_{200} = 1{,}413\ DA^{0.621}\ (IA{+}1)^{0.160}$	34.8	44	(A8.143)
$Q_{500} = 2{,}529\ DA^{0.623}\ (IA{+}1)^{0.079}$	41.2	40	(A8.144)

Western Coastal Plains Region

	SE	EY	EQ
$Q_{1.25} = 5.18\ DA^{0.694}\ (IA{+}1)^{0.382}\ (S_{CD}{+}1)^{0.414}$	39.0	3.6	(A8.145)
$Q_{1.50} = 6.73\ DA^{0.682}\ (IA{+}1)^{0.374}\ (S_{CD}{+}1)^{0.429}$	36.4	3.6	(A8.146)
$Q_{2} = 7.61\ DA^{0.678}\ (IA{+}1)^{0.362}\ (S_{CD}{+}1)^{0.475}$	33.2	4.6	(A8.147)
$Q_{5} = 10.5\ DA^{0.665}\ (IA{+}1)^{0.290}\ (S_{CD}{+}1)^{0.612}$	38.2	6.7	(A8.148)
$Q_{10} = 13.1\ DA^{0.653}\ (IA{+}1)^{0.270}\ (S_{CD}{+}1)^{0.669}$	42.7	8.2	(A8.149)
$Q_{25} = 17.5\ DA^{0.634}\ (IA{+}1)^{0.264}\ (S_{CD}{+}1)^{0.719}$	48.1	10	(A8.150)
$Q_{50} = 21.2\ DA^{0.621}\ (IA{+}1)^{0.263}\ (S_{CD}{+}1)^{0.751}$	54.0	11	(A8.151)
$Q_{100} = 25.6\ DA^{0.608}\ (IA{+}1)^{0.262}\ (S_{CD}{+}1)^{0.781}$	61.2	11	(A8.152)
$Q_{200} = 30.5\ DA^{0.596}\ (IA{+}1)^{0.261}\ (S_{CD}{+}1)^{0.812}$	69.6	10	(A8.153)
$Q_{500} = 37.9\ DA^{0.579}\ (IA{+}1)^{0.261}\ (S_{CD}{+}1)^{0.849}$	82.5	10	(A8.154)

Eastern Coastal Plains Region

	SE	EY	EQ
$Q_{1.25} = 24.44\ DA^{0.815}\ (S_A{+}1)^{-0.139}\ L_{SLOPE}^{0.115}$	32.4	4.6	(A8.155)
$Q_{1.50} = 32.14\ DA^{0.824}\ (S_A{+}1)^{-0.144}\ L_{SLOPE}^{0.194}$	32.3	4.1	(A8.156)
$Q_{2} = 42.48\ DA^{0.836}\ (S_A{+}1)^{-0.158}\ L_{SLOPE}^{0.249}$	32.8	4.4	(A8.157)
$Q_{5} = 81.20\ DA^{0.847}\ (S_A{+}1)^{-0.184}\ L_{SLOPE}^{0.385}$	35.1	7.0	(A8.158)
$Q_{10} = 119.3\ DA^{0.844}\ (S_A{+}1)^{-0.196}\ L_{SLOPE}^{0.445}$	36.7	9.7	(A8.159)
$Q_{25} = 186.7\ DA^{0.834}\ (S_A{+}1)^{-0.212}\ L_{SLOPE}^{0.499}$	39.3	13	(A8.160)
$Q_{50} = 254.7\ DA^{0.824}\ (S_A{+}1)^{-0.222}\ L_{SLOPE}^{0.531}$	41.6	15	(A8.161)
$Q_{100} = 340.4\ DA^{0.812}\ (S_A{+}1)^{-0.230}\ L_{SLOPE}^{0.557}$	44.2	17	(A8.162)
$Q_{200} = 450.5\ DA^{0.800}\ (S_A{+}1)^{-0.237}\ L_{SLOPE}^{0.582}$	47.2	18	(A8.163)
$Q_{500} = 638.7\ DA^{0.783}\ (S_A{+}1)^{-0.247}\ L_{SLOPE}^{0.610}$	51.6	19	(A8.164)

<u>Thomas and Moglen (2016) Equations - Published in Fourth Edition of Maryland Hydrology Panel report dated July 2016</u>

<u>Regression equations for the Western and Eastern Coastal Plain Regions were not updated in July 2016 report.</u>

