Chapter 6 – Hydrology

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Chapter 6 - Hydrology

6.1 Introduction

6.1.1 Objective

The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes drainage problems or oversized and costs more than necessary. On the other hand, it must be realized that any hydrologic analysis is only an approximation. The relationship between the amount of precipitation on a drainage basin and the amount of runoff from the basin is complex, and too little data are available on the factors influencing the rural and urban rainfall-runoff relationship to expect exact solutions.

6.1.2 Definition

Hydrology is generally defined as a science dealing with water on and under the earth and in the atmosphere. For the purpose of this manual, hydrology will deal with estimating stormwater runoff as the result of rainfall. In design of highway drainage structures, stormwater runoff is usually considered in terms of peak runoff or discharge in cubic feet per second (cfs) and hydrographs as discharge versus time. For structures which are designed to control the volume of runoff, like detention storage facilities, then the entire inflow and outflow hydrographs will be of interest. Wetland hydrology, the water-related driving force to create wetlands, is addressed in the AASHTO Highway Drainage Guidelines, Chapter 10 and the AASHTO Drainage Manual, Chapter 8.[*]

6.1.3 Factors Affecting Floods

In the hydrologic analysis for a drainage structure, it must be recognized that there are many variable factors that affect floods. Some of the factors which need to be recognized and considered on an individual site-by-site basis are things such as:

- Rainfall amount and storm distribution
- Drainage area size, shape, and orientation
- Ground cover
- Type of soil
- Slopes of terrain and stream(s)
- Antecedent moisture condition
- Storage potential (overbank, ponds, wetlands, reservoirs, channels, etc.)
- Watershed development potential
- Type of precipitation (rain, snow, hail, or combinations thereof)

[*] Rev. 1/17

6.1.4 Sources of Information

The type and source of information available for hydrologic analysis will vary from site to site and it is the responsibility of the designer to determine what information is needed and applicable to a particular analysis.

6.2 Design Policy

6.2.1 Introduction

The following sections summarize the policies which should be followed for hydrologic analysis for VDOT roadways. For a more detailed discussion refer to the publications, AASHTO Highway Drainage Guidelines (2007), and the AASHTO Drainage Manual Volumes 1 and 2 (2014).*

6.2.2 Surveys

Hydrologic considerations can significantly influence the selection of a highway corridor and the alternate routes within the corridor. Therefore, studies and investigations should consider the environmental and ecological impact of the project. Also special studies and investigations may be required at sensitive locations. The magnitude and complexity of these studies should be commensurate with the importance and magnitude of the project and problems encountered. Typical data to be included in such surveys or studies are: topographic maps, aerial photographs, streamflow records, historical highwater elevations, flood discharges, and locations of hydraulic features such as reservoirs, water projects, wetlands, karst topography and designated or regulatory floodplain areas.

6.2.3 Flood Hazards

A hydrologic analysis is prerequisite to identifying flood hazard areas and determining those locations at which construction and maintenance will be unusually expensive or hazardous.

6.2.4 Coordination

Since many levels of government plan, design, and construct highway and water resource projects which might have effects on each other, interagency coordination is desirable and often necessary. In addition, agencies can share data and experiences within project areas to assist in the completion of accurate hydrologic analyses.

6.2.5 Documentation

Experience indicates that the design of highway drainage facilities should be adequately documented. Frequently, it is necessary to refer to plans and specifications long after the actual construction has been completed. Thus it is necessary to fully document the results of all hydrologic analysis. Refer to Section 6.5.1 Documentation Requirements, Chapter 3 of this manual and AASHTO Highway Drainage Guidelines Chapter 4 for more details.

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* Rev. 7/16

6.2.6 Evaluation of Runoff Factors

For all hydrologic analyses, the following factors should be evaluated and included when they will have a significant effect on the final results:

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, surface infiltration, and storage
- Stream channel characteristics including: geometry and configuration, slope, hydraulic resistance, natural and artificial controls, channel modification, aggradation, degradation, and ice and debris
- Floodplain characteristics
- Meteorological characteristics such as precipitation amount and type (rain, snow, hail, or combinations thereof), rainfall intensity and pattern, areal distribution of rainfall over the basin, and duration of the storm event

6.2.7 Flood History

All hydrologic analyses should consider the flood history of the area and the effects of these historical floods on existing and proposed structures. The flood history should include the historical floods and the flood history of any existing structures.

6.2.8 Hydrologic Methods

Many hydrologic methods are available. If possible, the selected method should be calibrated to local conditions and verified for accuracy and reliability.

There is no single method for determining peak discharge that is applicable to all watersheds. It is the designer's responsibility to examine all methods that can apply to a particular site and to make the decision as to which is the most appropriate. Consequently, the designer must be familiar with the method sources of the various methods and their applications and limitations. It is not the intent of this manual to serve as a comprehensive text for the various methods of determining peak discharge.

6.2.9 Approved Peak Discharge Methods

In addition to the methods presented in this manual, the following methods are acceptable when appropriately used:

- Log Pearson III analyses of a suitable set of gage data[*] may be used for all routine designs provided there is at least 10 years of continuous or synthesized flow records for 10-yr discharge estimates and 25 years for 100-yr discharge estimates

[*] Rev. 7/16

- Suitable computer programs such as the USACE's HEC-HMS and the NRCS' EFH-2, TR-55 and TR-20 may be used to the[*] hydrologic calculations. The TR-55 method has been found best suited for drainage areas between 200 and 2000 acres (ac). When using any methodology predicated on the 24-hr. rainfall event (i.e. TR-55, TR-20, etc.) it will be necessary to use the values presented in Chapter 11, Appendix 11C-3.
- Other methods may be approved where applicable upon submission to the VDOT State Hydraulics Engineer
- The 100-yr discharges specified in the FEMA flood insurance study are preferred when the analysis includes a proposed crossing on a regulatory floodway. However, if these discharges are deemed to be outdated, the discharges based on current methodology may be used.

6.2.10 Design Frequency

A design frequency should be selected commensurate with the facility cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints as well as the magnitude and risk associated with damages from larger flood events. When long highway routes that have no practical detour are subject to independent flood events, it may be necessary to increase the design frequency at each site to avoid frequent route interruptions from floods. In selecting a design frequency, potential upstream land uses should be considered which could reasonably occur over the anticipated life of the drainage facility.

6.2.11 Economics

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. Section 6.3.1 outlines the design floods that shall be used for different drainage facilities. These frequencies are used to size drainage facilities for an optimum design, which considers both risk of damage and construction cost. Consideration should also be given to the frequency flood that was used to design other structures along a highway corridor.

6.2.12 Review Frequency

All proposed structures designed to accommodate the selected design frequency should be reviewed using a base flood and a check storm of a higher design frequency to ensure that there are no unexpected flood hazards.

[*] Rev. 7/16

6.3 Design Criteria

6.3.1 Design Frequency

6.3.1.1 Factors Governing Frequency Selections

The determination of design factors to be considered and the degree of documentation required depends upon the individual structure and site characteristics. The hydraulic design must be such that risks to traffic, potential property damage, and failure from floods is consistent with good engineering practice and economics. Recognizing that floods cannot be precisely predicted and that it is seldom economically feasible to design for the very rare flood, all designs should be reviewed for the extent of probable damage, should the design flood be exceeded. Design headwater/backwater and flood frequency criteria should be based upon these and other considerations:

- Damage to adjacent property
- Damage to the structure and roadway
- Traffic interruption
- Hazard to human life
- Damage to stream and floodplain environment

The potential damage to adjacent property or inconvenience to owners should be a major concern in the design of all hydraulic structures. The impacts of the 100-yr storm should be evaluated, regardless of the drainage area size.

Inundation of the traveled way indicates the level of traffic service provided by the facility. The traveled way overtopping flood level identifies the limit of serviceability. Table 6-1 relates desired minimum levels of protection from traveled way (edge of shoulder) inundation to the functional classifications of roadways. The design storm discussed here refers to roadway crossing (bridge or culvert) or roadways running parallel to streams. Other features such as storm sewer elements, E&S and SWM facilities will have specific design storms and rainfalls discussed in their respective Chapters.*

6.3.1.2 Minimum Criteria

No exact criteria for flood frequency or allowable backwater/headwater values can be set which will apply to an entire project or roadway classification. Minimum design frequency values relative to protection of the roadway from flooding or damage have been established. It should be emphasized that these values only apply to the level of protection afforded to the roadway.

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* Rev. 7/16

Table 6-1. Design Storm Selection Guidelines
(For Traveled Way Inundation)

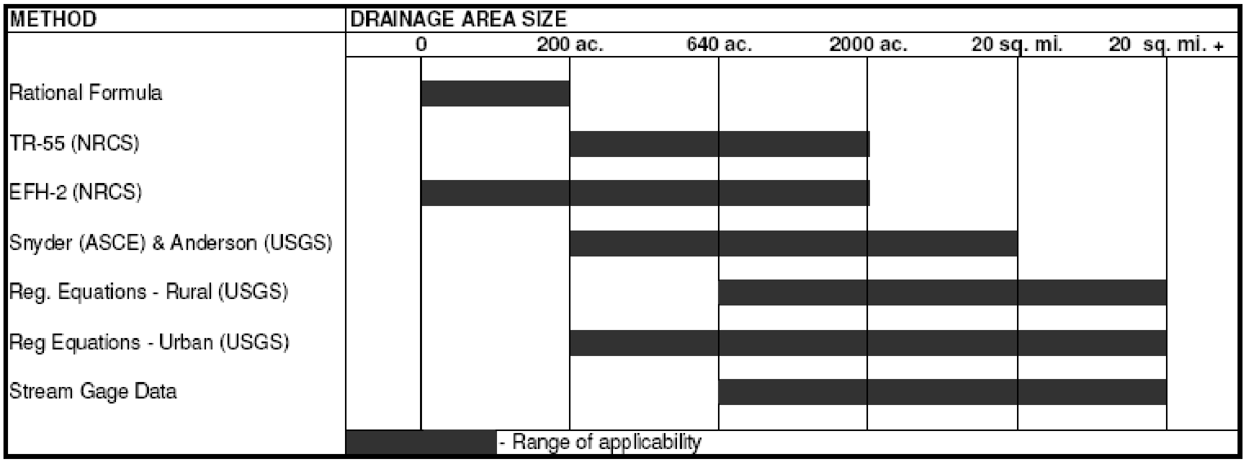
Roadway Classification	Exceedence Probability	Return Period
Rural Principal Arterial System	2%	50-yr
Rural Minor Arterial System	4% - 2%	25 yr - 50-yr
Rural Collector System, Major	4%	25-yr
Rural Collector System, Minor	10%	10-yr
Rural Local Road System	10%	10-yr
Urban Principal Arterial System	4% - 2%	25 yr - 50-yr
Urban Minor Arterial Street System	4%	25-yr
Urban Collector Street System	10%	10-yr
Urban Local Street System	10%	10-yr

Note: Federal law requires interstate highways to be provided with protection from the 2% flood. Facilities such as underpasses and depressed roadways, where no overflow relief is available, shall* also be designed for the 2% event.

* Rev. 7/16

6.3.2 Peak Discharge Method Selection

The methods to be used are shown in Figure 6-1. For watersheds greater than 200 ac, VDOT recommends evaluating several hydrologic methods for comparison purposes.



Note: The above does not indicate definite limits but does suggest a range in which the particular method is “best suited”.

Figure 6-1. Guidelines for Hydrologic Method Selection Based on Drainage Area

6.4 Design Concepts

6.4.1 Travel Time Estimation

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (t_c), which is the time for runoff to travel from the most hydraulically distant point in the watershed to a point of interest within the watershed. The time of concentration is computed by summing all the travel times for consecutive components of the drainage conveyance system.

The computation of travel time and time of concentration is discussed below.

Travel Time

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, pipe flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad (6.1)$$

Where:

T_t = Travel time, hour (hr)
L = Flow length, feet (ft)
V = Average velocity, feet per second (fps)
3600 = Conversion factor from seconds to hours

Time of Concentration

The time of concentration (t_c) is the sum of T_t values for the various consecutive flow segments. Separate flow segments should be computed for overland flow, shallow concentrated flow, channelized flow, and pipe systems.

$$t_c = T_{t1}+T_{t2}+...T_{tm} \qquad (6.2)$$

Where:

t_c = Time of concentration, hours (hrs)*
m = Number of flow segments

* Rev 9/09

Time of concentration is an important variable in most hydrologic methods. Several methods are available for estimating t_c. This chapter presents several methods for estimating overland flow and channel flow times. Any method used should only be used with the parameters given for the specific method. The calculated time should represent a reasonable flow velocity.

For additional information concerning time of concentration as used in the Rational Method, see Section 6.4.4.1.

6.4.1.1 Travel Time in Lakes or Reservoirs

Sometimes it is necessary to compute a t_c for a watershed having a relatively large body of water in the flow path. In such cases, t_c is computed to the upstream end of the lake or reservoir, and for the body of water the travel time is computed using the equation:

[1]

$$V_w = \left(gD_m\right)^{0.5} \tag{6.3}$$

Where:

- V_w = The wave velocity across the water, feet per second (fps)
- g = Acceleration due to gravity = 32.2 ft/s^2
- D_m = Mean depth of lake or reservoir, feet (ft)

Generally, V_w will be high (8 - 30 fps).

[1] From Chapter 15, Part 630, Section 630.1503 of the National Engineering Handbook*

Note that the above equation only provides for estimating travel time across the lake and for the inflow hydrograph to the lake's outlet. It does not account for the travel time involved with the passage of the inflow hydrograph through spillway storage and the reservoir or lake outlet. This time is added to the travel time across the lake. The travel time through lake storage and its outlet can be determined by the storage routing procedures in Chapter 11. The wave velocity Equation 6.3 can be used for swamps with much open water, but where the vegetation or debris is relatively thick (less than about 25% open water), Manning's equation is more appropriate.

For additional discussion of Equation 6.3 and travel time in lakes and reservoirs, see Elementary Mechanics of Fluids, by Hunter Rouse, John Wiley and Sons, Inc., 1946, page 142.

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* Rev. 7/16

6.4.2 Design Frequency

6.4.2.1 Overview

Since it is not economically feasible to design a structure for the maximum runoff a watershed is capable of producing, a design frequency must be established. The frequency with which a given flood can be expected to occur is the reciprocal of the probability, or the chance that the flood will be equaled or exceeded in a given year. If a flood has a 20% chance of being equaled or exceeded each year, over a long period of time, the flood will be equaled or exceeded on an average of once every five years. This is called the return period or recurrence interval (RI). Thus the exceedance probability (percentage) equals $100 \div \mathrm{RI}$.

6.4.2.2 Design Frequency

Roadway Stream Crossings:[*] A drainage facility should be designed to accommodate a discharge with a given return period(s). The design should ensure that the backwater (the headwater) caused by the structure for the design storm does not:

- Increase the flood hazard significantly for property
- Exceed a certain depth on the highway embankment

Based on these design criteria, a design involving roadway overtopping for floods larger than the design event is an acceptable practice. Factors to consider when determining whether roadway overtopping is acceptable are roadway classification, roadway use, impacts and frequency of overtopping, structural integrity, etc. If a culvert or bridge is designed to pass the 25-year flow, it would not be uncommon for a larger event storm (such as the 100-year event) to overtop the roadway. In this scenario, the larger event storm should be used as the "check" or review frequency in the hydraulic analysis. Refer to Chapter 8 for additional details.

Storm Drains: A storm drain should be designed to accommodate a discharge with a given return period(s). The design should be such that the storm runoff does not:

- Increase the flood hazard significantly for property
- Encroach onto the street or highway so as to cause a significant traffic hazard
- Limit traffic, emerging vehicle, or pedestrian movement to an unreasonable extent

Based on these design criteria, a design involving roadway inundation for floods larger than the design event is an acceptable practice. Factors to consider when determining whether roadway inundation is acceptable are roadway classification, roadway use, impacts and frequency of inundation, structural integrity, etc.

[*] Rev. 7/16

6.4.2.3 Review Frequency

After sizing a drainage facility, it will be necessary to review this proposed facility with a base discharge. This is done to ensure that there are no unexpected flood hazards inherent in the proposed facilities. The review flood is usually the 100-yr event. In some cases, a flood event larger than the 100-yr flood is used for analysis to ensure the safety of the drainage structure and nearby development.

6.4.2.4 Rainfall vs. Flood Frequency

Drainage structures are designed based on some flood frequency. However, certain hydrologic procedures use rainfall and rainfall frequency as the basic input. Thus it is commonly assumed that the 10-yr rainfall will produce the 10-yr flood.

6.4.2.5 Intensity-Duration-Frequency (IDF) Values

Rainfall data are available for many geographic areas. From these data, rainfall intensity-duration-frequency (IDF) values can be developed for the commonly used design frequencies using the B, D, & E factors described in Appendix 6C-1 and tabulated in Appendix 6C-2. They are available for mostly[*] every county and major city in the state, and broken down by their respective NOAA Atlas 14 stations. The B, D, & E factors were derived by the Department using the Rainfall Precipitation Frequency data provided by NOAA'S Atlas 14 at the following Internet address: http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html.

6.4.2.6 Discharge Determination

Estimating peak discharges of various recurrence intervals is one of the most common engineering challenges faced by drainage facility designers. The task can be divided into two general categories:

- Gaged sites - the site is at or near a gaging station and the streamflow record is of sufficient length to be used to provide estimates of peak discharges. A complete record is defined as one having at least 25 years of continuous or synthesized data. Ungaged sites - the site is not near a gaging station and no streamflow record is available. This situation is very common and is normal for small drainage areas.

This chapter will address hydrologic procedures that can be used for both categories.

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[*] Rev. 7/16

6.4.3 Peak Discharge Methods

6.4.3.1 Rational Method

6.4.3.1.1 Introduction

The Rational Method is recommended for estimating the design storm peak runoff for areas as large as 200 ac. In low-lying tidewater areas where the terrain is flat, the Rational Method can be considered for areas up to 300 ac. Considerable engineering judgment is required to reflect representative hydrologic characteristics, site conditions, and a reasonable time of concentration (t_c). Its widespread use in the engineering community represents its acceptance as a standard of care in engineering design.

6.4.3.1.2 Application

When applying the Rational Method (and other hydrologic methods), the following items should be considered:

- It is important to obtain a good topographic map and define the boundaries of the drainage area in question. A field inspection of the area should also be made to verify the drainage divides and to determine if the natural drainage divides have been altered.
- In determining the runoff coefficient C-value for the drainage area, the designer should use a comprehensive land use plan for predicting future discharges. Also, the effects of upstream detention facilities may be taken into account.
- Restrictions to the natural flow such as SWM facilities[*] and dams that exist in the drainage area should be investigated to see how they affect the design flows. Only facilities that are designed with the purpose to detain water should be considered.
- Charts, graphs, and tables included in this chapter are not intended to replace reasonable and prudent engineering judgment in the design process.
- The Department considers the Rational Method as the primary approach to hydrologic calculations for the design of closed drainage pipe systems, ditches, channels, culverts, inlets, gutter flow, and any other drainage conveyances other than SWM facilities. Please note that hydrologic methods pertaining to SWM facilities shall follow the guidance as provided in Chapter 11 of this Manual, whereby it is encouraged that the designer employ the use of TR-55/TR-20 methods, involving hydrologic soil group classification and implementation.

[*] Rev. 7/16

6.4.3.1.3 Characteristics

Characteristics of the Rational Method which generally limit its use to 200 acres include:

1. The rate of runoff resulting from any rainfall intensity is a maximum when the rainfall intensity lasts as long as or* longer than the time of concentration. That is, the entire drainage area does not contribute to the peak discharge until the time of concentration has elapsed.

   This assumption limits the size of the drainage basin that can be evaluated by the Rational Method. For large drainage areas, the time of concentration can be so large that constant rainfall intensities for such long periods do not occur and shorter more intense rainfalls can produce larger peak flows.

2. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

   Frequencies of peak discharges depend on rainfall frequencies, antecedent moisture conditions in the watershed, and the response characteristics of the drainage system. For small and largely impervious areas, rainfall frequency is the dominant factor. For larger drainage basins, the response characteristics control. For drainage areas with few impervious surfaces (less urban development), antecedent moisture conditions usually govern, especially for rainfall events with a return period of 10 years or less.

3. The fraction of rainfall that becomes runoff is independent of rainfall intensity or volume.

   The assumption is reasonable for impervious areas, such as streets, rooftops and parking lots. For pervious areas, the fraction of runoff varies with rainfall intensity and the accumulated volume of rainfall. Thus, the art necessary for application of the Rational Method involves the selection of a coefficient that is appropriate for the storm, soil, and land use conditions.

4. The peak rate of runoff is sufficient information for the design.

   Modern drainage practice often includes detention of urban storm runoff to reduce the peak rate of runoff downstream. When a hydrograph is needed for a small drainage area, the Modified Rational Method is normally used. *(See Section 6.4.5.1)*

6.4.3.1.4 Equations

The rational formula estimates the peak rate of runoff at any location in a watershed as a function of the drainage area, runoff coefficient, and mean rainfall intensity for a duration equal to the time of concentration (the time required for water to flow from the most hydraulically remote point of the basin to the point of study).

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* Rev.7/16

The Rational Method Formula is expressed as follows:

$$Q = C_f CiA \tag{6.4}$$

Where:

- Q = Maximum rate of runoff, cubic feet per second (cfs)
- C_f = Saturation factor
- C = Runoff coefficient representing a ratio of runoff to rainfall (dimensionless)
- i = Average rainfall intensity for a duration equal to the time of concentration for a selected return period, inches per hour (in/hr)
- A = Drainage area contributing to the point of study, acres (ac)

Note that conversion to consistent units is not required as 1 acre-inch per hour approximately equals 1 cubic foot/second.

6.4.3.1.5 Infrequent Storm

The coefficients given in Appendix 6E-1 are for storms with less than a 10-year recurrence interval. Less frequent, higher intensity storms will require modification of the coefficient because infiltration and other losses have a proportionally smaller effect on runoff (Wright-McLaughlin 1969). The adjustment of the Rational Method for use with larger storms can be made by multiplying the right side of the Rational Formula by a saturation factor, C_f. The product of C_f and C should not exceed 1.0. Table 6-2 lists the saturation factors for the Rational Method.