Piedmont/Blue Ridge Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 283.3\ DA^{0.724}\ (LIME{+}1)^{-0.124}\ (IA{+}1)^{0.143}(FOR{+}1)^{-0.412}$	44.3	2.8	(A8.165)
$Q_{1.50} = 352.4\ DA^{0.704}\ (LIME{+}1)^{-0.131}\ (IA{+}1)^{0.123}(FOR{+}1)^{-0.373}$	40.9	3.2	(A8.166)
$Q_{2} = 453.4\ DA^{0.683}\ (LIME{+}1)^{-0.140}\ (IA{+}1)^{0.105}(FOR{+}1)^{-0.334}$	37.5	3.7	(A8.167)
$Q_{5} = 746.8\ DA^{0.640}\ (LIME{+}1)^{-0.158}\ (IA{+}1)^{0.083}(FOR{+}1)^{-0.249}$	31.9	9.2	(A8.168)
$Q_{10} = 972.3\ DA^{0.615}\ (LIME{+}1)^{-0.169}\ (IA{+}1)^{0.076}(FOR{+}1)^{-0.195}$	29.6	16	(A8.169)
$Q_{25} = 1{,}327.6\ DA^{0.593}\ (LIME{+}1)^{-0.182}\ (IA{+}1)^{0.074}(FOR{+}1)^{-0.145}$	29.0	25	(A8.170)
$Q_{50} = 1{,}608.2\ DA^{0.576}\ (LIME{+}1)^{-0.191}\ (IA{+}1)^{0.073}(FOR{+}1)^{-0.103}$	29.8	31	(A8.171)
$Q_{100} = 1{,}928.5\ DA^{0.561}\ (LIME{+}1)^{-0.198}\ (IA{+}1)^{0.073}(FOR{+}1)^{-0.067}$	31.8	34	(A8.172)
$Q_{200} = 3{,}153.5\ DA^{0.550}\ (LIME{+}1)^{-0.222}\ (FOR{+}1)^{-0.090}$	35.7	32	(A8.173)
$Q_{500} = 3{,}905.3\ DA^{0.533}\ (LIME{+}1)^{-0.233}\ (FOR{+}1)^{-.0.045}$	42.0	30	(A8.174)

Appalachian Plateau Region	**SE**	**EY**	**EQ**
$Q_{1.25} = 71.0\ DA^{0.848}\ L_{SLOPE}^{0.342}$	30.9	1.2	(A8.175)
$Q_{1.50} = 86.3\ DA^{0.837}\ L_{SLOPE}^{0.312}$	23.3	3.7	(A8.176)
$Q_{2} = 112.7\ DA^{0.829}\ L_{SLOPE}^{0.319}$	21.1	6.6	(A8.177)
$Q_{5} = 199.1\ DA^{0.813}\ L_{SLOPE}^{0.339}$	21.1	11	(A8.178)
$Q_{10} = 272.2\ DA^{0.801}\ L_{SLOPE}^{0.338}$	24.5	12	(A8.179)
$Q_{25} = 416.9\ DA^{0.794}\ L_{SLOPE}^{0.380}$	27.9	14	(A8.180)
$Q_{50} = 570.5\ DA^{0.790}\ L_{SLOPE}^{0.422}$	32.5	14	(A8.181)
$Q_{100} = 722.0\ DA^{0.783}\ L_{SLOPE}^{0.429}$	37.1	13	(A8.182)
$Q_{200} = 914.5\ DA^{0.777}\ L_{SLOPE}^{0.445}$	42.6	12	(A8.183)
$Q_{500} = 1{,}174.3\ DA^{0.768}\ L_{SLOPE}^{0.437}$	49.8	11	(A8.184)

Fifth Edition of the Maryland Hydrology Panel report dated July 2020

Thomas and Moglen (2016) equations were revised by adjusting flood frequency estimates downward for 13 small rural watersheds, no other changes.

Piedmont-Blue Ridge Region

Equation	SE	EY	EQ
$Q_{1\_25} = 63.0\ DA^{0.685}\ (LIME+1)^{-0.090}\ (IA+1)^{0.284}$	53.1	2.0	(A8.185)
$Q_{1\_50} = 89.8\ DA^{0.669}\ (LIME+1)^{-0.100}\ (IA+1)^{0.253}$	48.3	2.4	(A8.186)
$Q_{2} = 131.7\ DA^{0.653}\ (LIME+1)^{-0.112}\ (IA+1)^{0.225}$	43.6	2.8	(A8.187)
$Q_{5} = 283.7\ DA^{0.625}\ (LIME+1)^{-0.136}\ (IA+1)^{0.184}$	35.2	8.3	(A8.188)
$Q_{10} = 434.7\ DA^{0.610}\ (LIME+1)^{-0.148}\ (IA+1)^{0.166}$	31.6	14	(A8.189)
$Q_{25} = 683.3\ DA^{0.599}\ (LIME+1)^{-0.164}\ (IA+1)^{0.153}$	30.0	24	(A8.190)
$Q_{50} = 929.3\ DA^{0.591}\ (LIME+1)^{-0.174}\ (IA+1)^{0.145}$	30.8	29	(A8.191)
$Q_{100} = 1{,}240.1\ DA^{0.584}\ (LIME+1)^{-0.184}\ (IA+1)^{0.139}$	33.0	32	(A8.192)
$Q_{200} = 1{,}616.8\ DA^{0.578}\ (LIME+1)^{-0.193}\ (IA+1)^{0.134}$	36.6	31	(A8.193)
$Q_{500} = 2{,}252.2\ DA^{0.571}\ (LIME+1)^{-0.205}\ (IA+1)^{0.129}$	42.9	29	(A8.194)

Thomas and Moglen (2016) equations were revised by using land slope data based on DEM dated May 2018, no other changes.

Appalachian Plateau Region

Equation	SE	EY	EQ
$Q_{1\_25} = 79.4\ DA^{0.840}\ LSLOPE^{0.397}$	29.2	1.3	(A8.195)
$Q_{1\_5} = 92.4\ DA^{0.831}\ LSLOPE^{0.348}$	21.8	4.4	(A8.196)
$Q_{2} = 115.2\ DA^{0.825}\ LSLOPE^{0.333}$	19.9	7.5	(A8.197)
$Q_{5} = 183.4\ DA^{0.813}\ LSLOPE^{0.306}$	20.7	11	(A8.198)
$Q_{10} = 221.2\ DA^{0.808}\ LSLOPE^{0.248}$	24.9	12	(A8.199)
$Q_{25} = 317.6\ DA^{0.803}\ LSLOPE^{0.261}$	28.7	13	(A8.200)
$Q_{50} = 397.6\ DA^{0.803}\ LSLOPE^{0.263}$	33.6	13	(A8.201)
$Q_{100} = 474.5\ DA^{0.799}\ LSLOPE^{0.244}$	38.3	12	(A8.202)
$Q_{200}= 559.4\ DA^{0.795}\ LSLOPE^{0.227}$	44.0	11	(A8.203)
$Q_{500} = 664.0\ DA^{0.790}\ LSLOPE^{0.183}$	51.3	10	(A8.204)

Eastern and Western Coastal Plain Regions equations were updated by Thomas and Sanchez-Claros (2019a, 2019b, respectively) – percent Anew soils based on SSURGO data dated May 2018 using the Dominant Component approach.

Eastern Coastal Plain

Equation	SE	EY	EQ
$Q_{1.25} = 35.6\ DA^{0.757}\ LSLOPE^{0.127}\ 10^{-0.00815*Anew}$	45.6	2.8	(A8.205)
$Q_{1.5} = 48.0\ DA^{0.757}\ LSLOPE^{0.202}\ 10^{-0.00871*Anew}$	43.6	3.0	(A8.206)
$Q_{2} = 67.3\ DA^{0.751}\ LSLOPE^{0.281}\ 10^{-0.00919*Anew}$	41.8	3.3	(A8.207)
$Q_{5} = 134.8\ DA^{0.737}\ LSLOPE^{0.473}\ 10^{-0.01027*Anew}$	39.5	6.9	(A8.208)
$Q_{10} = 200.0\ DA^{0.725}\ LSLOPE^{0.605}\ 10^{-0.01091*Anew}$	38.9	11	(A8.209)
$Q_{25} = 314.5\ DA^{0.707}\ LSLOPE^{0.793}\ 10^{-0.01151*Anew}$	39.0	19	(A8.210)
$Q_{50} = 420.6\ DA^{0.700}\ LSLOPE^{0.895}\ 10^{-0.01202*Anew}$	39.8	19	(A8.211)
$Q_{100} = 551.2\ DA^{0.692}\ LSLOPE^{0.991}\ 10^{-0.01249*Anew}$	41.5	22	(A8.212)
$Q_{200} = 709.7\ DA^{0.684}\ LSLOPE^{1.076}\ 10^{-0.01296*Anew}$	43.8	24	(A8.213)
$Q_{500} = 989.4\ DA^{0.670}\ LSLOPE^{1.177}\ 10^{-0.01347*Anew}$	47.4	25	(A8.214)