Table 6-2. Saturation Factors For Rational Formula

Recurrence Interval (Years)	C_f
2, 5, and 10	1.0
25	1.1
50	1.2
100	1.25

Note: C_f multiplied by C should not exceed 1.0

6.4.3.1.6 Time of Concentration

The time of concentration is the time required for water to flow from the hydraulically most remote point in the drainage area to the point of study. Use of the rational formula requires the time of concentration (t_c) for each design point within the drainage basin. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (i) by using the B, D, & E factors in the procedure described in Appendix 6C-1. A table showing the B, D, & E factors for Virginia counties and larger cities is presented in Appendix 6C-2.

Time of concentration (t_c) for most drainage areas less than about 200 ac will normally be comprised of overland flow (OLF), channel flow or concentrated flow (CF), and conveyance flow in manmade structures. For very small drainage areas such as those draining to drop inlets, the flow time may only consist of overland flow. For very large drainage areas, the overland flow time may not be significant and not be measurable, depending on the scale of the map depicting the drainage area. Overland flow should be limited to about 200'.

Overland Flow
Seelye Method

VDOT experience has determined that the "Overland Flow Time" nomograph developed by E.E. Seelye normally provides a realistic estimate of overland flow (OLF) time when properly applied within the limits shown on the nomograph. Refer to Appendix 6D-1 for the Seelye chart. The Seelye method is the preferred VDOT method for computing overland flow time.

Kinematic Wave Method

The Kinematic Wave Formulation provides an approximation of the rising side of the overland flow hydrograph. The formula is given as:

$$t_c = 0.93\frac{L^{0.6}n^{0.6}}{i^{0.4}{S_o}^{0.3}} \qquad (6.5)$$

Where:

L = Length of strip feet (ft)
n = Manning's roughness coefficient
i = Rainfall intensity (determined iteratively), inches per hour (in/hr)
S_o = Slope, feet/foot (ft/ft)

The determination of the appropriate rainfall intensity with the aid of the Kinematic Wave nomograph (Appendix 6D-2) is an iterative process. Two variables, rainfall intensity and time of concentration, appear in the nomograph and neither are known at the beginning of the computation. Thus, as a first step, a rainfall intensity must be assumed, which is then used in the nomograph to compute a time of concentration. Although this gives a correct solution of the equation, the rainfall intensity associated with the computed time of concentration on an appropriate rainfall - intensity curve may not be consistent with the assumed intensity. If the assumed intensity and that imposed by the frequency curve do not compare favorably, a new rainfall intensity must be assumed and the process repeated.

The kinematic wave method for estimating overland flow time has been determined to be most reliable and is recommended for use with **impervious type surfaces** with n=0.05 or less and a maximum length of 300'. It should be noted that the "n-values" used with the kinematic wave method are applicable only to this method and are for use with very shallow depths of flow such as 0.25". The "n-values" normally associated with channel or ditch flow do not apply to the Kinematic Wave calculations for overland flow time. A chart showing the recommended "n-values" to use with Kinematic Wave method is included as the second page of Appendix 6D-2.

Channel Flow

For channel flow or concentrated flow (CF) time VDOT has found that the nomograph entitled "Time of Concentration of Small Drainage Basins" developed by P.Z. Kirpich provides a reasonable time estimate. Refer to Appendix 6D-5 for the Kirpich nomograph.

When the total time of concentration has been calculated for a point of study (i.e.: culvert) the designer should determine if the calculated t_c is a reasonable estimate for the area under study. The flow length should be divided by the flow time (in seconds) to determine an average velocity of flow. The average velocity can be determined for the overland flow, the channel flow, and the total flow time. If any of the average velocities do not seem reasonable for the specific area of study, they should be checked and revised as needed to provide a reasonable velocity and flow time that will best represent the study area.

6.4.3.1.7 Runoff Coefficients

The runoff coefficient (C) is a variable of the Rational Method that requires significant judgment and understanding on the part of the designer. The coefficient must account for all the factors affecting the relation of peak flow to average rainfall intensity other than area and response time. A range of C-values is typically offered to account for slope, condition of cover, antecedent moisture condition, and other factors that may influence runoff quantities. Good engineering judgment must be used when selecting a C-value for design and peak flow values because a typical coefficient represents the integrated effects of many drainage basin parameters. When available, design and peak flows should be checked against observed flood data. The following discussion considers only the effects of soil groups, land use, and average land slope.

As the slope of the drainage basin increases, the selected C-value should also increase. This is because as the slope of the drainage area increases, the velocity of overland and channel flow will increase, allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area. The lowest range of C-values should be used for flat areas where the majority of grades and slopes are less than 2%. The average range of C-values should be used for intermediate areas where the majority of grades and slopes range from 2 to 5%. The highest range of C-values should be used for steep areas (grades greater than 5%), for cluster areas, and for development in clay soil areas.

It is often desirable to develop a composite runoff coefficient based on the percentage of different surface types in the drainage area. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Appendix 6E-1 shows runoff coefficients for both rural and urban land use conditions. Note that residential C-values exclude impervious area associated with roadways. The roadways need to be accounted for in actual design.

6.4.3.1.8 Common Errors

Two common errors should be avoided when calculating time of concentration (t_c). First, in some cases runoff from a portion of the drainage area that is highly impervious may result in a greater peak discharge than would occur if the entire area were considered. In these cases, adjustments can be made to the drainage area by disregarding those areas where flow time is too slow to add to the peak discharge. Sometimes it is necessary to estimate several different times of concentration to determine the design flow that is critical for a particular application. This is particularly true if a small portion of the drainage area has an unusually high travel time.

Second, when designing a drainage system, the overland flow path is not necessarily perpendicular to the contours shown on available mapping. Often the land will be graded and swales will intercept the natural contour and conduct the water to the streets which reduces the time of concentration. Care should be exercised in selecting overland flow paths in excess of 200’ in urban areas and 400’ in rural areas. The Department recommends a maximum flow length of 300’ to conform to the recommended flow length value in Section 6.4.4.1.6.

6.4.3.2 Anderson Method

6.4.3.2.1 Introduction

The Anderson Method was developed by the United States Geological Service (USGS) in 1968 to evaluate the effects of urban development on floods in Northern Virginia. Further discussion can be found in the publication “Effects of Urban Development on Floods in Northern Virginia” by Daniel G. Anderson, U.S.G.S. Water Resources Division 1968.

One of the advantages of the Anderson Method is that the lag time (T) can be easily calculated for drainage basins that fit the description for one of the three scenarios given:

1. Natural rural basin
2. Developed basin partly channeled or
3. Completely developed and sewered basin.

For basins that are partly developed, there is no direct method provided to calculate lag time. The following explanation of lag time is reproduced from the original report to provide the user with information to properly assess lag time for use in the Anderson Method based upon the parameters used in the study.

6.4.3.2.2 Application

This method was developed from analysis of drainage basins in Northern Virginia with drainage area sizes up to 570 mi^2.

6.4.3.2.3 Characteristics

The difference in flood peak size or magnitude because of drainage system improvement is related to lag time (T). Because lag time will change as a basin undergoes development, an estimate of the lag time for the degree of expected basin development is needed to predict future flood conditions.

Using data for 33 natural and 20 completely sewered basins, relationships were sought to define lag time (T) as function of length and slope. The effectiveness of each relationship was determined on the basis of its standard error of estimate, a measure of its accuracy. Approximately two-thirds of the estimates provided by an equation will be accurate within one standard error, and approximately 19 out of 20 estimates will be accurate within two standard errors. Although equations using log T = f (log L, log S) show a slightly smaller standard error, relations of the form log T = f (log $(L/\sqrt{S})$) were selected as more appropriate for use on the basis of independent work by Snyder (1958) and theoretical considerations.

The ultimate degree of improvement predicted for most drainage systems in the Alexandria-Fairfax area is storm sewering of all small tributaries but with natural larger channels or moderate improvement of larger channels by alignment and rough surfaced banks of rock or grass.

The center relation shown in Table 6-3 provides estimates of lag time for this type of drainage system. The position of the center relation was based upon plotted data for seven basins that are considered to have reached a condition of complete suburban development. The slope of the relation was computed by logarithmic interpolation between the slopes of the relations for natural and completely sewered basins which are also shown in Table 6-3. Data was insufficient to distinguish separate relations for basins with natural or moderately improved larger channels.

It should be noted that the equation for a developed basin partly channelized is for a drainage area with "complete suburban development" and "storm sewering of all small tributaries". The larger channels are either natural or have "moderate improvement". The user is cautioned to use proper engineering judgment in determining lag time for basins that are partly developed and do not fit the parameters used in the equation for developed basin partly channelized.

6.4.3.2.4 Equations

The equation for the Anderson Method is as follows:

$$Q_f = R_f\,(230)KA^{0.82}T^{-0.48} \tag{6.6}$$

Where:

Q_f = Maximum rate of runoff, cubic feet per second (cfs) for flood frequency "f" (i.e. 2.33, 5, 10, 25, 50, & 100). For 500-yr flood multiply calculated Q_{100} by 1.7.
R_f = Flood frequency ratio for Flood frequency "f" based on percentages of imperviousness from 0 to 100% (obtained from formula shown below)
K = Coefficient of imperviousness (obtained from formula shown below)
A = Drainage area, square miles (sq. mi.)
T = Time lag, hours (See Table 6-3)

Table 6-3. Anderson Time Lag Computation

Time Lag, T	Watershed Description
$4.64\left(\frac{L}{\sqrt{S}}\right)^{0.42}$	For natural rural watersheds
$0.90\left(\frac{L}{\sqrt{S}}\right)^{0.50}$	For developed watersheds partially channelized
$0.56\left(\frac{L}{\sqrt{S}}\right)^{0.52}$	For completely developed and sewered watersheds

Where:

L = Length in miles along primary watercourse from site to watershed boundary
S = Index of basin slope in feet per mile based on slope between points 10 and 85% of L

$K=1+0.015\mathbf{I}$

Where:

I = Percentage of imperviousness, in whole numbers (e.g. for 20% imperviousness, use I=20

$$R_f = \frac{R_N + 0.01I(2.5R_{100} - R_N)}{1.00 + 0.0151}$$

Where:

R_N = Flood frequency ratio for 0% imperviousness (i.e. completely rural) for flood frequency “f” (See Table 6-3A)

R_{100} = Flood frequency ratio for 100% imperviousness for flood for flood frequency “f” (See Table 6-3A)

Table 6-3A. Anderson Flood Frequency Ratios

f	2.33	5	10	25	50	100
R_n	1.00	1.65	2.20	3.30	4.40	5.50
R_{100}	1.00	1.24	1.45	1.80	2.00	2.20

Flood frequency ratio for the 5-yr events were derived by VDOT, all others were taken directly from the D.G. Anderson report. Refer to the Design Procedure and Simple Problem, Section 6.5.2.2.1.

6.4.3.3 Snyder Method

6.4.3.3.1 Introduction

The Snyder Method was developed as the “Synthetic Flood Frequency Method” by Franklin F. Snyder. This method was originally presented in the “ASCE Proceedings, Vol. 84 No. HYS) in October 1958.

6.4.3.3.2 Applications

The Snyder Method has been found to produce acceptable results when properly applied to drainage areas between 200 acres and 20 square miles. This method provides the user with an adjustment factor for partly developed basins by the use of percentage factors for the length of channel storm sewered and/or improved.

6.4.3.3.3 Equations

The Snyder Method can be used to determine peak discharges based on runoff, time of concentration, and drainage area. The Snyder Equation can be used for natural basins, partially developed basins, and completely sewered areas. The following is the Snyder Equation:

$$Q_p = 500AI_R \tag{6.7}$$

Where:

Q_p = Peak discharge, cubic feet per second (cfs)
A = Basin area, square miles (sq. mi.)
$I_R = \left[\dfrac{\text{Runoff}}{T_c}\right]$, inches per hour (in/hr)
T_c = Time of concentration, hours (hrs)

6.4.3.4 Rural Regression Method

6.4.3.4.1 Introduction

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Regression studies are statistical practices used to develop runoff equations. These equations are used to relate such things as the peak flow or some other flood characteristic at a specified recurrence interval to the watershed's physiographic, hydrologic and meteorological characteristics.

For details on the application of Rural Regression Equations in Virginia, the user is directed to the following publication: "Peak-Flow Characteristics of Virginia Streams,"[*] U.S.G.S. Scientific Investigations Report 2011-5144 (2011). This report does have some omissions in the standard storm events ranges. VDOT has developed equations based on the USGS data that may be used to supplement the equations in the above report. Tools for using these equations are available on the USGS website Stream Stats and in the VDOT online Hydraulic Applications.

6.4.3.4.2 Application

The regression equations should be used routinely in design for drainage areas greater than one square mile and where stream gage data is unavailable. Where there is stream gage data, the findings from a Log Pearson III (LPIII) method should govern if there is significant variance $\pm$10% from those obtained using the rural regression equations, and provided there is at least 10 years of continuous or synthesized stream gage record. The LPIII results state wide can be found in Table 2 of the above reference. For sites on completely un-gaged watersheds or gaged watersheds where the drainage area at the site is less than 50% of that at the gage or is more than 150% of that at the gage, peak discharges shall be computed using the regressions equations.

For sites on gaged watersheds where the drainage area at the site is equal to or more than 50% of that at the gage or is less than or equal to 150% of that at the gage the gage transposition method described in Section 6.4.4.6 may also be used.

[*] Rev. 7/16

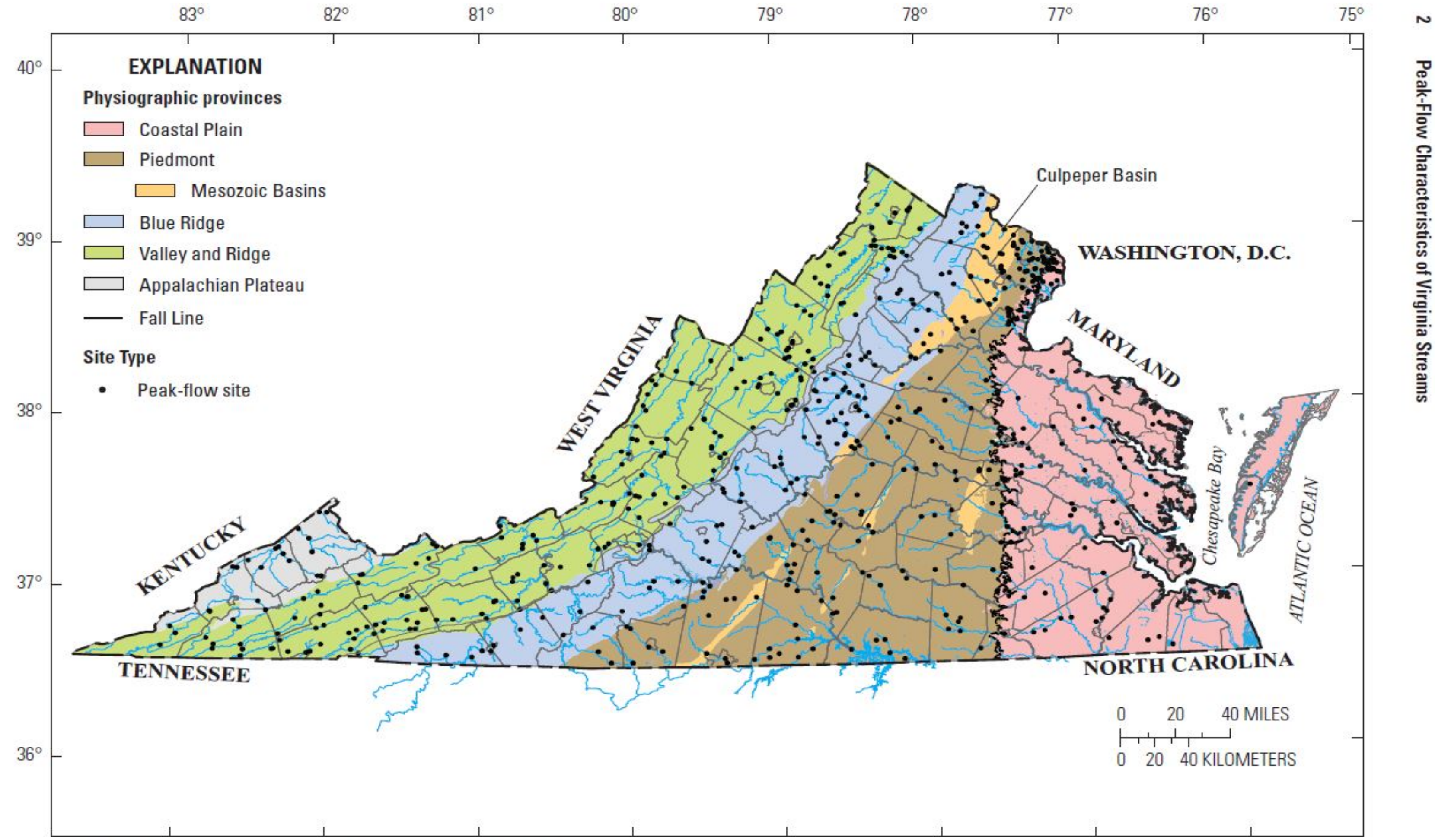
6.4.3.4.3 Hydrologic Regions

The gage data was grouped for the regression analyses based on the five physiographic regions found in Virginia. Each region has distinctive geologic features, landforms and similar runoff characteristics. These regions include: Coastal Plain, Piedmont, Mesozoic Basin, Blue Ridge, Valley and Ridge, and Appalachian Plateau. Figure 6-3 shows the hydrologic regional boundaries for Virginia and can also be seen in the VDOT GIS Intergrator.[*]

6.4.3.4.4 Equations

Table 6-4 contains the drainage-area-only regression equations for estimating peak discharges in Virginia.

[*] Rev. 7/16



Base from U.S. Geological Survey Digital Line Graph, 1:2,000,000, 1987
Adapted physiography from Fenneman and Johnson (1946), 1:7,000,000, and Virginia Geologic Map, Dicken and others (2005), 1:500,000

Figure 1. Selected peak-flow study sites and physiographic provinces for application of peak-flow regional estimating equations.

Figure 6-2. Peak Discharge Regions

Table 6-4. Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia

Basins in the Coastal Plain region	
2-year**	$\text{Log10}(Q_2) = 1.758 + 0.659 \bullet \text{Log10}(DA)$
5-year	$\text{Log10}(Q_5) = 1.918 + 0.644 \bullet \text{Log10}(DA)$
10-year	$\text{Log10}(Q_{10}) = 2.107 + 0.626 \bullet \text{Log10}(DA)$
25-year	$\text{Log10}(Q_{25}) = 2.315 + 0.609 \bullet \text{Log10}(DA)$
50-year	$\text{Log10}(Q_{50}) = 2.457 + 0.594 \bullet \text{Log10}(DA)$
100-year	$\text{Log10}(Q_{100}) = 2.580 + 0.583 \bullet \text{Log10}(DA)$
200-year	$\text{Log10}(Q_{200}) = 2.698 + 0.573 \bullet \text{Log10}(DA)$
500-year**	$\text{Log10}(Q_{500}) = 2.918 + 0.554 \bullet \text{Log10}(DA)$

Basins in the Piedmont region, except those within the Mesozoic Basin region	
2-year	$\text{Log10}(Q_2) = 2.197 + 0.593 \bullet \text{Log10}(DA)$
5-year	$\text{Log10}(Q_5) = 2.540 + 0.551 \bullet \text{Log10}(DA)$
10-year	$\text{Log10}(Q_{10}) = 2.719 + 0.534 \bullet \text{Log10}(DA)$
25-year	$\text{Log10}(Q_{25}) = 2.916 + 0.514 \bullet \text{Log10}(DA)$
50-year	$\text{Log10}(Q_{50}) = 3.043 + 0.501 \bullet \text{Log10}(DA)$
100-year	$\text{Log10}(Q_{100}) = 3.157 + 0.490 \bullet \text{Log10}(DA)$
200-year	$\text{Log10}(Q_{200}) = 3.263 + 0.480 \bullet \text{Log10}(DA)$
500-year**	$\text{Log10}(Q_{500}) = 3.420 + 0.466 \bullet \text{Log10}(DA)$

Basins in the Mesozoic Basin region	
2-year	$\text{Log10}(Q_2) = 2.002 + 0.722 \bullet \text{Log10}(DA)$
5-year	$\text{Log10}(Q_5) = 2.416 + 0.660 \bullet \text{Log10}(DA)$
10-year	$\text{Log10}(Q_{10}) = 2.656 + 0.624 \bullet \text{Log10}(DA)$
25-year	$\text{Log10}(Q_{25}) = 2.923 + 0.586 \bullet \text{Log10}(DA)$
50-year	$\text{Log10}(Q_{50}) = 3.097 + 0.561 \bullet \text{Log10}(DA)$
100-year	$\text{Log10}(Q_{100}) = 3.265 + 0.537 \bullet \text{Log10}(DA)$
200-year	$\text{Log10}(Q_{200}) = 3.401 + 0.521 \bullet \text{Log10}(DA)$
500-year**	$\text{Log10}(Q_{500}) = 3.623 + 0.487 \bullet \text{Log10}(DA)$

Basins in the Blue Ridge region	
2-year	$Log10(Q_2) = 2.127 + 0.709 \bullet Log10(DA)$
5-year	$Log10(Q_5) = 2.490 + 0.668 \bullet Log10(DA)$
10-year	$Log10(Q_{10}) = 2.689 + 0.647 \bullet Log10(DA)$
25-year	$Log10(Q_{25}) = 2.893 + 0.629 \bullet Log10(DA)$
50-year	$Log10(Q_{50}) = 3.030 + 0.616 \bullet Log10(DA)$
100-year	$Log10(Q_{100}) = 3.184 + 0.593 \bullet Log10(DA)$
200-year	$Log10(Q_{200}) = 3.288 + 0.586 \bullet Log10(DA)$
500-year**	$Log10(Q_{500}) = 3.477 + 0.563 \bullet Log10(DA)$

Basins in the Valley and Ridge region	
2-year	$Log10(Q_2) = 2.053 + 0.733 \bullet Log10(DA)$
5-year	$Log10(Q_5) = 2.382 + 0.689 \bullet Log10(DA)$
10-year	$Log10(Q_{10}) = 2.557 + 0.665 \bullet Log10(DA)$
25-year	$Log10(Q_{25}) = 2.741 + 0.642 \bullet Log10(DA)$
50-year	$Log10(Q_{50}) = 2.862 + 0.626 \bullet Log10(DA)$
100-year	$Log10(Q_{100}) = 2.963 + 0.615 \bullet Log10(DA)$
200-year	$Log10(Q_{200}) = 3.063 + 0.603 \bullet Log10(DA)$
500-year**	$Log10(Q_{500}) = 3.208 + 0.588 \bullet Log10(DA)$

Basins in the Appalachian Plateau region	
2-year	$Log10(Q_2) = 1.980 + 0.833 \bullet Log10(DA)$
5-year	$Log10(Q_5) = 2.289 + 0.798 \bullet Log10(DA)$
10-year	$Log10(Q_{10}) = 2.450 + 0.781 \bullet Log10(DA)$
25-year	$Log10(Q_{25}) = 2.631 + 0.759 \bullet Log10(DA)$
50-year	$Log10(Q_{50}) = 2.740 + 0.750 \bullet Log10(DA)$
100-year**	$Log10(Q_{100}) = 2.890 + 0.734 \bullet Log10(DA)$
200-year**	$Log10(Q_{200}) = 3.025 + 0.719 \bullet Log10(DA)$
500-year**	$Log10(Q_{500}) = 3.187 + 0.685 \bullet Log10(DA)$

** Derived by VDOT for use in Roadway Projects for VDOT

6.4.3.4.5 Mixed Population

Mixed population floods are those derived from two (or more) causative factors; e.g., rainfall on a snow pack or hurricane generated floods where convective storm events commonly predominate. To evaluate the effect of such occurrences requires reasonable and prudent judgment.

6.4.3.5 Urban Regression Method

6.4.3.5.1 Introduction

Regression equations developed by the USGS can be found in "Methods and Equations for Estimating Peak Stream Flow per square mile in Virginia's Urban Basins" Scientific Investigations Report 2014-5090 developed specifically for unbanized watersheds in Virginia. It was observed in this study that the urban regression relationship was consistent across the state and was not regionalized.*

6.4.3.5.2 Application

These urban equations may be used for the final hydraulic design of bridges, culverts, and similar structures where such structures are not an integral part of a storm drain system, and provided the contributing watershed either is, or is expected to become, at least 10% urban in nature.

6.4.3.5.3 Characteristics

The methodology as described determines the flow per square mile based upon the percent urbanization in the equations below (URBAN) entered as a % value (ie 20% = 20) and the overall watershed area. The resulting discharge must be multiplied by the watershed area. The percent urbanization made be estimated or determined by the methods available through the USGS Stream Stats website to compute basin parameter LC11DEV.

* Rev. 7/16

6.4.3.5.4 Equations

The equations for urban conditions take the following general form:

$$Log_{10}(q) = \beta_0 + (URBAN - \beta_1) \times \left((Log_{10}(A) - \beta_2) \times \beta_3\right) + URBAN \times \beta_4 + Log_{10}(A) \times \beta_5$$

$$q = 10^{Log_{10}(q)}$$

$$Q = q * A$$

Where:

q = Unit discharge per square mile (cfs/mi^2)*
Q = Total Discharge (cfs)
Urban = Percent Urbanization 10-100 (dimensionless)
A = Contributing drainage area, square miles (mi^2)
$\beta_0 - \beta_5$ = Regression constants a given return period

Event	β_0	β_1	β_2	β_3	β_4	β_5
2-year	2.027	40.290	1.216	-0.00414	0.00468	-0.366
5-year	2.229	39.370	1.139	-0.00346	0.00487	-0.338
10-year	2.373	38.706	1.103	-0.00313	0.00470	-0.334
25-year	2.557	39.168	1.083	-0.00224	0.00434	-0.332
50-year	2.697	39.168	1.083	-0.00219	0.00390	-0.343
100-year	2.776	38.765	1.070	-0.00242	0.00434	-0.342
200-year	2.863	39.063	1.057	-0.00223	0.00465	-0.329
500-year	2.961	39.287	0.904	-0.00049	0.00636	-0.317

6.4.3.6 Stream Gage Data

6.4.3.6.1 Introduction

Many gauging stations exist throughout Virginia where data can be obtained and used for hydrologic studies. If a project is located near one of these gages and the gaging record is of sufficient length in time, a frequency analysis may be made according to the following discussion. The most important aspect of applicable station records is the series of annual peak discharges. It is possible to apply a frequency analysis to that data for the derivation of flood-frequency curves. Such curves can then be used in several different ways.

- If the subject site is at or very near the gaging site and on the same stream and watershed, the discharge for a specific frequency from the flood-frequency curve can be used directly.

* Rev. 7/16

- If the facility site is up or downstream of the gaging site or on a nearby or representative watershed with similar hydrologic characteristics, transposition of frequency discharges is possible, provided the watershed area at the facility site is no less than 1/2 nor more than 1.5 times the watershed area at the gaging site.
- If the flood-frequency curve is from one of a group of several gaging stations comprising a hydrologic region, then regional regression relations may be derived. Regional regression relations are usually furnished by established hydrologic agencies and the designer will not be involved in their development.

The Log Pearson Type III frequency distribution will be used to estimate flood frequency in this manual.

6.4.3.6.2 Application

The stream gage analysis findings may be used for design when there are sufficient years of measured or synthesized stream gage data. The Log Pearson Type III method data is available in the "Peak-Flow Characteristics of Virginia Streams," U.S.G.S. Scientific Investigations Report 2011-5144 (2011).* The U.S. Geological Survey has developed a computer program entitled "PEAKFQWIN" for performing Log Pearson Type III computations. Gage data may be obtained from the USGS, Transposition of Data.

6.4.3.6.3 Transposition of Data

The transposition of design discharges from one basin to another basin with similar hydrologic characteristics is accomplished by multiplying the design discharge by the direct ratio of the respective drainage areas raised to the power shown in Table 6-7. Thus on streams where no gaging station is in existence, records of gaging stations in one or more nearby hydrologically similar watersheds may be used. The discharge for such an ungaged stream may be determined by the transposition of records using a similar procedure. This procedure is repeated for each available nearby watershed and the results are averaged to obtain a value for the desired flood frequency relationships in the ungaged watershed.

Table 6-7. Transposition of Data Sample Problem

Watershed	Q_{25}, cfs	Area, sq. mi
Gaged Watershed A	4,100	42.1
Gaged Watershed B	7,2000	79.6
Gaged Watershed C	12,000	124
Ungaged Watershed D	Find Q_{25}	83.0

* Rev. 7/16

Adjust Q_{25} for each subshed by area ratio:

A: $4{,}100 \times \left(\frac{83.0}{42.1}\right)^{0.8} = 7{,}057$

B: $7{,}200 \times \left(\frac{83.0}{79.6}\right)^{0.8} = 7{,}445$

C: $12{,}000 \times \left(\frac{83.0}{124}\right)^{0.8} = 8{,}704$

Average the Q_{25} for subsheds A, B, and C to Obtain Q_{25} for subshed D:

D: $Q_{25} = \frac{(7{,}057 + 7{,}445 + 8{,}704)}{3} = 7{,}735\ use\ 7{,}700\ cfs$

6.4.4 Hydrograph Methods

6.4.4.1 Modified Rational Method

6.4.4.1.1 Introduction

The Modified Rational Method provides hydrographs for small drainage areas where the peak, Q, is normally calculated by the Rational Method.

6.4.4.1.2 Application

Hydrographs produced by the Modified Rational Method can be used for the analysis and design of stormwater management (SWM) basins, temporary sediment basins, or other applications needing a hydrograph for a drainage area of less than 200 ac.

6.4.4.1.3 Characteristics

Hydrographs developed by the Modified Rational Method are based upon different duration storms of the same frequency and have the following parameters:

- Time of concentration (t_c) = Time to peak (T_p)
- Time to recede (T_r) = T_p
- The duration, D_e, of the storm is from 0 minutes until the time of selected duration
- Base of hydrograph (T_b) = D_e + T_r
- The peak Q (top of trapezoidal hydrograph) is calculated using the intensity (I) value predicated on the “B, D, & E” factors (Appendix 6C-2) for the selected duration and frequency.
- Hydrographs are normally calculated for durations of:

  1. t_c
  2. $1.5t_c$
  3. $2t_c$
  4. $3t_c$

- Longer duration hydrographs may need to be calculated if reservoir routing computations show that the ponded depth in a basin is increasing with each successive hydrograph that is routed through the basin.

Hydrographs with durations less than t_c are not valid and should not be calculated.

The Modified Rational Method recognizes that the duration of a storm can and will sometimes be longer than the time of concentration. This longer duration storm, even though it produces a lower peak Q, can produce a larger volume of runoff than the storm duration equal to the actual time of concentration of the drainage area. In order to ensure the proper design of stormwater management basins, the volume of runoff for the critical storm duration should be calculated.

6.4.4.1.4 Critical Storm Duration

The storm duration that produces the greatest volume of storage and highest ponded depth within a basin is considered the critical duration storm (T_c). Reservoir routing computations for the basin will need to incorporate several different duration storms in order to determine the critical duration and the highest pond level for each frequency storm required. The operation of any basin is dependent on the interaction of:

- Inflow (hydrograph)
- Storage characteristics of the basin
- Performance of the outlet control structure

Therefore, each basin will respond to different duration storms in dissimilar patterns. The approximate critical storm can be estimated but the actual critical duration storm can only be determined by performing reservoir routing computations for several different duration storms.

6.4.4.1.5 Estimating the Critical Duration Storm

The Virginia Department of Conservation and Recreation (DCR) has developed a method to estimate the critical duration storm. The following items should be taken into consideration when using this method:

- For estimation only
- May provide a critical storm duration which is less than t_c, this is not valid
- Does not work well when t_c is decreased only slightly by development
- Does not work well when the peak Q is not significantly increased by development
- The a and b factors for equation 6.9 are listed in Chapter 11, Appendix 11 H-2 and are to be used for no other purpose

For further explanation see Chapter 11, section 11.5.4.2.

6.4.4.1.6 Equations

The approximate length of the critical storm duration can be estimated by the following equation:

$$T_c = \sqrt{\frac{2CAa(b-\frac{t_c}{4})}{q_o}}-b \tag{6.9}$$

Where:

T_c = Critical storm duration, minute (min)
C = Rational coefficient for developed area
A = Drainage area, acres (ac)
t_c = Time of concentration after development, minute (min)
q_o = Allowable peak outflow, cubic feet per second (cfs)
a & b = Rainfall regression constants, Appendix 11 H-2

6.4.4.2 SCS Unit Hydrograph

6.4.4.2.1 Introduction

Techniques developed by the former United States Department of Agriculture, Soil Conservation Service (SCS) for calculating rates of runoff require the same basic data as the Rational Method: drainage area, a runoff factor, time of concentration, and rainfall. The SCS has been renamed the National Resources Conservation Service or NRCS. Because this method has been traditionally called the SCS method, this manual will continue to use this terminology. The SCS approach, however, also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or synthetic, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. Details of the methodology can be found in the SCS National Engineering Handbook, Part 630 - Hydrology.

6.4.4.2.2 Application

Two types of hydrographs are used in the SCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from one-inch of direct runoff occurring over the watershed in a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in nondimensional units of time versus time to peak and discharge at any time versus peak discharge.

6.4.4.2.3 Characteristics

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

6.4.4.2.4 Time of Concentration

The average slope within the watershed together with the overall length and retardance of overland flow are the major factors affecting the runoff rate through the watershed. VDOT recommends using the Rational Method procedures to calculate time of concentration (t_c). Lag time (L) can be considered as a weighted time of concentration and is related to the physical properties of a watershed, such as area, length and slope. The SCS derived the following empirical relationship between lag time and time of concentration:

$$L = 0.6t_c \qquad (6.10)$$

6.4.4.2.5 Curve Numbers

In hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall - all defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rain water. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope.

Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use. Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The SCS uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen. The higher the CN, the higher is the runoff potential. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). Soil type A has the highest infiltration and soil type D has the least amount of infiltration. Soil surveys are available from the NRCS website at http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm, or your local NRCS office at:

Virginia FSA, NRCS & RD State Offices
1606 Santa Rosa Road, Suite 209
Richmond, VA 23229-5014
Phone: 804-287-1500

Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected. Also runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm. A five (5) day period is used as the minimum for estimating antecedent moisture conditions. Antecedent soil moisture conditions also vary during a storm; heavy rain falling on a dry soil can change the soil moisture condition from dry to average to wet during the storm period.

6.4.4.2.6 Equations

The following discussion outlines the equations and basic concepts utilized in the SCS method.

Drainage Area - The drainage area of a watershed is determined from topographic maps and field surveys. For large drainage areas it might be necessary to divide the area into sub-drainage areas to account for major land use changes, obtain analysis results at different points within the drainage area, or locate stormwater drainage facilities and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems should also be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub-drainage areas.

Rainfall - The rainfall employed in the NRCS method (the variable “P” in equation 6.11) for both duration and frequency may be obtained directly from NOAA’s Precipitation Frequency Data Server (based on their ATLAS-14 publication) at the following Internet address: http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html. When the opening screen appears be sure to choose “Data Type:” as “Precipitation Depth” from the pull-down options menu.

Rainfall-Runoff Equation - A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures, such as contouring and terracing, from experimental watersheds were included. (The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included the total amount of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall). The equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad (6.11)$$

Where:

Q = Direct runoff, inches (in)
P = Precipitation, inches (in)
I_a = Initial abstractions, inches (in)

$$I_a = 0.2S \qquad (6.12)$$

S = Potential maximum retention after runoff begins, inches (in)

$$S = \frac{1000}{CN} - 10 \qquad (6.13)$$

CN = SCS Runoff curve number

The Virginia office of the NRCS has recently advised that the NOAA ATLAS-14 rainfall data does not, in many instances, follow the current Type II and Type III temporal distribution curves. They indicate that the Type II curve, will only give reasonable results for return interval (frequency) storm events up to and including a 10-year event and should be used with caution. They have advised that the soon to be released revised “TR-20” software package will provide a routine that will convert the ATLAS-14 rainfall data from NOAA’s Precipitation Frequency Data Server to county-specific temporal distribution curves. Their “TR-55” and “EFH-2” software packages will ultimately contain this same feature. The NRCS Virginia office has indicated that additional information on this issue will be posted on their web site as it becomes available.

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Appendix 6B-1 Runoff Depth for Runoff Curve Number (RCN)

Runoff depth for selected NRCS TR-55 CN's and rainfall amounts*

Rainfall (inches)	Runoff depth (in inches) for Curve Number (CN) of -												
	40	45	50	55	60	65	70	75	80	85	90	95	98
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.08	0.17	0.32	0.56	0.79
1.2	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.07	0.15	0.27	0.46	0.74	0.99
1.4	0.00	0.00	0.00	0.00	0.00	0.02	0.06	0.13	0.24	0.39	0.61	0.92	1.18
1.6	0.00	0.00	0.00	0.00	0.01	0.05	0.11	0.20	0.34	0.52	0.76	1.11	1.38
1.8	0.00	0.00	0.00	0.00	0.03	0.09	0.17	0.29	0.44	0.65	0.93	1.29	1.58
2.0	0.00	0.00	0.00	0.02	0.06	0.14	0.24	0.38	0.56	0.80	1.09	1.48	1.77
2.5	0.00	0.00	0.02	0.08	0.17	0.30	0.46	0.65	0.89	1.18	1.53	1.96	2.27
3.0	0.00	0.02	0.09	0.19	0.33	0.51	0.71	0.96	1.25	1.59	1.98	2.45	2.77
3.5	0.02	0.08	0.20	0.35	0.53	0.75	1.01	1.30	1.64	2.02	2.45	2.94	3.27
4.0	0.06	0.18	0.33	0.53	0.76	1.03	1.33	1.67	2.04	2.46	2.92	3.43	3.77
4.5	0.14	0.30	0.50	0.74	1.02	1.33	1.67	2.05	2.46	2.91	3.40	3.92	4.26
5.0	0.24	0.44	0.69	0.98	1.30	1.65	2.04	2.45	2.89	3.37	3.88	4.42	4.76
6.0	0.50	0.80	1.14	1.52	1.92	2.35	2.81	3.28	3.78	4.30	4.85	5.41	5.76
7.0	0.84	1.24	1.68	2.12	2.60	3.10	3.62	4.15	4.69	5.25	5.82	6.41	6.76
8.0	1.25	1.74	2.25	2.78	3.33	3.89	4.46	5.04	5.63	6.21	6.81	7.40	7.76
9.0	1.71	2.29	2.88	3.49	4.10	4.72	5.33	5.95	6.57	7.18	7.79	8.40	8.76
10.0	2.23	2.89	3.56	4.23	4.90	5.56	6.22	6.88	7.52	8.16	8.78	9.40	9.76
11.0	2.78	3.52	4.26	5.00	5.72	6.43	7.13	7.81	8.48	9.13	9.77	10.39	10.76
12.0	3.38	4.19	5.00	5.79	6.56	7.32	8.05	8.76	9.45	10.11	10.76	11.39	11.76
13.0	4.00	4.89	5.76	6.61	7.42	8.21	8.98	9.71	10.42	11.10	11.76	12.39	12.76
14.0	4.65	5.62	6.55	7.44	8.30	9.12	9.91	10.67	11.39	12.08	12.75	13.39	13.76
15.0	5.33	6.36	7.35	8.29	9.19	10.04	10.85	11.63	12.37	13.07	13.74	14.39	14.76

*Interpolate the values shown to obtain runoff depths for CN's or rainfall amounts not shown.

Source: SCS TR-55

APPENDIX 6B-02 24-HR. RAINFALL DEPTHS (INCHES)

APPENDIX 11C-3 24-HOUR RAINFALL DEPTH (INCHES)

County	Frequency (Years)							
	1	**2**	**5**	**10**	**25**	**50**	**100**	**500**
Accomack	2.67	3.25	4.22	5.07	6.35	7.48	8.75	12.36
Albemarle (Zone 1)	3.40	4.12	5.24	6.18	7.56	8.74	10.06	13.79
Albemarle (Zone 2)	2.99	3.62	4.63	5.47	6.71	7.78	8.96	12.19
Alleghany	2.35	2.83	3.56	4.17	5.04	5.76	6.54	8.62
Amelia	2.73	3.30	4.22	5.00	6.15	7.13	8.21	11.17
Amherst	2.82	3.42	4.35	5.13	6.27	7.24	8.29	11.14
Appomattox	2.83	3.43	4.38	5.18	6.36	7.37	8.48	11.50
Augusta (Zone 1)	2.44	2.95	3.73	4.38	5.31	6.09	6.92	9.09
Augusta (Zone 2)	2.80	3.38	4.29	5.05	6.15	7.08	8.10	10.84
Bath	2.46	2.96	3.72	4.35	5.25	6.00	6.80	8.90
Bedford (Zone 1)	3.11	3.78	4.82	5.69	6.96	8.05	9.25	12.62
Bedford (Zone 2)	2.75	3.34	4.26	5.03	6.16	7.13	8.19	11.07
Bland	2.18	2.59	3.15	3.59	4.20	4.69	5.19	6.42
Botetourt	2.63	3.19	4.05	4.76	5.79	6.66	7.61	10.18
Brunswick	2.78	3.38	4.35	5.16	6.32	7.29	8.33	11.10
Buchanan	2.17	2.59	3.17	3.65	4.34	4.91	5.52	7.11
Buckingham	2.78	3.36	4.30	5.09	6.26	7.26	8.36	11.37
Campbell	2.74	3.32	4.25	5.03	6.18	7.17	8.25	11.20
Caroline	2.68	3.25	4.19	5.01	6.25	7.34	8.57	12.06
Carroll (Zone 1)	2.26	2.73	3.44	4.01	4.80	5.45	6.12	7.80
Carroll (Zone 2)	2.63	3.19	4.04	4.72	5.67	6.46	7.29	9.43
Carroll (Zone 3)	2.95	3.57	4.55	5.34	6.49	7.45	8.48	11.27
Carroll (Zone 4)	3.35	4.07	5.19	6.12	7.47	8.62	9.88	13.36
Charles City	2.81	3.41	4.39	5.23	6.48	7.56	8.75	12.05
Charlotte	2.71	3.28	4.19	4.96	6.10	7.07	8.14	11.07
Chesapeake (city)	3.03	3.68	4.75	5.67	7.01	8.16	9.44	12.94
Chesterfield	2.77	3.35	4.29	5.09	6.27	7.28	8.39	11.44
Clarke	2.40	2.89	3.63	4.25	5.16	5.92	6.75	8.97
Craig	2.39	2.88	3.63	4.24	5.12	5.86	6.64	8.73
Culpeper	2.70	3.27	4.18	4.97	6.18	7.23	8.42	11.79
Cumberland	2.71	3.27	4.18	4.95	6.09	7.06	8.13	11.05
Dickenson	2.21	2.63	3.22	3.72	4.44	5.04	5.69	7.41
Dinwiddie	2.80	3.39	4.35	5.15	6.31	7.30	8.37	11.23
Essex	2.67	3.24	4.20	5.03	6.29	7.39	8.63	12.16
Fairfax	2.57	3.11	3.99	4.78	5.97	7.04	8.24	11.72
Fauquier	2.63	3.17	4.03	4.77	5.89	6.86	7.95	11.05
Floyd (Zone 1)	2.52	3.06	3.89	4.58	5.58	6.42	7.33	9.73