Western Coastal Plain

Equation	SE	EY	EQ
$Q_{1.25} = 40.7\ DA^{0.683}\ (IA+1)^{0.366}\ 10^{-0.00849*Anew}$	45.6	2.8	(A8.215)
$Q_{1.5} = 56.3\ DA^{0.671}\ (IA+1)^{0.354}\ 10^{-0.00865*Anew}$	45.3	2.8	(A8.216)
$Q_{2} = 81.3\ DA^{0.656}\ (IA+1)^{0.340}\ 10^{-0.00878*Anew}$	45.9	2.7	(A8.217)
$Q_{5} = 185.5\ DA^{0.622}\ (IA+1)^{0.311}\ 10^{-0.00916*Anew}$	41.2	6.3	(A8.218)
$Q_{10} = 301.4\ DA^{0.607}\ (IA+1)^{0.296}\ 10^{-0.00943*Anew}$	37.3	12	(A8.219)
$Q_{25} = 536.1\ DA^{0.570}\ (IA+1)^{0.275}\ 10^{-0.00954*Anew}$	34.0	21	(A8.220)
$Q_{50} = 791.3\ DA^{0.546}\ (IA+1)^{0.260}\ 10^{-0.00956*Anew}$	33.3	29	(A8.221)
$Q_{100} = 1{,}132.3\ DA^{0.526}\ (IA+1)^{0.247}\ 10^{-0.00957*Anew}$	35.2	32	(A8.222)
$Q_{200} = 1610.4\ DA^{0.501}\ (IA+1)^{0.234}\ 10^{-0.00955*Anew}$	39.8	31	(A8.223)
$Q_{500} = 2523.0\ DA^{0.469}\ (IA+1)^{0.216}\ 10^{-0.00956*Anew}$	49.5	26	(A8.224)

Thomas and Sanchez (2019a, 2019b) equations were revised in 2022 - percent Acond soils based on SSURGO data dated October 2021 using the Dominant Condition approach.

Eastern Coastal Plain

Equation	SE	EY	EQ
$Q_{1.25} = 35.3\ DA^{0.763}\ LSLOPE^{0.120}\ 10^{-0.00815\ Acond}$	46.0	2.8	(A8.225)
$Q_{1.5} = 47.7\ DA^{0.762}\ LSLOPE^{0.195}\ 10^{-0.009\ Acond}$	44.1	3.0	(A8.226)
$Q_{2} = 67.1\ DA^{0.754}\ LSLOPE^{0.276}\ 10^{-0.00921\ Acond}$	42.3	3.2	(A8.227)
$Q_{5} = 135.6\ DA^{0.738}\ LSLOPE^{0.470}\ 10^{-0.01032\ Acond}$	40.0	6.8	(A8.228)
$Q_{10} = 202.4\ DA^{0.726}\ LSLOPE^{0.603}\ 10^{-0.01098\ Acond}$	39.2	10	(A8.229)
$Q_{25} = 321.4\ DA^{0.705}\ LSLOPE^{0.795}\ 10^{-0.01161\ Acond}$	39.6	19	(A8.230)
$Q_{50} = 432.1\ DA^{0.696}\ LSLOPE^{0.898}\ 10^{-0.01214\ Acond}$	40.4	19	(A8.231)
$Q_{100} = 569.0\ DA^{0.687}\ LSLOPE^{0.996}\ 10^{-0.01263\ Acond}$	42.1	21	(A8.232)
$Q_{200} = 736.0\ DA^{0.679}\ LSLOPE^{1.082}\ 10^{-0.01312\ Acond}$	44.4	23	(A8.233)
$Q_{500} = 1033.8\ DA^{0.663}\ LSLOPE^{1.185}\ 10^{-0.01365\ Acond}$	47.9	24	(A8.234)