Floyd (Zone 2)	2.87	3.47	4.43	5.22	6.37	7.35	8.41	11.31
Floyd (Zone 3)	3.39	4.12	5.26	6.21	7.62	8.83	10.15	13.85
Floyd (Zone 4)	3.81	4.63	5.93	7.01	8.60	9.97	11.50	15.82
Fluvanna	2.69	3.25	4.15	4.91	6.04	7.00	8.05	10.92
Franklin	2.83	3.43	4.37	5.16	6.32	7.31	8.39	11.33
Frederick	2.36	2.83	3.53	4.11	4.96	5.67	6.45	8.53
Giles (Zone 1)	2.11	2.53	3.14	3.63	4.34	4.92	5.54	7.15
Giles (Zone 2)	2.30	2.78	3.49	4.07	4.90	5.60	6.35	8.37
Gloucester	2.86	3.48	4.51	5.40	6.73	7.89	9.18	12.83
Goochland	2.71	3.28	4.20	4.97	6.12	7.11	8.19	11.17
Grayson (Zone 1)	3.25	3.91	4.88	5.66	6.78	7.72	8.74	11.54
Grayson (Zone 2)	2.37	2.85	3.56	4.13	4.91	5.55	6.20	7.83
Grayson (Zone 3)	2.66	3.22	4.06	4.73	5.67	6.45	7.26	9.35
Greene	3.03	3.67	4.67	5.50	6.73	7.78	8.94	12.13
Greensville	2.72	3.29	4.24	5.04	6.21	7.20	8.28	11.21
Halifax	2.69	3.25	4.13	4.87	5.95	6.86	7.85	10.51
Hampton (city)	2.94	3.58	4.63	5.53	6.88	8.05	9.34	12.96
Hanover	2.71	3.28	4.20	5.00	6.20	7.25	8.42	11.70
Henrico	2.74	3.32	4.25	5.05	6.25	7.28	8.42	11.59
Henry	2.89	3.50	4.46	5.28	6.49	7.52	8.65	11.72
Highland	2.42	2.90	3.60	4.18	5.00	5.69	6.41	8.29
Isle of Wight	2.96	3.60	4.65	5.53	6.84	7.96	9.19	12.57
James City	2.91	3.54	4.57	5.45	6.76	7.89	9.14	12.62
King and Queen	2.72	3.31	4.28	5.11	6.38	7.49	8.72	12.22
King George	2.62	3.19	4.13	4.95	6.19	7.29	8.53	12.06
King William	2.69	3.27	4.22	5.04	6.29	7.37	8.58	12.00
Lancaster	2.74	3.33	4.32	5.18	6.49	7.63	8.91	12.56
Lee	2.54	3.03	3.69	4.23	5.01	5.65	6.34	8.14
Loudoun	2.58	3.11	3.95	4.67	5.74	6.67	7.71	10.64
Louisa	2.73	3.31	4.24	5.02	6.18	7.18	8.29	11.32
Lunenburg	2.72	3.29	4.21	4.98	6.12	7.09	8.15	11.03
Lynchburg (city)	2.76	3.34	4.26	5.04	6.18	7.15	8.21	11.11
Madison (Zone 1)	3.38	4.09	5.20	6.13	7.48	8.65	9.93	13.47
Madison (Zone 2)	2.89	3.50	4.47	5.28	6.48	7.50	8.64	11.75
Mathews	2.83	3.44	4.47	5.35	6.69	7.86	9.17	12.88
Mecklenburg	2.67	3.23	4.12	4.85	5.93	6.84	7.83	10.47
Middlesex	2.78	3.38	4.38	5.25	6.55	7.68	8.96	12.55
Montgomery (Zone 1)	1.98	2.39	3.03	3.55	4.30	4.92	5.59	7.29
Montgomery (Zone 2)	2.27	2.75	3.50	4.11	4.99	5.73	6.52	8.61
Montgomery (Zone 3)	2.60	3.15	4.01	4.72	5.75	6.61	7.55	10.07
Nelson	2.98	3.61	4.60	5.43	6.64	7.67	8.80	11.87

New Kent	2.77	3.37	4.35	5.19	6.44	7.53	8.74	12.11
Newport News (city)	2.94	3.58	4.63	5.53	6.86	8.01	9.29	12.83
Norfolk (city)	2.94	3.58	4.62	5.51	6.82	7.96	9.20	12.64
Northampton	2.74	3.33	4.33	5.19	6.48	7.62	8.89	12.50
Northumberland	2.69	3.27	4.25	5.10	6.38	7.51	8.79	12.41
Nottoway	2.74	3.31	4.23	5.01	6.15	7.12	8.19	11.10
Orange	2.75	3.33	4.26	5.06	6.26	7.30	8.46	11.68
Page (Zone 1)	2.43	2.93	3.70	4.35	5.29	6.08	6.93	9.17
Page (Zone 2)	3.01	3.64	4.62	5.43	6.61	7.62	8.71	11.65
Patrick (Zone 1)	3.79	4.60	5.89	6.97	8.56	9.93	11.46	15.77
Patrick (Zone 2)	3.32	4.03	5.15	6.10	7.49	8.67	9.98	13.59
Patrick (Zone 3)	3.04	3.68	4.70	5.57	6.84	7.93	9.12	12.38
Petersburg (city)	2.80	3.39	4.35	5.16	6.34	7.35	8.46	11.45
Pittsylvania	2.77	3.36	4.28	5.06	6.21	7.18	8.25	11.15
Poquoson (city)	2.92	3.56	4.61	5.51	6.86	8.04	9.35	13.01
Portsmouth (city)	2.96	3.61	4.66	5.55	6.88	8.01	9.27	12.72
Powhatan	2.71	3.28	4.20	4.97	6.12	7.11	8.19	11.17
Prince Edward	2.74	3.32	4.25	5.03	6.18	7.17	8.26	11.23
Prince George	2.81	3.41	4.38	5.21	6.41	7.45	8.57	11.64
Prince William	2.50	3.02	3.89	4.65	5.82	6.85	8.01	11.35
Pulaski	2.01	2.43	3.08	3.60	4.37	4.99	5.66	7.36
Rappahannock	2.77	3.36	4.27	5.03	6.15	7.12	8.17	11.06
Richmond (city)	2.76	3.35	4.29	5.09	6.28	7.30	8.43	11.53
Richmond	2.70	3.29	4.26	5.11	6.40	7.53	8.80	12.41
Roanoke (Zone 1)	2.33	2.83	3.59	4.21	5.12	5.88	6.69	8.83
Roanoke (Zone 2)	2.61	3.17	4.03	4.74	5.77	6.64	7.57	10.08
Rockbridge	2.50	3.03	3.85	4.52	5.49	6.30	7.17	9.45
Rockingham (Zone 1)	2.32	2.79	3.50	4.09	4.94	5.64	6.40	8.35
Rockingham (Zone 2)	2.86	3.44	4.35	5.10	6.20	7.12	8.14	10.92
Russell	2.18	2.60	3.15	3.60	4.25	4.79	5.36	6.84
Scott (Zone 1)	2.38	2.83	3.42	3.91	4.61	5.18	5.79	7.38
Scott (Zone 2)	2.27	2.69	3.20	3.61	4.15	4.59	5.03	6.12
Shenandoah	2.31	2.79	3.49	4.08	4.94	5.67	6.46	8.57
Smyth	2.27	2.70	3.23	3.65	4.21	4.66	5.10	6.13
Southampton	2.88	3.50	4.51	5.36	6.60	7.66	8.82	11.97
Spotsylvania	2.67	3.23	4.15	4.94	6.14	7.20	8.38	11.71
Stafford	2.56	3.10	4.00	4.78	5.98	7.04	8.24	11.66
Suffolk (city)	2.99	3.64	4.70	5.59	6.91	8.04	9.28	12.69
Surry	2.90	3.52	4.55	5.42	6.70	7.80	9.02	12.36
Sussex	2.85	3.46	4.46	5.29	6.50	7.52	8.62	11.58
Tazewell	2.12	2.52	3.04	3.48	4.09	4.59	5.11	6.43

Virginia Beach (city)	3.02	3.67	4.74	5.65	6.99	8.15	9.42	12.94
Warren (Zone 1)	2.49	3.00	3.79	4.45	5.40	6.22	7.10	9.48
Warren (Zone 2)	2.84	3.43	4.34	5.11	6.24	7.22	8.30	11.30
Washington	2.16	2.56	3.06	3.46	3.99	4.41	4.83	5.79
Westmoreland	2.66	3.24	4.20	5.03	6.30	7.41	8.67	12.23
Wise	2.27	2.71	3.32	3.83	4.57	5.19	5.86	7.64
Wythe	2.06	2.47	3.07	3.55	4.20	4.73	5.26	6.55
York	2.93	3.56	4.61	5.50	6.84	8.00	9.29	12.87

Source: National Resource Conservation Service, Richmond, Va. office – Based on their implementation of NOAA's ATLAS-14 rainfall data

Note: Maps are available showing the zone boundaries for counties with multiple rainfall zones at the following NOAA web site:
http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_maps.html

Appendix 6C-1 B, D, and E Factors - Application

B, D and E Factors that Define Intensity-Duration-Frequency (IDF) Values* for Use with the Rational Method and the Modified Rational Method

The rainfall IDF values are described by the equation:

$$i = \frac{B}{(t_c + D)^E}$$

Where:

i = Intensity, inches per hour (in/hr)
t_c = Time of concentration, minutes (min)

The B, D and E factors for all counties and major cities have been tabulated in Appendix 6C-2. These values were derived by the Department using the Rainfall Precipitation Frequency data provided by NOAA's "Atlas 14" at the following Internet address: http://hdsc.nws.noaa.gov/hdsc/pfds/orb/va_pfds.html. A Microsoft EXCEL spreadsheet containing all the B, D, and E factors for the state of Virginia as shown in Appendix 6C-2 is available upon request or may by downloaded at the following Internet address: http://www.virginiadot.org/business/resources/LocDes/BDE_2016.xlsx .

It should be noted, since the regression procedure used to derive these values was predicated on 5 and 60 minute storm durations, that the accuracy of the calculations performed using these values decreases significantly for times of concentration in excess of 60 minutes and the error becomes greater as the time increases. For long storm durations and/or long times of concentration, the rainfall intensity and/or total point rainfall should be obtained directly from NOAA'S Precipitation Frequency Data Server at the Internet address shown above.

An example problem employing the above equation is shown below.

Given: Chesterfield County, Storm Duration (t_c) = 30 minutes

Find: 10-yr. frequency rainfall intensity

* Rev 7/09

$i_{10} = B / (t_c + D)^E = 50.71 / (30 + 10.00)^{0.73} = 3.43$ in/hr

It should be noted that the above procedure could also be used for applications employing time of concentration (t_c) in hours and total rainfall (as opposed to rainfall intensity) in inches. It is merely necessary to multiply the calculated rainfall intensity (based on a t_c in minutes) by the time of concentration (in hours) to determine the total point rainfall.

Appendix 6C-2 B, D, and E Factors

B. D. & E factors for determining rainfall intensity in the Rational and Modified Rational Methods (based on NOAA NW-14 Atlas data)

STATION	ID	1-YR			2-YR			5-YR			10-YR			25-YR			50-YR			100-YR		
		B	*D*	*E*	*B*	*D*	*E*	*B*	*D*	*E*	*B*	*D*	*E*	*B*	*D*	*E*	*B*	*D*	*E*	*B*	*D*	*E*
Abingdon 3 S	44-0021	40.08	11.76	0.86	49.15	12.35	0.87	56.06	12.86	0.85	59.35	12.81	0.84	61.55	12.56	0.81	62.63	12.31	0.78	63.71	12.10	0.76
Abingdon 7 WSW	44-0013	43.34	12.21	0.88	49.59	12.21	0.87	56.26	12.74	0.85	59.38	12.75	0.84	62.41	12.70	0.81	63.39	12.46	0.79	65.54	12.44	0.77
Allisonia 2 SSE	44-0135	41.50	11.96	0.87	49.41	12.26	0.87	55.38	12.54	0.84	58.04	12.32	0.82	58.43	11.84	0.78	58.83	11.53	0.76	57.64	10.99	0.74
Altavista	44-0166	43.03	10.97	0.84	51.76	11.34	0.84	56.95	11.46	0.81	57.34	11.01	0.78	55.75	10.31	0.74	53.93	9.81	0.71	52.14	9.25	0.69
Amelia 4 SW	44-0187	45.68	11.12	0.84	54.40	11.55	0.84	58.09	11.54	0.81	58.26	10.98	0.78	56.78	10.37	0.74	55.81	9.91	0.71	53.17	9.21	0.68
Amissville	44-0193	41.15	10.45	0.83	50.12	10.79	0.83	55.24	10.80	0.81	56.53	10.51	0.78	55.74	9.75	0.74	54.80	9.11	0.71	53.96	8.54	0.68
Appomattox	44-0243	41.20	10.74	0.83	49.67	11.16	0.83	52.95	11.08	0.80	54.17	10.74	0.77	52.97	10.06	0.73	51.54	9.50	0.70	49.67	8.92	0.67
Ashland 1 SW	44-0327	44.60	10.94	0.84	52.62	11.11	0.83	56.33	11.15	0.80	57.21	10.73	0.77	56.07	10.14	0.73	55.04	9.68	0.71	52.71	9.01	0.68
Back Bay Wildlife Refu	44-0385	53.68	11.15	0.85	59.68	11.38	0.84	54.44	11.20	0.80	61.10	10.84	0.77	58.58	10.07	0.73	59.22	9.46	0.70	57.04	8.76	0.67
Bedford	44-0551	37.67	10.46	0.82	45.85	10.94	0.82	50.40	10.91	0.79	51.89	10.64	0.77	51.58	10.03	0.73	50.54	9.57	0.70	48.91	8.99	0.68
Berryville	44-0670	36.70	9.20	0.84	44.43	9.46	0.84	47.82	9.02	0.80	49.21	8.53	0.78	49.83	7.75	0.74	49.99	7.14	0.71	49.54	6.50	0.69
Big Meadows 2	44-0720	38.14	9.24	0.82	46.37	9.48	0.82	50.45	9.27	0.79	50.00	8.57	0.75	49.65	7.78	0.72	48.46	7.02	0.69	47.82	6.48	0.66
Big Stone Gap	44-0733	41.03	11.46	0.86	49.16	11.90	0.86	55.20	12.20	0.83	59.15	12.23	0.82	61.10	11.80	0.78	62.05	11.42	0.76	63.89	11.25	0.74
Blacksburg	44-0765	39.42	11.59	0.86	46.75	11.84	0.85	52.88	12.09	0.83	54.67	11.84	0.80	55.05	11.37	0.77	53.76	10.89	0.75	52.89	10.48	0.72
Blacksburg 3 SE	44-0766	39.87	11.64	0.86	46.90	11.80	0.85	52.71	11.95	0.83	54.71	11.78	0.80	54.72	11.21	0.77	53.42	10.73	0.74	52.88	10.38	0.72
Blackstone Water Works	44-0778	47.58	11.18	0.85	57.66	11.62	0.85	61.32	11.66	0.81	61.53	11.16	0.78	58.93	10.40	0.74	57.64	9.90	0.71	55.04	9.25	0.68
Bland	44-0792	39.57	11.80	0.87	45.97	11.89	0.86	52.78	12.34	0.84	55.93	12.26	0.82	58.71	12.11	0.79	59.90	11.88	0.77	60.64	11.55	0.75
Bremo Bluff Pwr	44-0993	41.66	11.01	0.84	45.44	10.84	0.82	47.37	10.99	0.80	51.96	10.83	0.77	50.16	10.06	0.73	50.71	9.67	0.71	49.32	9.16	0.68
Brookneal	44-1082	45.17	11.34	0.85	52.53	11.38	0.84	57.02	11.41	0.81	57.43	11.00	0.78	56.50	10.42	0.74	54.70	9.87	0.71	52.27	9.16	0.68
Buchanan	44-1121	37.03	10.79	0.83	44.69	11.09	0.83	50.96	11.38	0.81	51.57	10.98	0.78	51.57	10.37	0.74	51.00	9.95	0.72	50.15	9.54	0.69
Buckingham	44-1136	40.38	10.66	0.83	46.68	10.80	0.82	51.25	11.10	0.80	53.35	10.75	0.77	51.81	10.03	0.73	50.52	9.40	0.70	49.20	8.90	0.67
Buena Vista	44-1159	37.49	11.19	0.85	45.39	11.59	0.85	51.46	11.77	0.82	54.43	11.67	0.80	53.71	10.98	0.76	54.10	10.73	0.74	53.58	10.35	0.72
Burkes Garden	44-1209	39.00	11.45	0.86	46.42	11.74	0.85	51.87	12.04	0.83	55.10	11.97	0.81	56.42	11.47	0.78	59.60	11.52	0.76	60.81	11.26	0.72
Byllesby 3 W	44-1259	40.08	11.76	0.86	46.38	11.80	0.86	53.19	12.20	0.83	55.73	12.01	0.81	58.62	11.79	0.78	59.61	11.50	0.76	61.47	11.34	0.74
Camp Pickett	44-1322	48.57	11.29	0.85	57.25	11.47	0.84	61.39	11.58	0.81	61.86	11.13	0.78	58.96	10.27	0.74	58.70	10.01	0.71	56.01	9.35	0.68
Catawba Sanatorium	44-1471	37.71	10.91	0.84	45.37	11.19	0.83	52.26	11.55	0.81	53.70	11.24	0.79	53.21	10.59	0.75	52.90	10.26	0.73	52.21	9.85	0.70
Charlotte Court Hse 3W	44-1585	47.18	11.32	0.85	54.86	11.54	0.84	58.40	11.47	0.81	60.03	11.22	0.78	57.46	10.35	0.74	56.45	9.93	0.71	53.54	9.19	0.68
Charlottesville	44-1598/1593	38.82	10.26	0.81	46.43	10.50	0.81	49.30	10.45	0.78	49.49	9.85	0.75	49.52	9.34	0.71	49.10	8.93	0.69	47.51	8.34	0.66
Chase City	44-1606	48.14	11.33	0.85	55.69	11.52	0.84	59.04	11.49	0.81	60.40	11.15	0.78	58.34	10.42	0.74	56.82	9.93	0.71	54.19	9.24	0.68

Chatham	44-1614	43.32	10.85	0.84	53.11	11.37	0.84	57.19	11.32	0.80	57.03	10.78	0.77	55.10	10.09	0.73	53.77	9.68	0.71	51.15	9.01	0.68
Churchville	44-1708	33.15	10.58	0.83	40.88	11.05	0.83	48.66	11.55	0.81	50.52	11.26	0.79	51.69	10.80	0.76	52.44	10.57	0.74	51.47	10.03	0.71
Clarendon Lyon Park	44-1729	46.45	11.40	0.85	57.70	11.93	0.86	62.74	11.96	0.82	63.92	11.66	0.80	61.35	10.75	0.75	59.64	10.21	0.72	57.16	9.49	0.69
Clarksville	44-1746	49.25	11.51	0.86	57.91	11.84	0.85	61.56	11.85	0.82	62.15	11.40	0.79	60.87	10.82	0.76	58.72	10.26	0.72	56.40	9.72	0.70
Clifton Forge 2 NW	44-1801	36.72	11.27	0.85	42.99	11.46	0.84	50.17	11.80	0.82	52.43	11.59	0.80	53.47	11.16	0.77	53.34	10.81	0.74	53.08	10.47	0.72
Colonial Beach	44-1913	46.14	11.30	0.85	55.01	11.45	0.84	59.36	11.49	0.81	60.10	11.05	0.78	58.89	10.39	0.74	57.06	9.79	0.71	55.34	9.22	0.68
Columbia 2SSE	44-1929	42.13	10.78	0.84	49.95	11.21	0.83	52.98	11.29	0.80	55.50	11.01	0.78	53.83	10.29	0.74	53.13	9.81	0.71	50.89	9.15	0.68
Concord 4 SSW	44-1955	41.64	10.99	0.84	49.11	11.18	0.83	52.16	10.99	0.80	53.67	10.74	0.77	52.42	10.02	0.73	51.28	9.62	0.70	48.81	8.85	0.67
Copper Hill	44-1999	39.18	10.57	0.82	46.69	10.76	0.82	51.90	10.91	0.79	53.32	10.63	0.77	52.05	9.93	0.73	51.95	9.61	0.71	49.94	8.98	0.68
Corbin	44-2009	46.26	11.17	0.84	56.32	11.59	0.85	61.07	11.74	0.82	60.47	11.11	0.78	59.27	10.42	0.74	57.94	9.90	0.71	56.31	9.34	0.69
Covington	44-2041	39.87	11.97	0.87	48.06	12.35	0.87	53.81	12.46	0.84	56.58	12.31	0.82	56.81	11.75	0.79	57.36	11.54	0.76	56.09	11.03	0.74
Covington Filter Plant	44-2044	37.45	11.51	0.86	46.17	12.02	0.86	54.30	12.47	0.84	54.61	11.93	0.81	56.33	11.65	0.78	56.75	11.40	0.76	55.38	10.88	0.73
Craigsville 2 S	44-2064	34.19	10.69	0.83	41.07	11.00	0.83	47.80	11.29	0.81	50.46	11.16	0.79	51.45	10.73	0.75	51.35	10.41	0.73	49.47	9.69	0.70
Crozier	44-2142	45.61	11.13	0.85	53.08	11.36	0.84	56.88	11.46	0.81	58.61	11.09	0.78	56.55	10.31	0.74	56.04	9.93	0.71	54.05	9.33	0.69
Culpeper	44-2155	42.76	10.88	0.84	51.02	11.09	0.83	55.05	11.00	0.80	56.19	10.60	0.77	55.55	9.96	0.74	54.58	9.42	0.71	52.82	8.74	0.68
Dahlgren Proving Groun	44-2195	45.40	11.15	0.84	55.01	11.45	0.84	59.96	11.56	0.81	60.24	11.10	0.78	60.24	11.10	0.78	57.84	9.92	0.71	55.40	9.18	0.68
Dale Enterprise	44-2208	33.63	10.08	0.84	40.13	10.30	0.83	45.74	10.33	0.81	46.93	9.83	0.78	48.07	9.37	0.75	47.26	8.70	0.72	46.43	8.11	0.69
Damascus	44-2216	41.72	11.83	0.87	48.39	12.03	0.86	55.16	12.60	0.84	58.03	12.43	0.82	60.19	12.17	0.79	61.09	11.84	0.77	62.12	11.62	0.75
Dante	44-2237	40.59	11.57	0.86	49.26	12.11	0.86	56.46	12.59	0.84	58.03	12.20	0.82	61.11	12.03	0.79	61.32	11.61	0.76	62.21	11.30	0.74
Danville	44-2245	46.61	11.13	0.85	55.93	11.49	0.84	60.27	11.49	0.81	60.15	11.09	0.78	58.22	10.43	0.74	55.97	9.94	0.71	52.67	9.21	0.68
Davenport 2 NE	44-2269	43.80	12.00	0.88	53.34	12.45	0.88	60.09	12.84	0.85	63.43	12.76	0.83	64.88	12.27	0.80	65.09	11.83	0.77	64.73	11.33	0.74
Deerfield 1 S	44-2315	32.81	10.33	0.82	41.33	11.03	0.83	47.83	11.32	0.81	50.26	11.11	0.79	50.79	10.62	0.75	50.75	10.30	0.73	49.48	9.67	0.70
Delaplane 1 N	44-2326	40.36	9.90	0.83	48.95	10.19	0.83	53.44	10.02	0.80	56.37	9.92	0.78	55.41	9.01	0.74	55.43	8.43	0.71	55.17	7.91	0.69
Driver 4 NE	44-2504	57.41	11.64	0.86	67.21	11.85	0.85	68.11	11.67	0.81	67.05	10.92	0.78	64.86	10.22	0.73	63.82	9.65	0.71	61.07	8.93	0.67
Elkwood 7 SE	44-2729	44.31	11.16	0.85	52.76	11.42	0.84	56.83	11.40	0.81	59.21	11.23	0.78	56.99	10.36	0.74	56.31	9.91	0.71	54.92	9.37	0.69
Emporia 1 WNW	44-2790	50.22	11.38	0.85	59.16	11.75	0.85	61.55	11.65	0.81	61.73	11.08	0.78	59.75	10.32	0.74	58.97	9.96	0.71	55.82	9.14	0.68
Farmville 2 N	44-2941	44.33	11.00	0.84	52.65	11.41	0.84	56.27	11.43	0.81	57.34	10.96	0.78	54.61	10.11	0.73	54.52	9.81	0.71	52.21	9.14	0.68
Floyd	44-3071	40.81	11.12	0.84	47.69	11.21	0.83	53.11	11.37	0.81	54.72	11.10	0.78	54.31	10.51	0.75	52.84	9.95	0.72	51.49	9.46	0.69
Fredericksburg Sewage	44-3204	46.35	11.29	0.85	56.02	11.64	0.85	60.87	11.75	0.82	60.89	11.22	0.79	59.48	10.49	0.75	57.99	9.96	0.72	55.77	9.30	0.69
Free Union	44-3213	37.59	10.17	0.81	45.08	10.37	0.81	50.36	10.68	0.78	50.36	10.68	0.78	49.90	9.49	0.72	49.24	9.00	0.69	48.09	8.48	0.66
Galax Radio WBOB	44-3267	38.43	11.06	0.84	45.97	11.47	0.84	50.86	11.43	0.81	54.24	11.34	0.79	57.06	11.02	0.76	60.03	10.97	0.74	62.25	10.80	0.73
Galax Water Plant	44-3272	38.93	11.16	0.84	45.33	11.30	0.84	51.95	11.61	0.81	54.28	11.32	0.79	57.77	11.14	0.76	60.03	10.97	0.74	62.07	10.71	0.73

Gathright Dam	44-3310	35.05	10.95	0.84	42.43	11.31	0.84	50.81	11.86	0.82	52.65	11.64	0.80	53.84	11.20	0.77	54.38	10.96	0.75	53.98	10.57	0.73
Glasgow	44-3375	36.76	10.76	0.84	44.97	11.31	0.83	50.03	11.24	0.80	52.53	11.16	0.78	52.33	10.50	0.75	51.95	10.08	0.72	51.03	9.62	0.70
Glen Lyn	44-3397	39.95	11.93	0.87	51.38	12.80	0.89	57.35	13.04	0.86	59.20	12.85	0.84	60.52	12.51	0.81	60.55	12.33	0.79	60.06	12.01	0.76
Gordonsville 3 S	44-3466	41.14	10.65	0.83	49.89	11.11	0.83	52.73	10.92	0.79	53.86	10.53	0.77	53.33	10.00	0.73	52.59	9.48	0.70	51.03	8.88	0.67
Goshen	44-3470	34.19	10.69	0.83	41.07	11.00	0.83	49.30	11.59	0.81	49.99	11.09	0.78	51.35	10.69	0.75	50.53	10.20	0.73	49.78	9.76	0.70
Groseclose	44-3623	40.12	11.55	0.86	47.25	11.84	0.86	53.15	12.26	0.84	55.61	12.02	0.81	57.56	11.77	0.79	60.23	11.77	0.77	62.30	11.65	0.75
Grundy	44-3640	41.36	11.37	0.87	50.25	11.88	0.87	58.56	12.46	0.85	59.45	11.92	0.82	60.68	11.34	0.79	63.55	11.28	0.77	64.08	10.88	0.75
Halifax 1 N	44-3690	46.57	11.26	0.85	54.66	11.39	0.84	60.00	11.54	0.81	60.18	11.16	0.78	58.41	10.51	0.74	55.87	9.89	0.71	53.46	9.27	0.69
Hillsville	44-3991	40.27	11.33	0.85	48.37	11.80	0.85	52.76	11.73	0.82	55.22	11.55	0.80	55.40	10.93	0.76	57.46	10.89	0.74	57.04	10.45	0.72
Holland 1 E	44-4044	59.92	11.72	0.86	72.03	12.17	0.86	71.64	11.88	0.82	71.84	11.29	0.79	68.93	10.52	0.74	67.04	9.86	0.71	64.87	9.26	0.68
Honaker	44-4078	44.62	12.33	0.88	51.96	12.41	0.87	58.38	12.81	0.85	61.46	12.66	0.83	62.01	12.07	0.79	62.13	11.58	0.76	63.92	11.39	0.74
Hopewell	44-4101	47.07	10.99	0.84	56.89	11.48	0.84	59.47	11.34	0.80	60.39	10.88	0.77	58.31	10.16	0.73	56.79	9.60	0.70	54.74	8.98	0.67
Hot Springs	44-4128	33.65	10.58	0.83	41.22	10.99	0.83	47.03	11.19	0.80	48.50	10.86	0.78	49.52	10.44	0.75	49.68	10.14	0.72	49.32	9.73	0.70
Huddleston 4 SW	44-4148	41.10	11.05	0.84	48.72	11.24	0.83	53.17	11.23	0.80	53.46	10.73	0.77	53.74	10.32	0.74	52.80	9.89	0.71	50.14	9.17	0.68
Hurley	44-4180	40.20	10.74	0.86	48.93	11.12	0.86	56.96	11.56	0.85	57.42	10.91	0.82	58.82	10.41	0.79	60.06	10.11	0.76	59.86	9.54	0.74
Hurley 1 SE	44-4185	39.92	10.83	0.86	49.20	11.37	0.86	56.70	11.79	0.85	58.86	11.42	0.82	60.45	10.93	0.79	61.38	10.45	0.77	64.39	10.37	0.75
Independence 2	44-4234	40.54	11.85	0.87	48.90	12.36	0.87	55.25	12.65	0.85	58.03	12.43	0.82	59.65	12.07	0.79	61.51	11.87	0.77	62.27	11.57	0.75
Indian Valley	44-4246	38.75	10.95	0.84	46.81	11.36	0.84	52.40	11.47	0.81	54.68	11.27	0.79	54.60	10.62	0.75	54.45	10.31	0.73	53.29	9.74	0.70
John Flannagan Reservo	44-4410	41.92	11.52	0.87	50.88	11.94	0.87	59.33	12.60	0.85	60.39	12.09	0.82	61.78	11.54	0.79	62.87	11.26	0.76	62.10	10.66	0.73
John H Kerr Dam	44-4414	48.04	11.36	0.85	56.91	11.71	0.85	60.35	11.69	0.82	60.88	11.24	0.79	59.21	10.55	0.75	57.68	10.10	0.72	55.00	9.42	0.69
Jordon Mines	44-4452	37.79	11.37	0.85	47.23	12.00	0.86	54.31	12.33	0.84	56.36	12.01	0.81	57.24	11.59	0.78	57.60	11.34	0.76	57.67	11.04	0.74
Kerrs Creek 1 WSW	44-4565	34.55	10.54	0.82	42.09	10.90	0.82	48.82	11.24	0.80	50.16	10.87	0.78	51.08	10.46	0.75	50.66	10.01	0.72	50.05	9.62	0.70
Lafayette 1 NE	44-4676	37.46	11.18	0.85	46.18	11.73	0.85	52.03	11.96	0.83	53.76	11.71	0.80	53.84	11.17	0.77	53.44	10.87	0.74	51.37	10.20	0.72
Langley Air Force Base	44-4720	53.52	11.35	0.85	60.58	11.30	0.84	61.40	11.17	0.80	62.92	10.71	0.77	60.11	9.85	0.72	59.43	9.35	0.69	56.83	8.56	0.66
Lawrenceville 3 E	44-4768	50.43	11.31	0.85	59.85	11.66	0.85	62.69	11.63	0.82	62.92	11.12	0.78	61.03	10.44	0.74	59.88	9.96	0.71	57.07	9.26	0.68
Lexington	44-4876	36.49	11.12	0.84	44.47	11.47	0.84	51.48	11.82	0.82	53.93	11.65	0.80	54.00	11.09	0.76	53.54	10.67	0.74	53.09	10.30	0.72
Lincoln	44-4909	40.83	10.15	0.84	48.62	10.34	0.83	53.84	10.36	0.80	53.98	9.72	0.77	53.48	9.01	0.73	52.67	8.37	0.70	51.79	7.77	0.68
Louisa	44-5050	42.48	10.88	0.83	49.64	10.98	0.83	54.09	11.15	0.80	56.45	10.94	0.78	54.69	10.19	0.74	53.96	9.70	0.71	52.46	9.15	0.68
Luray 5 E	44-5096	35.98	9.21	0.83	43.12	9.39	0.83	46.81	9.05	0.79	47.73	8.51	0.77	47.06	7.61	0.73	47.08	7.07	0.70	46.06	6.37	0.67
Lynchburg WSO Airport	44-5120	39.65	10.84	0.84	47.72	11.23	0.83	51.39	11.13	0.80	51.92	10.70	0.77	51.68	10.17	0.74	50.64	9.73	0.71	48.39	9.00	0.68
Manassas	44-5213	43.15	11.20	0.85	52.92	11.71	0.85	56.91	11.62	0.81	58.32	11.31	0.79	57.48	10.68	0.75	54.82	9.81	0.72	52.93	9.15	0.69
Marion	44-5271	41.12	11.78	0.87	49.24	12.15	0.87	55.97	12.64	0.85	58.39	12.42	0.82	61.68	12.22	0.80	62.62	11.94	0.77	65.03	11.81	0.76

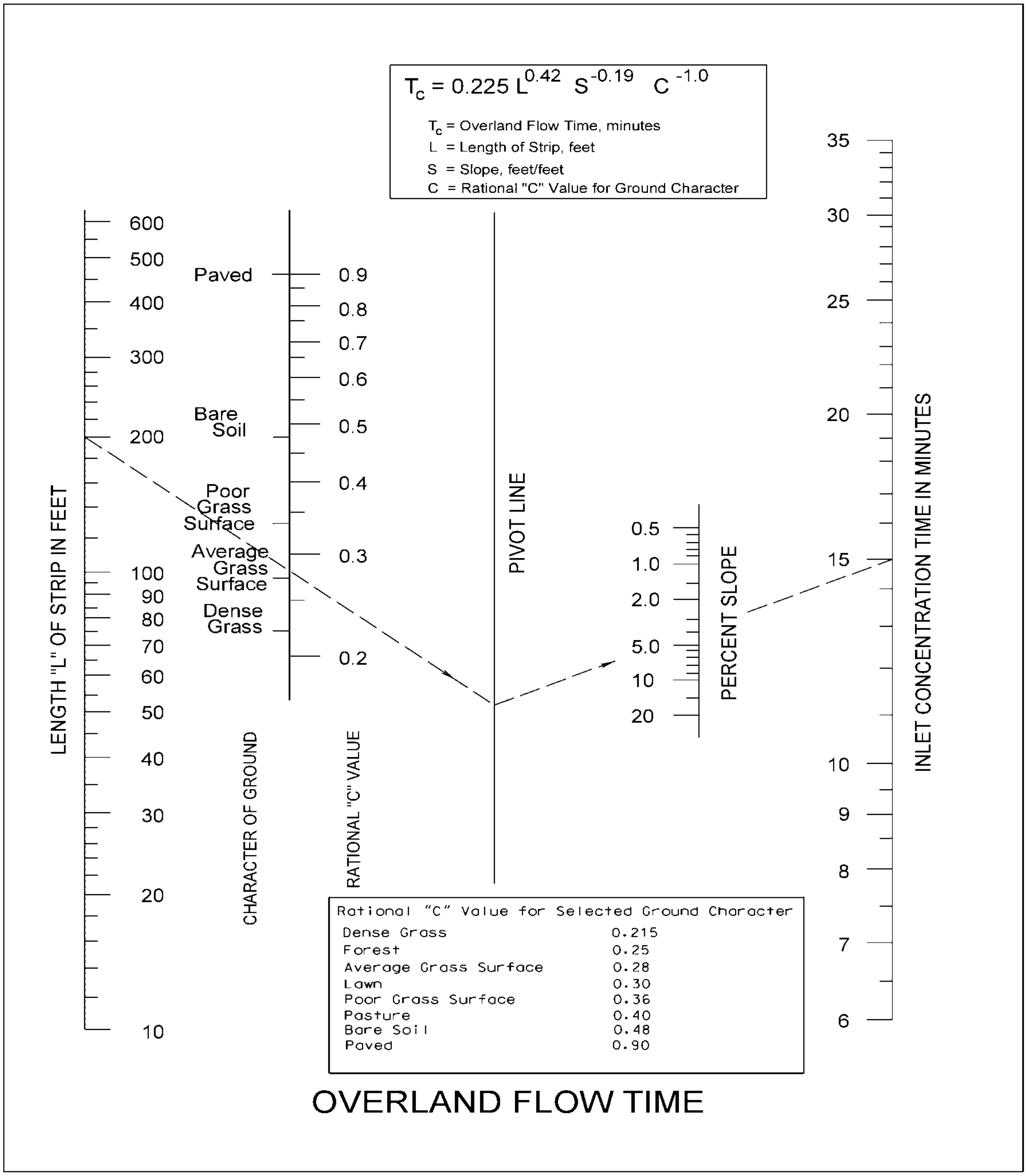
Martinsville Filter Pl	44-5300	42.43	10.66	0.83	49.13	10.74	0.82	54.66	10.94	0.79	53.78	10.32	0.76	53.56	9.85	0.72	51.28	9.28	0.69	49.08	8.65	0.67
Mathews 2 ENE	44-5338	48.85	10.96	0.84	57.22	11.17	0.83	58.37	11.08	0.80	59.17	10.48	0.76	57.56	9.79	0.72	56.81	9.21	0.69	54.84	8.52	0.66
MC Gaheysville 3 S	44-5423	34.09	10.19	0.83	39.73	10.16	0.82	46.51	10.47	0.80	48.98	10.17	0.78	50.91	9.89	0.75	49.93	9.20	0.72	49.93	8.83	0.70
Meadows of Dan 5 SW	44-5453	37.31	9.46	0.78	44.19	9.54	0.78	47.59	9.43	0.74	48.75	9.11	0.72	48.87	8.54	0.68	48.85	8.19	0.66	47.80	7.63	0.63
Mendota	44-5501	41.72	11.83	0.87	52.64	12.66	0.88	57.00	12.76	0.85	61.09	12.83	0.84	63.50	12.58	0.81	64.62	12.35	0.79	65.99	12.12	0.77
Millgap 1 NNE	44-5595	32.97	10.62	0.83	39.61	10.90	0.82	48.46	11.59	0.82	50.76	11.34	0.79	52.44	11.00	0.76	53.37	10.81	0.74	53.65	10.52	0.72
Montebello	44-5685	33.38	9.57	0.79	41.05	9.99	0.79	45.94	10.01	0.76	47.53	9.66	0.74	48.04	9.13	0.70	47.96	8.69	0.68	46.96	8.17	0.65
Montebello Fish Nurser	44-5690	33.46	9.35	0.78	40.63	9.64	0.78	46.12	9.83	0.76	46.69	9.26	0.73	47.78	8.82	0.70	47.54	8.38	0.67	46.95	7.93	0.65
Monterey	44-5698	32.89	10.62	0.83	40.89	11.17	0.83	46.60	11.41	0.81	50.16	11.45	0.79	51.01	10.95	0.76	50.84	10.62	0.74	50.08	10.13	0.71
Mount Weather	44-5851	40.22	9.77	0.84	48.94	10.07	0.84	51.83	9.66	0.80	53.68	9.24	0.78	53.58	8.47	0.74	53.77	7.90	0.71	52.68	7.12	0.68
New Castle	44-6012	39.37	11.41	0.86	48.42	12.08	0.86	53.04	12.04	0.83	56.22	12.03	0.81	57.07	11.60	0.78	56.33	11.20	0.76	55.40	10.78	0.73
Newport 2 NNW	44-6046	38.91	11.68	0.86	47.47	12.10	0.86	53.31	12.37	0.84	56.32	12.31	0.82	57.56	12.00	0.79	55.81	11.49	0.76	53.88	10.91	0.74
Newport News Press Bld	44-6054	54.86	11.46	0.85	63.46	11.62	0.85	63.77	11.35	0.81	64.67	10.83	0.77	61.73	9.97	0.73	60.65	9.43	0.70	58.86	8.84	0.67
Norfolk WSO Airport	44-6139	52.15	11.26	0.85	59.70	11.35	0.84	60.76	11.16	0.80	60.37	10.45	0.76	58.19	9.67	0.72	57.50	9.19	0.69	55.23	8.48	0.66
North Fork Reservoir	44-6173	41.85	11.73	0.86	50.98	12.26	0.87	57.93	12.64	0.85	58.91	12.15	0.82	61.65	11.90	0.79	64.23	11.81	0.77	64.08	11.29	0.74
North River Dam	44-6199	33.22	10.39	0.83	41.27	10.96	0.83	47.08	11.06	0.81	49.12	10.88	0.79	49.22	10.27	0.75	48.32	9.78	0.72	48.72	9.51	0.70
Onley 1 S	44-6362	36.59	9.73	0.79	42.32	9.78	0.78	45.90	9.93	0.76	46.78	9.40	0.73	45.86	8.69	0.69	45.74	8.21	0.66	44.68	7.67	0.63
Oyster 1 W	44-6456	38.18	10.02	0.80	46.91	10.69	0.81	43.63	10.41	0.78	49.35	10.17	0.75	48.24	9.50	0.71	49.31	8.95	0.68	48.12	8.32	0.65
Painter 2 W	44-6475	37.27	9.88	0.80	44.27	10.19	0.79	46.59	10.01	0.76	46.39	9.32	0.72	46.60	8.85	0.69	46.19	8.29	0.66	45.16	7.75	0.63
Palmyra 2	44-6491	40.67	10.77	0.83	47.31	11.02	0.83	49.87	11.07	0.80	51.96	10.65	0.77	51.58	10.10	0.73	51.09	9.66	0.71	49.88	9.15	0.68
Pedlar Dam	44-6593	37.16	10.88	0.84	44.92	11.27	0.83	50.23	11.38	0.81	53.17	11.23	0.79	52.13	10.50	0.75	52.32	10.15	0.72	51.07	9.67	0.70
Pennington Gap 1 W	44-6626	36.29	10.73	0.83	43.20	11.06	0.83	49.78	11.45	0.81	52.48	11.31	0.79	56.15	11.17	0.77	58.59	11.07	0.75	59.52	10.68	0.72
Philpott Dam 2	44-6692	40.89	10.29	0.81	47.86	10.33	0.80	52.10	10.38	0.77	52.58	9.94	0.75	51.65	9.32	0.71	50.18	8.82	0.68	48.25	8.21	0.65
Piedmont Research Stn	44-6712	41.58	10.63	0.83	50.68	11.07	0.83	54.16	11.00	0.80	54.81	10.46	0.77	53.97	9.88	0.73	53.25	9.42	0.70	51.65	8.83	0.68
Pilot 1 ENE	44-6723	39.70	11.25	0.85	47.55	11.60	0.84	53.04	11.73	0.82	54.24	11.34	0.79	55.31	10.96	0.76	54.34	10.49	0.73	53.40	10.03	0.71
Powhatan	44-6906	45.14	11.16	0.84	53.84	11.53	0.84	57.71	11.61	0.81	58.65	11.14	0.78	55.84	10.25	0.74	55.12	9.83	0.71	53.32	9.30	0.68
Pulaski	44-6955	42.02	12.35	0.89	50.66	12.82	0.88	56.31	12.92	0.85	57.82	12.61	0.83	58.75	12.18	0.79	58.88	11.84	0.77	58.12	11.38	0.75
Quantico 1 S	44-6979	45.69	11.29	0.85	56.10	11.72	0.85	59.84	11.54	0.81	61.60	11.34	0.79	60.40	10.68	0.75	58.12	9.99	0.72	55.45	9.25	0.69
Radford	44-6999	40.09	12.01	0.88	49.38	12.57	0.88	56.82	12.95	0.86	59.46	12.88	0.84	60.02	12.40	0.81	58.91	11.98	0.78	57.86	11.54	0.75
Randolph 5 NNE	44-7025	45.81	11.21	0.85	52.73	11.26	0.83	57.91	11.41	0.81	57.95	10.90	0.78	56.95	10.37	0.74	55.23	9.84	0.71	52.43	9.10	0.68
Rapidan	44-7033	41.20	10.59	0.83	49.81	10.95	0.83	53.97	10.96	0.80	55.05	10.58	0.77	54.18	9.97	0.73	53.51	9.48	0.70	52.10	8.94	0.68