Western Coastal Plain

Equation	SE	EY	EQ
$Q_{1.25} = 33.0\ DA^{0.709}\ (IA+1)^{0.389}\ 10^{-0.00734\ Acond}$	50.8	2.3	(A8.235)
$Q_{1.5} = 46.7\ DA^{0.696}\ (IA+1)^{0.374}\ 10^{-0.00778\ Acond}$	49.2	2.4	(A8.236)
$Q_{2} = 69.0\ DA^{0.680}\ (IA+1)^{0.357}\ 10^{-0.00819\ Acond}$	48.4	2.5	(A8.237)
$Q_{5} = 164.1\ DA^{0.645}\ (IA+1)^{0.321}\ 10^{-0.00908\ Acond}$	40.8	6.4	(A8.238)
$Q_{10} = 272.0\ DA^{0.630}\ (IA+1)^{0.303}\ 10^{-0.00960\ Acond}$	34.7	13	(A8.239)
$Q_{25} = 493.2\ DA^{0.592}\ (IA+1)^{0.279}\ 10^{-0.00994\ Acond}$	29.2	28	(A8.240)
$Q_{50} = 736.9\ DA^{0.567}\ (IA+1)^{0.262}\ 10^{-0.01010\ Acond}$	27.0	43	(A8.241)
$Q_{100} = 1065.3\ DA^{0.547}\ (IA+1)^{0.248}\ 10^{-0.01022\ Acond}$	28.0	50	(A8.242)
$Q_{200} = 1529.3\ DA^{0.521}\ (IA+1)^{0.234}\ 10^{-0.01030\ Acond}$	32.6	45	(A8.243)
$Q_{500} = 2418.7\ DA^{0.489}\ (IA+1)^{0.215}\ 10^{-0.01041\ Acond}$	42.7	34	(A8.244)

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APPENDIX 9
LINKS TO WEBSITES WITH HYDROLOGIC RESOURCES AND PROGRAMS

Site Name	Website Link	Information
University of Maryland GISHydro	http://www.gishydro.eng.umd.edu	Download software and references for GISHydroNXT and GISHydro2000
NRCS Water Quality and Quantity Technology Development Team	http://go.usa.gov/KoZ	Download NRCS software and technical references: TR-55, TR-20
US Army Corps of Engineers – Hydrologic Engineering Center	http://www.hec.usace.army.mil	Download software and references: HEC-RAS, HEC-HMS
USGS Water Resources – Surface Water Data	http://waterdata.usgs.gov/nwis/sw	Stream gage data and statistics
USGS Water Resources – MD, DE, DC	http://md.water.usgs.gov/ http://water.usgs.gov/md/nwis/sw	Stream gage data and statistics for MD, DE, and DC.
USGS Water Resources – Maps and GIS DataMaps	http://water.usgs.gov/maps.html	Stream gage data and watershed characteristics, GIS format
FHWA Hydraulics Engineering	www.fhwa.dot.gov/engineering/hydraulics/	Hydraulic Engineering Circulars and other references
Maryland State Data Center	https://planning.maryland.gov/MSDC/Pages/s5_map_gis.aspx	Comprehensive plan references and maps
Maryland Department of the Environment – Water Programs	https://mde.state.md.us/programs/Water/Pages/index.aspx	References for Stormwater Management, Flood Hazard Mitigation, Water Quality
Maryland Department of Natural Resources – Guide to Finding DNR Publications	https://dnr.maryland.gov/streams/Pages/publications.aspx	References and publications
U.S. Fish and Wildlife Service, Chesapeake Bay Office – Stream Survey Publications	Coastal Plain: https://www.fws.gov/ChesapeakeBay/PDF/stream-restoration/CoastalPlainweb2.pdf Piedmont: https://pdfs.semanticscholar.org/b198/69a2d63516e3af8a3ab0197224a97d51eaba.pdf Allegheny Plateau/Valley and Ridge: https://www.fws.gov/ChesapeakeBay/PDF/stream-restoration/Plateau.pdf	Maryland stream hydraulic geometry
NRCS Geospatial Data Gateway	http://datagateway.nrcs.usda.gov/	GIS data products including DEMs, land use, stream line work, HUC boundaries, and soil types.
Maryland State Highway Administration	https://roads.maryland.gov/pages/home.aspx	MSHA references and downloads

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