Richardsville	44-7164	43.52	11.05	0.84	52.84	11.40	0.84	57.42	11.52	0.81	59.21	11.23	0.78	56.96	10.38	0.74	56.53	9.94	0.72	54.66	9.31	0.69
Richmond WB City	44-7206	46.49	11.09	0.84	54.21	11.19	0.83	58.15	11.23	0.80	59.29	10.90	0.77	57.44	10.18	0.73	55.73	9.63	0.70	53.10	8.97	0.67
Richmond WSO Airport	44-7201	46.27	11.01	0.84	54.61	11.24	0.83	59.16	11.40	0.80	59.77	10.92	0.78	57.15	10.11	0.73	55.88	9.61	0.70	53.69	9.01	0.67
Riverton	44-7254	37.30	9.34	0.84	44.53	9.59	0.83	48.63	9.34	0.80	49.29	8.68	0.77	49.64	7.90	0.74	49.36	7.29	0.71	48.90	6.60	0.68
Roanoke	44-7275	38.05	10.85	0.84	45.84	11.22	0.83	50.67	11.22	0.80	52.24	10.93	0.78	52.34	10.41	0.74	51.84	10.05	0.72	50.33	9.49	0.69
Roanoke WSO Airport	44-7285	37.89	10.88	0.84	45.50	11.25	0.83	51.74	11.44	0.81	52.42	10.99	0.78	53.22	10.56	0.75	53.01	10.16	0.72	50.98	9.46	0.69
Rockfish	44-7312	36.79	10.02	0.81	43.61	10.25	0.80	47.83	10.36	0.77	48.82	9.87	0.75	48.47	9.31	0.71	48.10	8.85	0.68	46.95	8.33	0.66
Rocky Knob	44-7330	37.63	9.55	0.79	44.65	9.71	0.78	49.54	9.75	0.75	49.92	9.35	0.72	49.70	8.73	0.69	79.83	8.43	0.67	48.73	7.90	0.64
Rocky Mount	44-7338	40.68	10.55	0.83	48.41	10.83	0.82	53.20	10.87	0.79	53.86	10.53	0.76	52.02	9.72	0.72	51.24	9.32	0.70	49.34	8.70	0.67
Saltville	44-7501	40.01	11.62	0.86	50.60	12.48	0.88	56.98	12.82	0.85	59.22	12.58	0.83	61.05	12.15	0.80	62.41	11.93	0.77	64.89	11.84	0.76
Somerset	44-7904	40.03	10.45	0.82	48.82	10.89	0.82	52.48	10.83	0.79	53.27	10.35	0.76	52.94	9.82	0.73	52.59	9.38	0.70	50.99	8.76	0.67
Speedwell	44-7971	40.35	11.82	0.87	47.18	12.03	0.86	51.69	12.17	0.83	55.93	12.22	0.82	58.45	11.94	0.79	59.51	11.64	0.76	61.33	11.49	0.75
Spring Creek 2	44-7997	39.85	11.90	0.87	47.79	12.20	0.87	54.04	12.49	0.84	58.31	12.46	0.83	61.20	12.16	0.80	63.47	12.01	0.78	64.73	11.66	0.75
Staffordsville 3 N	44-8022	40.44	11.95	0.87	47.98	12.18	0.87	56.06	12.75	0.85	57.29	12.43	0.82	58.31	12.08	0.79	58.14	11.77	0.77	56.66	11.28	0.74
Star Tannery	44-8046	33.90	8.64	0.83	40.98	8.84	0.83	44.88	8.49	0.80	45.80	7.87	0.77	47.14	7.19	0.74	47.46	6.55	0.72	47.13	5.85	0.69
Staunton Sewage Plant	44-8062	34.08	10.75	0.83	40.54	10.88	0.83	48.22	11.45	0.81	49.32	10.98	0.78	51.27	10.72	0.76	51.41	10.42	0.73	50.72	9.95	0.71
Stony Creek 3 ESE	44-8129	51.34	11.33	0.85	60.72	11.62	0.85	62.74	11.47	0.81	62.98	10.93	0.78	61.51	10.30	0.74	59.69	9.73	0.71	57.14	9.04	0.68
Stuart 1 SSE	44-8170	39.76	9.88	0.80	46.59	9.95	0.79	50.19	9.82	0.76	51.49	9.57	0.73	50.62	8.92	0.70	49.90	8.53	0.67	47.97	7.88	0.64
Stuart's Draft	44-8172	36.05	10.57	0.83	41.78	10.61	0.82	47.64	10.94	0.79	49.66	10.64	0.77	50.80	10.23	0.74	50.66	9.87	0.71	49.62	9.33	0.69
Suffolk Lake Kilby	44-8192	60.33	11.72	0.86	70.33	11.91	0.86	71.21	11.73	0.82	71.38	11.15	0.78	68.08	10.31	0.74	66.66	9.76	0.71	64.58	9.13	0.68
Tangier Island	44-8323	40.42	10.25	0.81	48.76	10.62	0.81	51.41	10.58	0.78	52.60	10.13	0.75	51.21	9.36	0.71	50.53	8.79	0.68	49.14	8.20	0.65
Tazewell	44-8354	39.70	11.89	0.87	47.17	12.31	0.87	53.55	12.64	0.85	56.80	12.47	0.83	58.38	12.03	0.79	60.70	11.94	0.77	62.25	11.70	0.75
The Plains 2 NNE	44-8396	43.37	10.59	0.84	52.84	10.98	0.84	56.98	10.93	0.81	57.60	10.48	0.78	54.98	9.46	0.73	53.73	8.90	0.70	52.20	8.26	0.67
Timberville 3 E	44-8448	33.39	9.05	0.83	40.49	9.37	0.83	43.99	8.92	0.80	44.78	8.30	0.77	45.74	7.61	0.74	44.81	6.90	0.71	44.05	6.22	0.68
Trout Dale 3 SSE	44-8547	38.67	10.88	0.84	47.29	11.45	0.84	52.47	11.82	0.82	55.31	11.70	0.80	57.92	11.52	0.78	58.92	11.24	0.76	60.25	11.09	0.74
Tye River 1 SE	44-8600	37.81	10.29	0.82	46.12	10.75	0.82	49.76	10.72	0.79	51.85	10.47	0.76	51.64	9.92	0.73	51.10	9.44	0.70	49.29	8.82	0.67
Vienna Tysons Corner	44-8737	44.85	11.13	0.85	54.58	11.50	0.84	60.33	11.70	0.82	61.48	11.37	0.79	60.17	10.68	0.75	58.24	10.02	0.72	55.82	9.34	0.69
Walkerton 2 NW	44-8829	45.88	10.95	0.84	55.33	11.35	0.84	58.04	11.31	0.80	59.43	10.90	0.77	57.42	10.14	0.73	55.91	9.56	0.70	54.01	8.95	0.67
Wallaceton LK Drumond	44-8837	63.89	11.84	0.87	73.86	12.00	0.86	72.94	11.68	0.82	73.73	11.16	0.78	70.31	10.31	0.74	68.75	9.70	0.71	65.94	8.99	0.67
Wallops Island WSSF	44-8849	44.13	11.06	0.84	52.61	11.34	0.84	56.92	11.37	0.80	55.92	10.66	0.77	54.88	10.02	0.73	54.25	9.55	0.70	52.51	8.89	0.67
Warrenton 3 SE	44-8888	44.95	11.37	0.85	53.02	11.52	0.84	58.30	11.53	0.81	59.00	11.10	0.79	57.33	10.27	0.74	56.88	9.86	0.72	55.65	9.27	0.69
Warsaw 2 N	44-8894	45.98	10.98	0.84	54.91	11.29	0.84	58.57	11.32	0.80	59.01	10.81	0.77	57.09	9.98	0.73	56.65	9.61	0.70	54.41	8.88	0.67

Washington Reagan AP	44-8906	48.41	11.64	0.86	59.17	12.12	0.86	64.60	12.16	0.83	65.80	11.88	0.80	62.99	10.95	0.76	60.95	10.35	0.73	58.74	9.73	0.70
Washington WB Chantill	44-8903	43.11	11.17	0.85	52.08	11.51	0.84	57.55	11.60	0.82	57.91	11.16	0.79	55.81	10.28	0.74	55.52	9.90	0.72	54.59	9.38	0.69
West Point 2 SW	44-9025	48.52	11.11	0.84	56.84	11.21	0.83	59.14	11.15	0.80	60.65	10.79	0.77	58.96	10.10	0.73	57.21	9.43	0.70	55.77	8.92	0.67
White Gate	44-9060	40.13	11.92	0.87	47.61	12.12	0.87	54.07	12.40	0.84	55.89	12.13	0.82	57.17	11.82	0.79	57.71	11.59	0.76	56.51	11.08	0.74
Williamsburg 2 N	44-9151	49.10	10.95	0.84	59.56	11.34	0.84	60.95	11.13	0.80	59.67	10.41	0.76	58.00	9.71	0.72	56.12	9.04	0.69	54.53	8.47	0.66
Winchester 3 ESE	44-9186	34.80	8.75	0.83	41.67	9.00	0.83	45.89	8.65	0.80	47.04	8.04	0.77	47.39	7.22	0.74	47.88	6.68	0.71	47.74	6.05	0.68
Wise 1 SE	44-9215	42.80	11.95	0.87	50.92	12.34	0.87	58.80	12.90	0.85	59.64	12.42	0.82	61.08	11.94	0.79	62.13	11.58	0.76	62.93	11.25	0.74
Woodstock 2 NE	44-9263	32.91	8.48	0.82	40.73	8.92	0.83	43.92	8.42	0.79	45.40	7.87	0.77	45.39	6.97	0.73	44.97	6.26	0.70	45.00	5.68	0.68
Woolwine 4 S	44-9272	38.07	9.51	0.79	44.50	9.54	0.78	48.24	9.47	0.74	49.68	9.22	0.72	48.64	8.55	0.68	48.00	8.12	0.66	46.17	7.45	0.63
Wytheville Post Office	44-9301	40.44	11.99	0.88	50.04	12.70	0.88	53.06	12.58	0.84	55.24	12.30	0.82	58.72	12.09	0.79	61.32	12.02	0.77	62.68	11.62	0.75

The B, D, and E factors for the state of Virginia are also available upon request in the form of a Microsoft EXCEL spreadsheet.

Appendix 6D-1 Overland Flow Time - Seelye



$$T_c = 0.225\, L^{0.42}\ S^{-0.19}\ C^{-1.0}$$

T_c = Overland Flow Time, minutes
L = Length of Strip, feet
S = Slope, feet/feet
C = Rational "C" Value for Ground Character

Rational "C" Value for Selected Ground Character	
Dense Grass	0.215
Forest	0.25
Average Grass Surface	0.28
Lawn	0.30
Poor Grass Surface	0.36
Pasture	0.40
Bare Soil	0.48
Paved	0.90

OVERLAND FLOW TIME

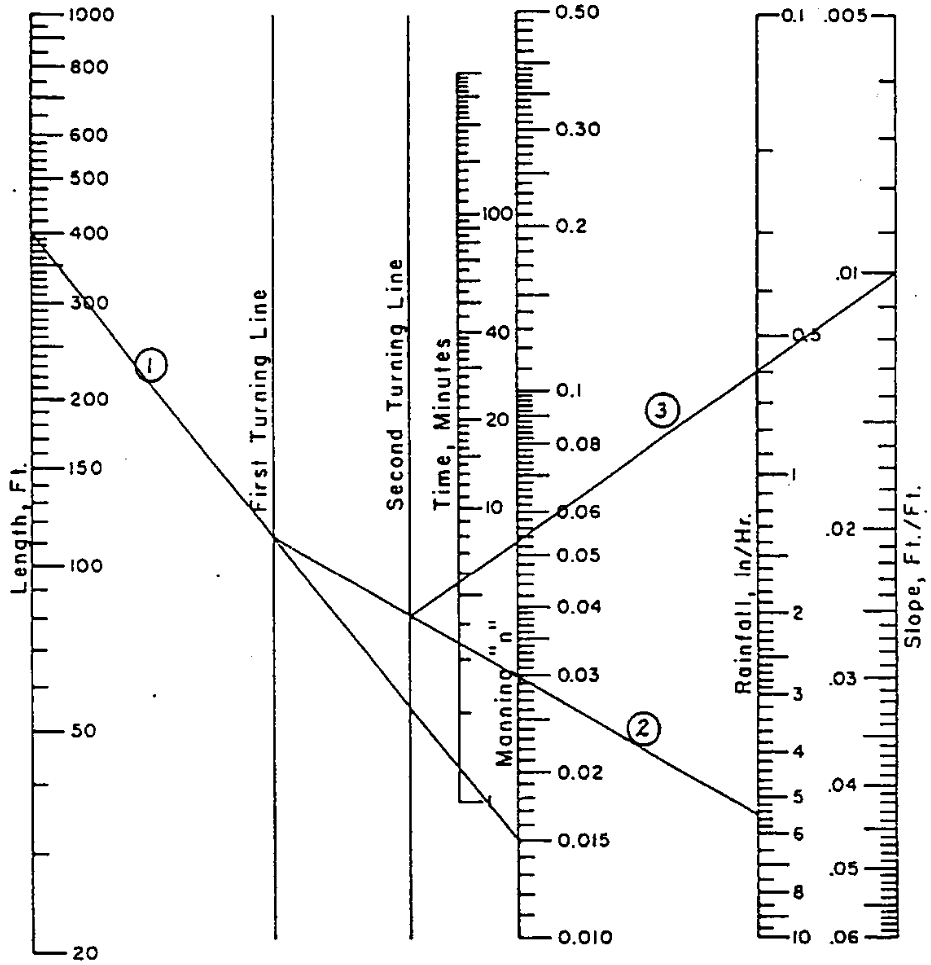
REPRINTED WITH PERMISSION FROM "DATA BOOK FOR CIVIL ENGINEERS" VOL. I - DESIGN
2nd EDITION (1951) BY E. E. SEELYE

Comments:
VDOT added a 'C-VALUE' scale and table* and a derived equation for Overland Flow Time to this nomograph. This was done without the permission of the author in the interest of providing the user with a quantitative comparison for the selection of 'CHARACTER OF GROUND' and an optional numerical solution to the nomograph. The Department warrants neither the accuracy nor the validity of either enhancement and cautions the user that it be used at their own risk.

* Rev 9/11

Appendix 6D-2 Kinematic Wave Formulation Overland Flow

Equation solved by nomograph: $t_c\,(min) = .93\dfrac{L^{0.6}n^{0.6}}{i^{0.4}S_0^{0.3}}$



The initially assumed value of i and the nomograph value of t must be checked against the applicable intensity-duration-frequency curve by trial and error.

Example:
L = 400 ft.
n = 0.015
i = 5.5 in./hr.
S_0 = 0.01
t = 5.5 min.

ONE INCH is 25.4mm
ONE FOOT is 0.3048m

Nomograph for determining time of concentration for overland flow, Kinematic Wave Formulation. *(After Ragan.)*

Comments:
VDOT has determined that the Kinematic Wave Method should only be used for:
a) Impervious Surfaces
b) n = 0.05 or less
c) Length = 300' Maximum
d) See page 2 of 2 for suggested Manning's roughness coefficients

Appendix 6D-2 Mannings Roughness Coefficient for Shallow Sheet Flow

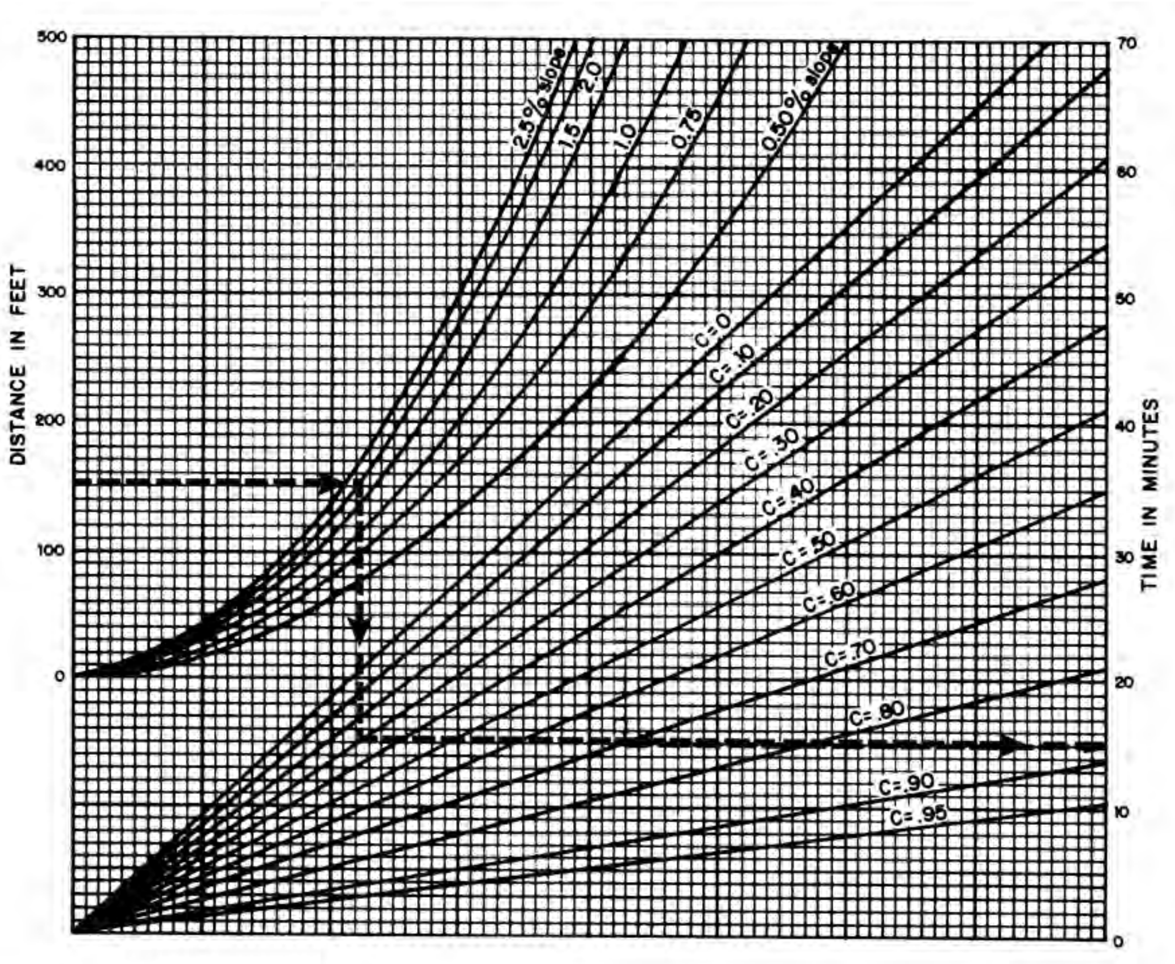
Surface Description	n[1]
Smooth surfaces - concrete, asphalt, gravel, or bare soil (compacted)	0.011
Fallow – no residue (non-compacted bare, plowed soil)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass prairie	0.15
Dense grasses[2]	0.24
Bermuda grass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

Soil Conservation Service Urban Hydrology for small water sheds Technical Release No. 55, Natural Resources Conservation Service, Washington, D.C. 1986

[1] The n values are a composite of information complied by Engman (1986).
[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass and native grass mixtures.
[3] When selecting n, consider cover to a height of about 1 inch. This is the only part of the plant cover that will obstruct sheet flow.

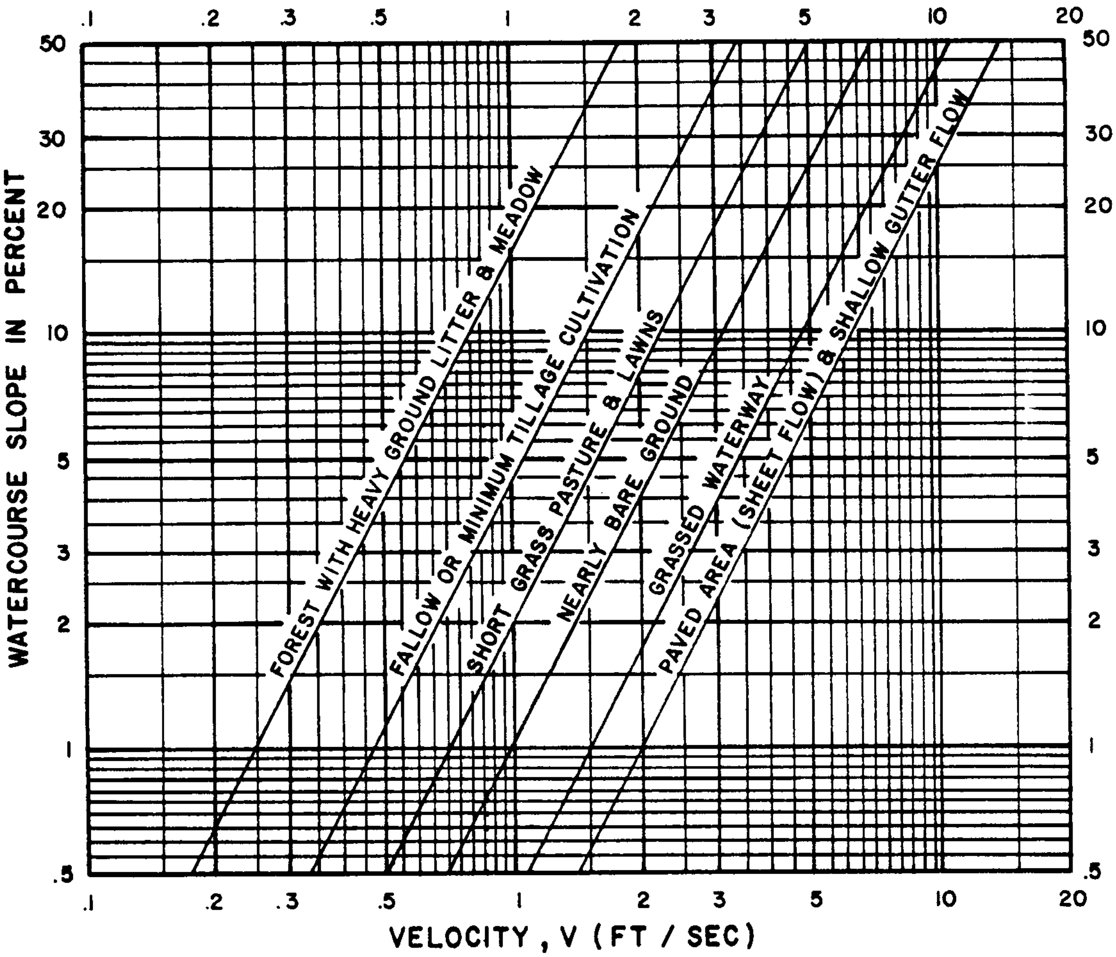
Source: AASHTO 2005 MODEL DRAINAGE MANUAL (text shown in parentheses are VDOT additions to the original chart which were included to simplify interpretation and application)

Appendix 6D-3 Overland Time of Flow



Source:
Airport Drainage, Federal Aviation Administration, 1965

Appendix 6D-4 Overland Flow Velocity

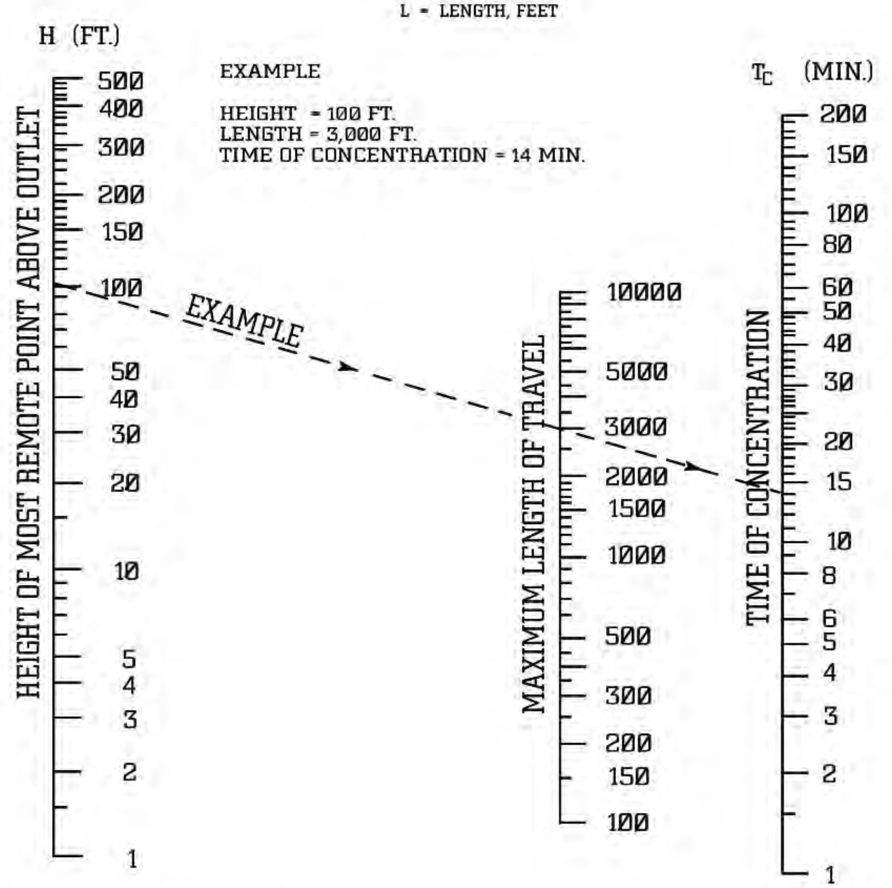


Source:
HEC No. 19, FHWA (archived)

Appendix 6D-5 Time of Concentration for Small Drainage Basins - Kirpich

$$T_C = 0.00948\ H^{-0.38}\ L^{1.13}$$

T_C = FLOW TIME, MINUTES
H = HEIGHT, FEET
L = LENGTH, FEET



Based on study by P.Z. Kirpich,
Civil Engineering, Vol. 10 No. 6, June 1940, p. 362

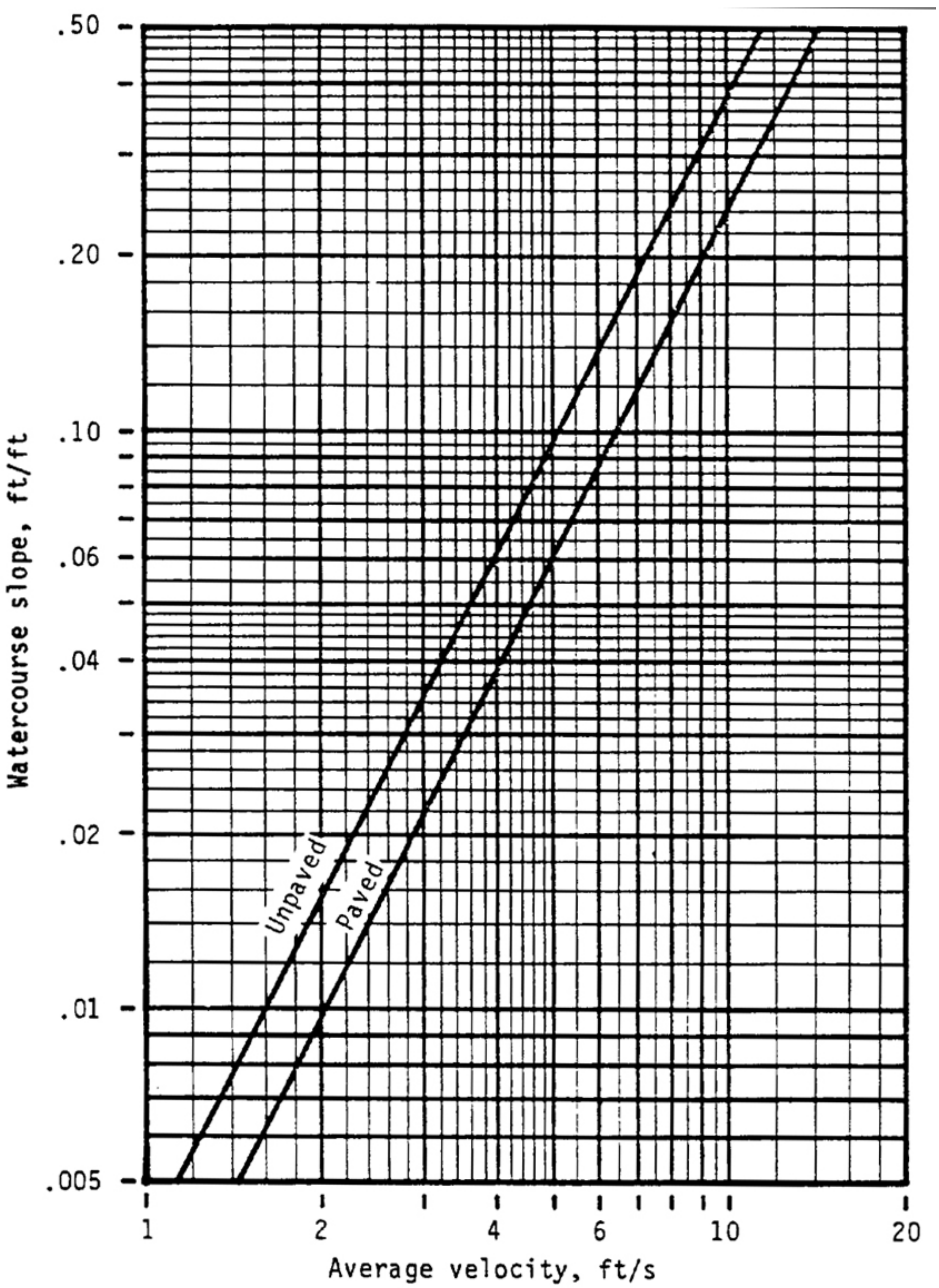
TIME OF CONCENTRATION OF SMALL DRAINAGE BASINS

* NOTE:
USE NOMOGRAPH FOR NATURAL BASINS WITH WELL-DEFINED CHANNELS AND FOR MOWED GRASS ROADSIDE CHANNELS.

Comments:
VDOT derived an equation from and added it to this nomograph. This was done without the author's permission in the interest of providing the user with an optional mathematical solution. The Department warrants neither the accuracy nor the validity of this equation and cautions the user that it be used at their own risk.

****The Kirpich Chart should only be used for channel time in Virginia.**

Appendix 6D-6 **Average Velocities for Estimating Travel Time for Shallow Concentrated Flow**



Source:
SCS, 210-VI-TR-55, Second Edition, June, 1986
VDOT has determined that this nomograph produces essentially the same flow time as the "Kirpich" Method.

Appendix 6E-1 Rational Method Runoff Coefficients

Recommended Coefficient of Runoff Values for Various Selected Land Uses

Description of Area		Runoff Coefficients
Business: Industrial and Commercial		0.80-0.90
Apartments and Townhomes		0.65-0.75
Schools		0.50-0.60
Residential	- lots 10,000 sq. ft.	0.40-0.50
	- lots 12,000 sq. ft.	0.40-0.45
	- lots 17,000 sq. ft.	0.35-0.45
	- lots ½ acre or more	0.30-0.40
Parks, Cemeteries and Unimproved Areas		0.20-0.35
Paved and Roof Areas		0.90
Cultivated Areas		0.50-0.70
Pasture		0.35-0.45
Lawns		0.25-0.35
Forest		0.20-0.30
Steep Grass (2:1)*		0.40-0.70
Shoulder and Ditch Areas *		0.35-0.50

Comments:

1. The lowest range of runoff coefficients may be used for flat areas (areas where the majority of the grades and slopes are 2% and less).

2. The average range of runoff coefficients should be used for intermediate areas (areas where the majority of the grades and slopes are from 2% to 6%).

3. The highest range of runoff coefficients shall be used for steep areas (areas where the majority of the grades are greater than 6%), for cluster areas, and for development in clay soil areas.

4. See Appendixes 6E-2, 6E-3, 6E-4 and 6E-5 for runoff coefficients with the C_f factor applied.

*Lower runoff coefficients should be used for permanent or established conditions (post-construction), i.e. sizing stormwater management basins.

*Higher runoff coefficients should be used to design roadside ditch linings (construction). The design considers the ditch lining as not yet established.

Comments: Runoff Coefficients compiled from various sources.

Appendix 6E-2 Rational Method Runoff Coefficients with 10 yr C_f factor Applied

C_{Cf} Values for 10 Year Storm Frequency (C_f=1.0)					
Land Use		**Average Watershed Slope**			**Average % Impervious**
		Flat <2%	**Rolling 2% - 6%**	**Steep >6%**	
Business, Commercial & Industrial		0.8	0.85	0.90	90%
Apartments and Townhomes		0.65	0.70	0.75	75%
Schools		0.50	0.55	0.60	50%
Residential	lots 10,000 sq. ft	0.40	0.45	0.50	35%
	lots 12,000 sq. ft.	0.40	0.43	0.45	30%
	lots 17,000 sq. ft.	0.35	0.40	0.45	25%
	lots ½ acre or more	0.30	0.35	0.40	20%
Parks, Cemeteries and Unimproved Areas		0.20	0.28	0.35	15%
Paved and Roof Areas		0.90			100%
Cultivated Areas		0.50	0.60	0.70	Varies
Pasture		0.35	0.40	0.45	Varies
Lawns		0.25	0.30	0.35	Varies
Forest		0.20	0.25	0.30	Varies
Railroad Yard Areas		0.20	0.30	0.40	
Roadway Slopes (2:1) w/ Little or No Vegetated Cover		0.70			
Roadway Shoulder & Ditch Areas w/ Little or No Vegetated Cover		0.50			
Roadway Slopes (2:1) w/ Established Vegetated Cover		0.40			
Roadway Shoulder & Ditch Areas w/ Established Vegetated Cover		0.35			

Rational Formula (Revised) – Q = $\mathbf{C_{Cf}}$ **A i**‡

‡ Source: VDOT

Appendix 6E-3 Rational Method Runoff Coefficients with 25 yr C_f factor Applied

C_{Cf} Values for 50 Year Storm Frequency (C_f=1.1)					
Land Use		**Average Watershed Slope**			**Average % Impervious**
		Flat <2%	**Rolling 2% - 6%**	**Steep >6%**	
Business, Commercial & Industrial		0.88	0.94	0.99	90%
Apartments and Townhomes		0.72	0.77	0.83	75%
Schools		0.55	0.61	0.66	50%
Residential	lots 10,000 sq. ft	0.44	0.50	0.55	35%
	lots 12,000 sq. ft.	0.44	0.47	0.50	30%
	lots 17,000 sq. ft.	0.39	0.44	0.50	25%
	lots ½ acre or more	0.33	0.39	0.44	20%
Parks, Cemeteries and Unimproved Areas		0.22	0.30	0.39	15%
Paved and Roof Areas		0.99			100%
Cultivated Areas		0.55	0.66	0.77	Varies
Pasture		0.39	0.44	0.50	Varies
Lawns		0.28	0.33	0.39	Varies
Forest		0.22	0.28	0.33	Varies
Railroad Yard Areas		0.22	0.33	0.44	
Roadway Slopes (2:1) w/ Little or No Vegetated Cover		0.77			
Roadway Shoulder & Ditch Areas w/ Little or No Vegetated Cover		0.55			
Roadway Slopes (2:1) w/ Established Vegetated Cover		0.44			
Roadway Shoulder & Ditch Areas w/ Established Vegetated Cover		0.39			

Rational Formula (Revised) – Q = $\mathbf{C_{Cf}\ A\ i}$[‡]

‡ Source: VDOT

Appendix 6E-4 Rational Method Runoff Coefficients with 50 yr C_f factor Applied

C_{Cf} Values for 50 Year Storm Frequency (C_f=1.2)					
Land Use		**Average Watershed Slope**			**Average % Impervious**
		Flat <2%	**Rolling 2% - 6%**	**Steep >6%**	
Business, Commercial & Industrial		0.96	1.00	1.00	90%
Apartments and Townhomes		0.78	0.84	0.90	75%
Schools		0.6	0.66	0.72	50%
Residential	lots 10,000 sq. ft	0.48	0.54	0.60	35%
	lots 12,000 sq. ft.	0.48	0.51	0.54	30%
	lots 17,000 sq. ft.	0.42	0.48	0.54	25%
	lots ½ acre or more	0.36	0.42	0.48	20%
Parks, Cemeteries and Unimproved Areas		0.24	0.33	0.42	15%
Paved and Roof Areas			1.00		100%
Cultivated Areas		0.6	0.72	0.84	Varies
Pasture		0.42	0.48	0.54	Varies
Lawns		0.3	0.36	0.42	Varies
Forest		0.24	0.30	0.36	Varies
Railroad Yard Areas		0.24	0.36	0.48	
Roadway Slopes (2:1) w/ Little or No Vegetated Cover			0.84		
Roadway Shoulder & Ditch Areas w/ Little or No Vegetated Cover			0.60		
Roadway Slopes (2:1) w/ Established Vegetated Cover			0.48		
Roadway Shoulder & Ditch Areas w/ Established Vegetated Cover			0.42		

Rational Formula (Revised) – Q = $\mathbf{C_{Cf}\ A\ i}$[‡]

[‡] Source: VDOT

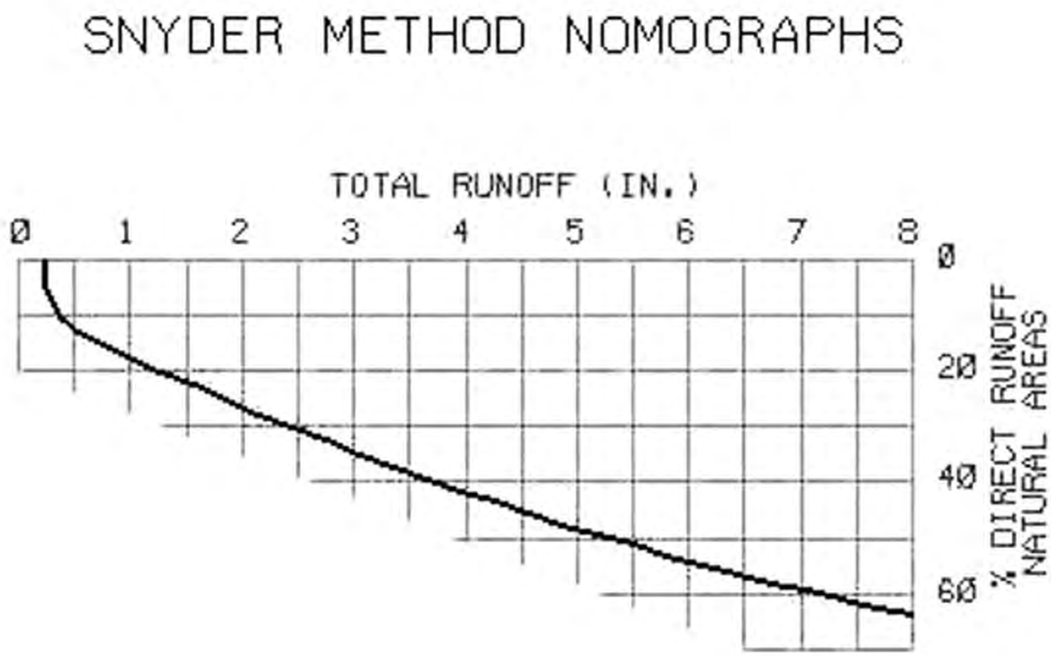
Appendix 6E-5 Rational Method Runoff Coefficients with 100 yr. C_f factor Applied

C_{Cf} Values for 100 Year Storm Frequency (C_f=1.25)					
Land Use		**Average Watershed Slope**			**Average % Impervious**
		Flat <2%	**Rolling 2% - 6%**	**Steep >6%**	
Business, Commercial & Industrial		1.00			90%
Apartments and Townhomes		0.81	0.88	0.94	75%
Schools		0.63	0.69	0.75	50%
Residential	lots 10,000 sq. ft	0.50	0.56	0.63	35%
	lots 12,000 sq. ft.	0.50	0.53	0.56	30%
	lots 17,000 sq. ft.	0.44	0.50	0.56	25%
	lots ½ acre or more	0.38	0.44	0.50	20%
Parks, Cemeteries and Unimproved Areas		0.25	0.34	0.44	15%
Paved and Roof Areas		1.00			100%
Cultivated Areas		0.63	0.75	0.88	Varies
Pasture		0.44	0.50	0.56	Varies
Lawns		0.31	0.38	0.44	Varies
Forest		0.25	0.31	0.38	Varies
Railroad Yard Areas		0.25	0.38	0.50	
Roadway Slopes (2:1) w/ Little or No Vegetated Cover		0.88			
Roadway Shoulder & Ditch Areas w/ Little or No Vegetated Cover		0.63			
Roadway Slopes (2:1) w/ Established Vegetated Cover		0.50			
Roadway Shoulder & Ditch Areas w/ Established Vegetated Cover		0.44			

Rational Formula (Revised) – Q = $\mathbf{C_{Cf}}$ **A i**‡

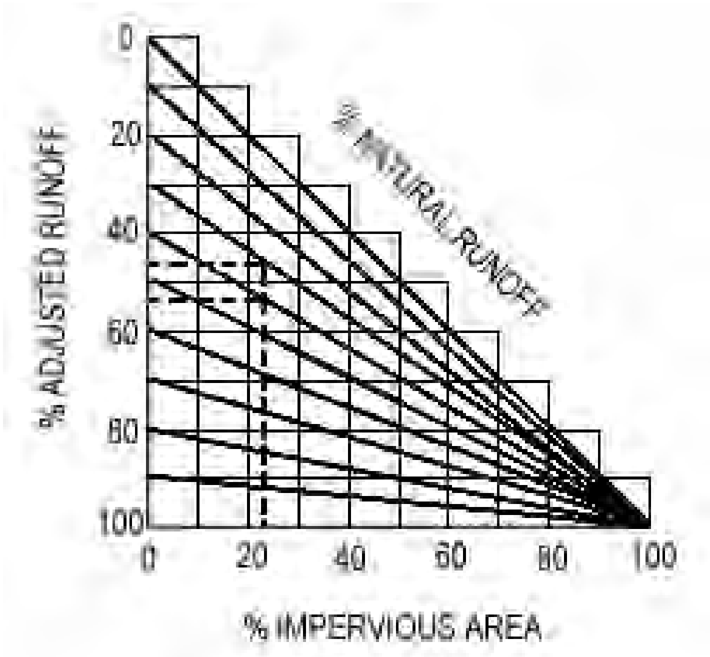
‡ Source: VDOT

Appendix 6G-1 Total Runoff vs. % Direct Runoff



Source:

Appendix 6G-2 % Impervious Area vs. % Adjusted Runoff



Source:

Appendix 6I-1 Joint Probability – Flood Frequency Analysis

a) Concept

One of the most frequently occurring hydrologic and hydraulic problems about which little literature is available is the problem of joint or coincidental occurrence of two or more events. If the events are caused by the same factors, then the events may be assumed to occur coincidentally. On the other hand, if the events are mutually independent and they leave probabilities of P_1 and P_2, then the probability of a coincidental occurrence is P_1 x P_2. In many cases, the events are somewhat related so that the probability of a joint occurrence is something other than P_1 x P_2.

The ideal solution to this problem would be a frequency analysis segregated with respect to the second variable. Unfortunately, unless long records are available, the number of primary events in any class interval of the secondary variables may be so limited that reliable analysis is not possible.

b) Antecedent and sequential conditions

- AMC II
- Type II Storm Distribution
- 1- to 10-day Storms

c) Outlet blockage

Tides –The Norfolk District has developed a coincidental frequency analysis of the tide and precipitation. The results of this study are shown in appendices 6I-2 and 6I-3. A stage-frequency analysis of all tides is shown in appendix 6I-4. Tide and precipitation are not totally independent and in fact, the severity of joint occurrences is less than if the variables were total independent.

For design purposes the following combinations of tide and precipitation may be used.

Frequencies for Coincidental Occurrences
(Norfolk Harbor)

10-Year Design

Tide	Ppt.
6.3	0.0
2.5	1-year
2.0	5-year
1.5	10-year

100-Year Design

Tide	Ppt.
8.4	0.00
5.4	1-year
4.2	5-year
1.5	100-year

The combinations of events at the extremities of the tables will usually be critical and often the intermediate combinations may be ignored.

d) Channel Flow

For the case of a tributary stream its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the mainstream. A short duration storm which causes peak discharge on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharge on both basins, the peaks will be out of phase.

The Norfolk District of the U.S. Army Corps of Engineers developed the following criteria for a single project in the city of Virginia Beach, Va., and it should be used with extreme caution for other locations and/or situations.*

Joint Probability Analysis

Area Ratio	Frequencies for Coincidental Occurrence			
	10-year Design		100-year Design	
	Main Stream	Tributary	Main Stream	Tributary
10,000 to 1	1	10	2	100
	10	1	100	2
1,000 to 1	2	10	10	100
	10	2	100	10
100 to 1	5	10	25	100
	10	5	100	25
10 to 1	10	10	50	100
	10	10	100	50
1 to 1	10	10	100	100
	10	10	100	100

The design frequencies suggested in the table represent the extreme combinations of frequencies for each event. Experience has shown that these combinations will usually be critical. For example, the combinations of frequency for a 1,000 to 1 area ratio and a 100-year design frequency area 10- and 100-year frequencies. A 20- and 50-year frequency combination would have the same joint frequency, but the stage elevations, for instances, for these combinations will usually be less than the stages for the given combinations. Maximum stages should be reached when the 100-year storm on the tributary is coincident with the 10-year storm on the mainstream (greatest channel flow with moderate backwater effects) or when the 10-year storm on the tributary is coincidental with the 100-year storm on the mainstream (greatest backwater effect with moderate channel flow). Both cases must be analyzed.

* Rev 7/09

Appendix 6I-2 **Rainfall Coincident with Tidal EL. 2.5 FT and 5.4 FT**

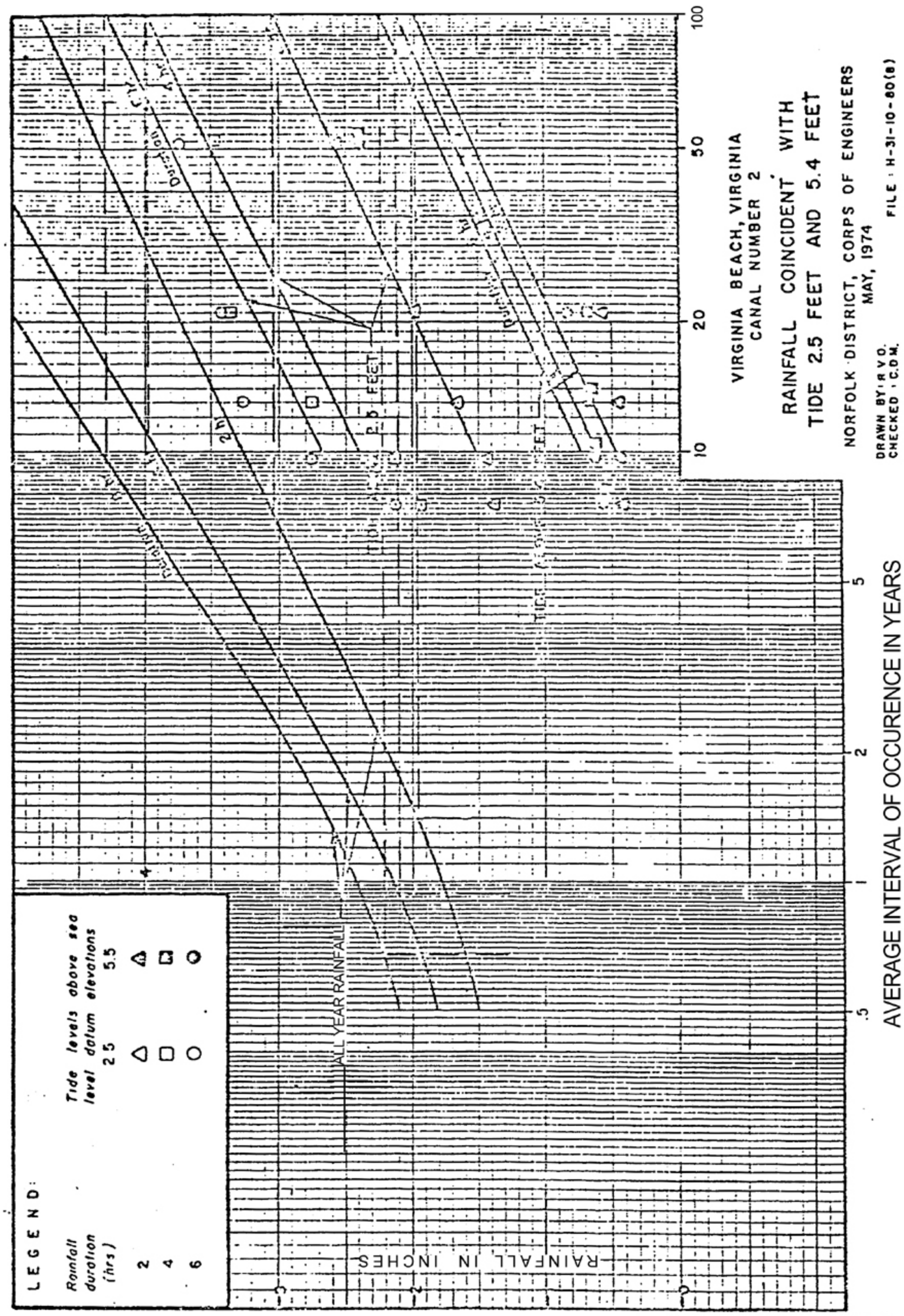
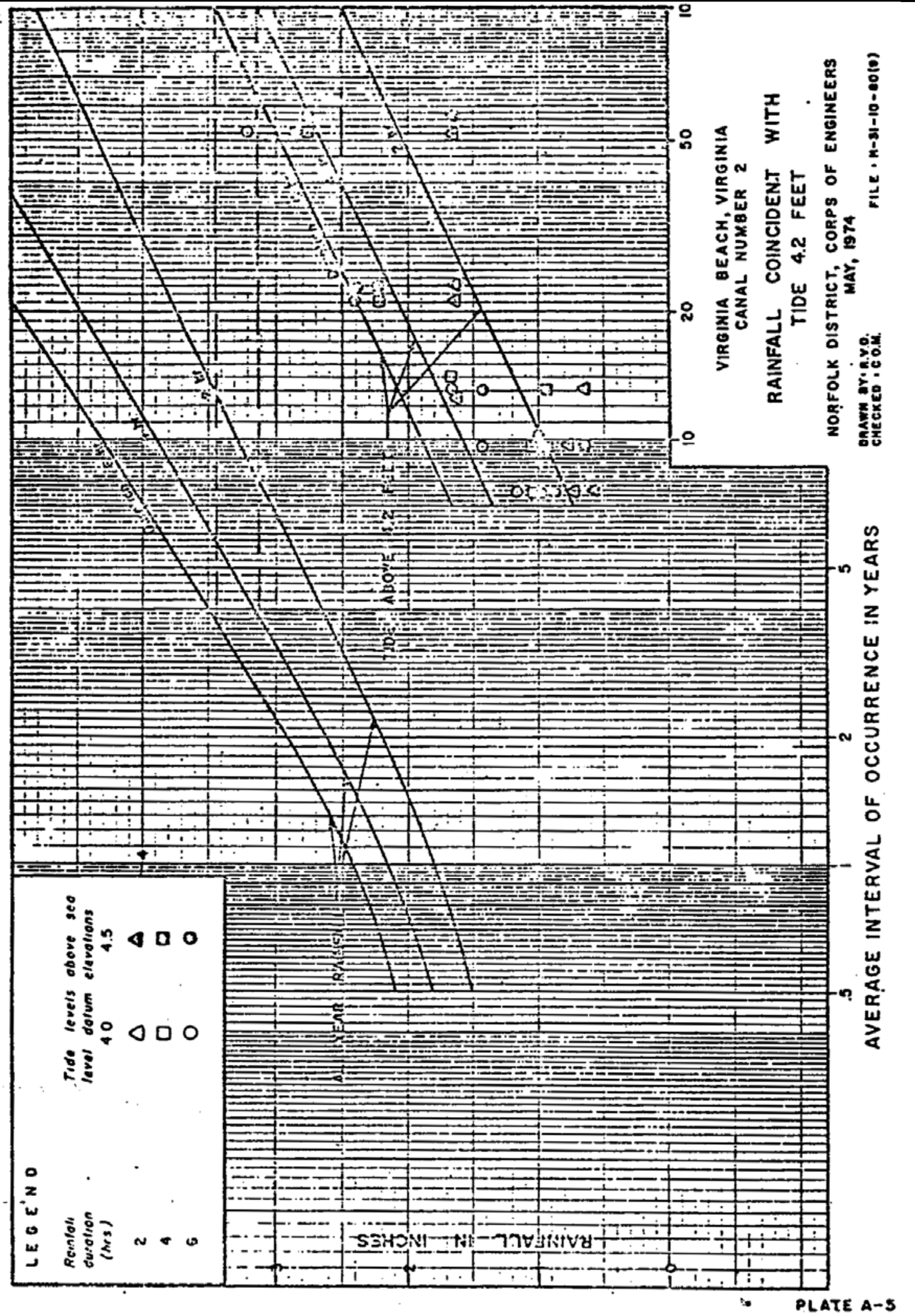


PLATE A-4

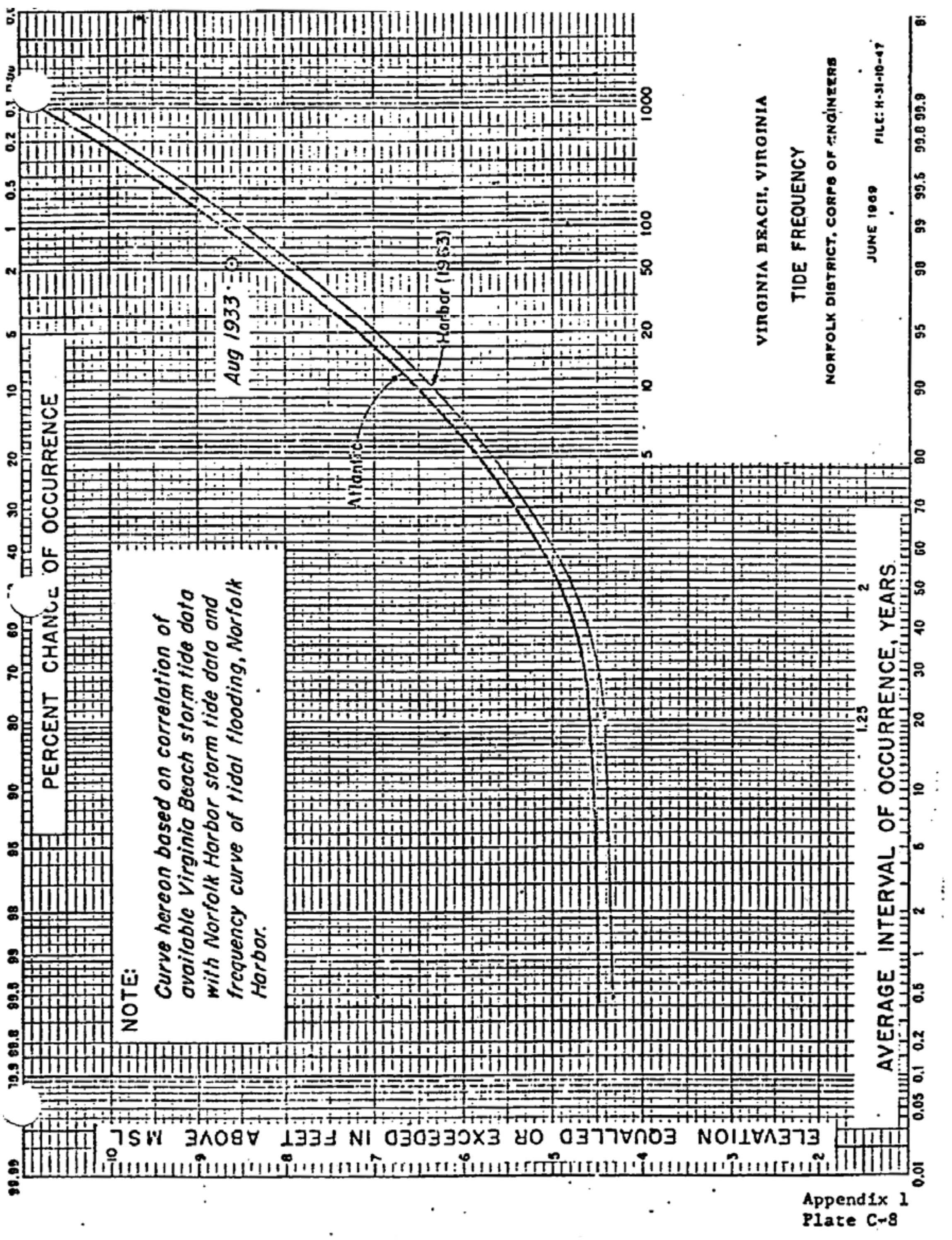
Source: U. S. Army Corps of Engineers

Appendix 6I-3 Rainfall Coincident with Tidal EL. 4.2 FT



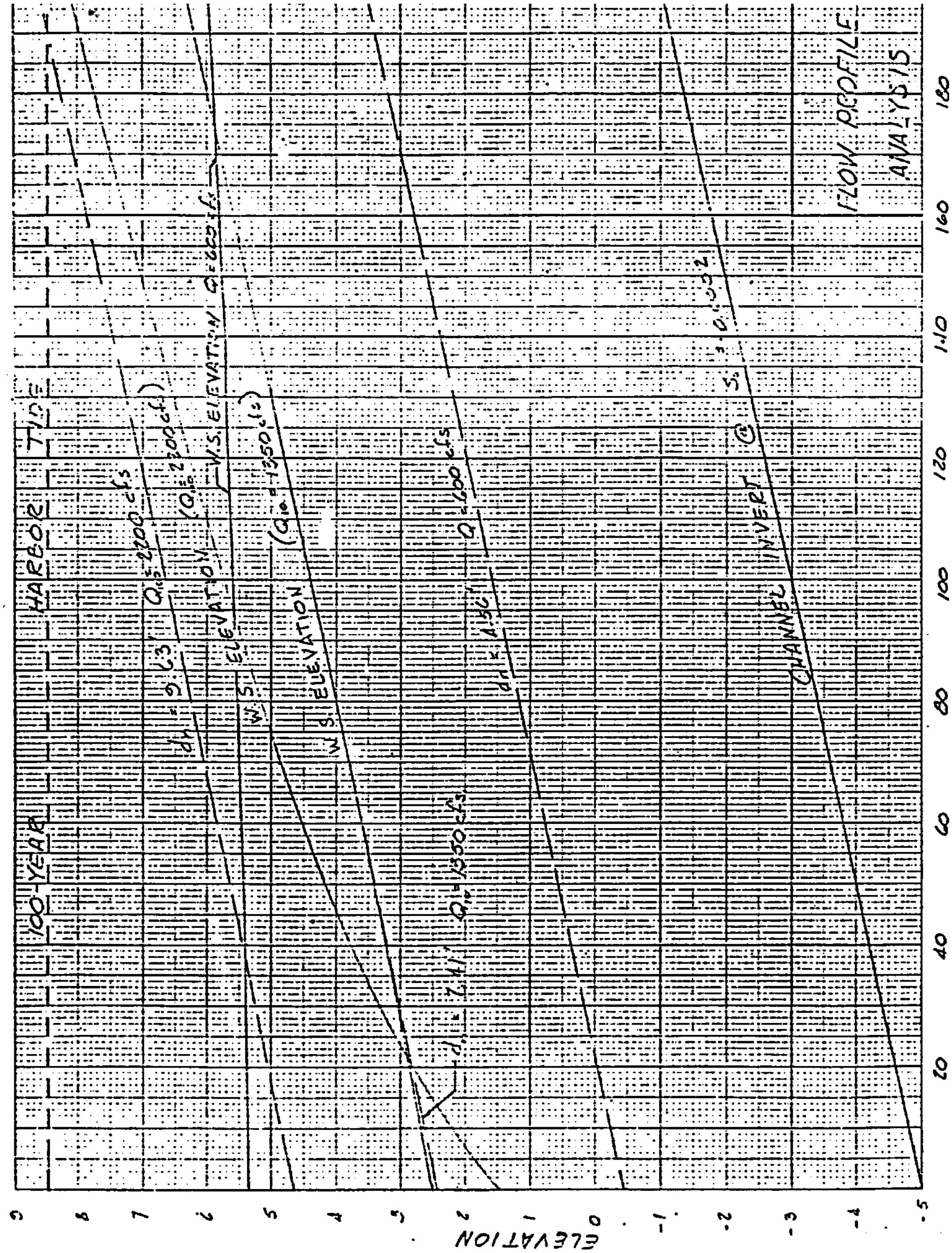
Source: U. S. Army Corps of Engineers

Appendix 6I-4 Tide Frequency, Virginia Beach



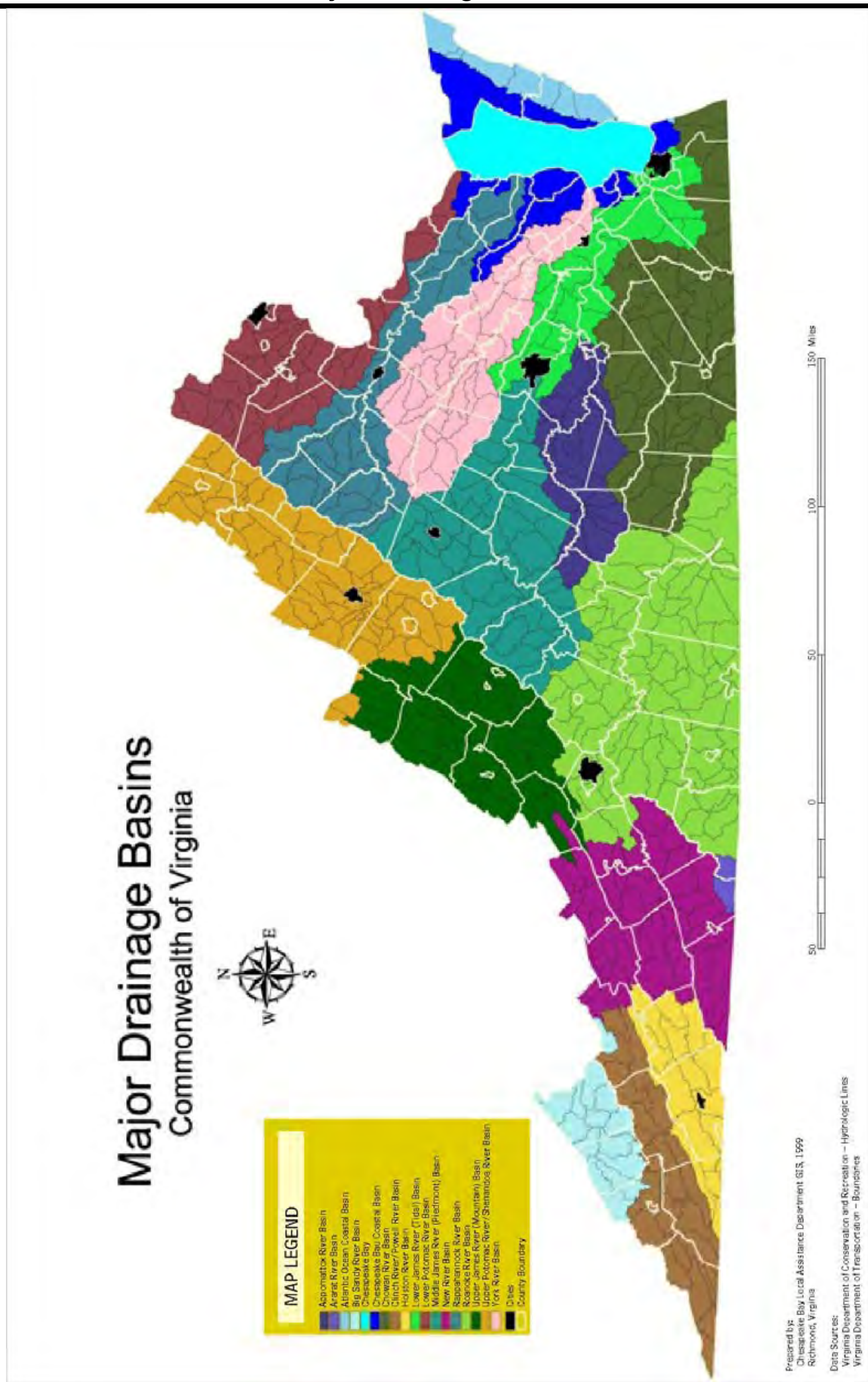
Source: U. S. Army Corps of Engineers

Appendix 6I-5 Flow Profile Analysis



Source: U. S. Army Corps of Engineers

Appendix 6J-1 Major Drainage Basins



Source: CBLAD

Appendix 6K-1 A and B Factors that Define Intensity-Duration-Frequency (IDF) Values for Use only with the Critical Storm Determination Procedure

The rainfall IDF curves are described by the equation:

$$i = \frac{a}{b + t_c}$$

Where:

i = Intensity, inches per hour (in/hr)
t_c = Rainfall duration, minutes (min)

The a and b factors describing the 2, 10 and 100-year IDF curves are provided in Appendix 6B-2.

The a and b factors are not based on NOAA “Atlas 14” Rainfall Precipitation Frequency data* and are therefore to be used only in conjunction with Equation 11.5 that estimates the “Critical Storm Duration” (T_d).

* Rev. 7/16

Appendix 6K-2 Regression Constants a and b for Virginia

		2 YEAR		10 YEAR		100 YEAR	
COUNTY	**#**	**A**	**B**	**A**	**B**	**A**	**B**
Arlington	00	119.34	17.86	178.78	20.66	267.54	22.32
Accomack	01	107.75	14.69	175.90	20.64	277.44	24.82
Albemarle	02	106.02	15.51	161.60	18.73	244.82	20.81
Allegheny	03	95.47	13.98	145.89	17.27	220.94	19.29
Amelia	04	112.68	15.11	173.16	18.81	266.77	22.13
Amherst	05	106.72	15.39	162.75	18.83	245.52	21.02
Appomattox	06	109.11	15.39	167.44	19.12	254.03	21.61
Augusta	07	84.21	10.44	135.74	14.54	210.02	16.99
Bedford	09	114.59	17.21	171.51	20.47	258.17	22.80
Bland	10	105.33	16.56	162.75	20.41	247.84	22.87
Botetourt	11	110.32	16.95	164.94	20.01	247.92	22.16
Brunswick	12	126.74	17.27	190.73	21.52	287.02	24.46
Buchanan	13	87.14	13.22	128.51	15.15	189.98	16.22
Buckingham	14	109.95	15.41	168.28	19.11	254.59	21.47
Campbell	15	110.26	15.76	167.27	19.18	252.65	21.56
Caroline	16	121.21	17.33	182.56	20.88	275.65	23.30
Carroll	17	119.79	18.65	188.13	23.81	288.94	27.06
Charles City	18	124.23	17.14	186.52	21.05	281.04	23.85
Charlotte	19	109.87	14.71	171.75	19.25	265.18	22.56
Chesterfield	20	124.66	17.55	186.15	21.03	277.94	23.26
Clarke	21	94.13	12.88	141.03	15.39	210.66	16.85
Craig	22	106.67	16.54	166.19	20.94	251.27	22.95
Culpeper	23	111.90	16.25	169.78	19.51	255.26	21.52
Cumberland	24	111.34	15.29	172.73	19.29	271.55	24.02
Dickenson	25	87.03	13.10	128.09	14.82	190.08	15.98
Dinwiddie	26	125.08	17.29	189.77	21.51	284.68	24.02
Essex	28	119.70	16.76	180.50	20.18	271.79	22.58
Fairfax	29	117.06	17.34	178.32	20.49	269.23	22.40
Fauquier	30	116.55	17.52	172.47	20.02	255.06	21.38
Floyd	31	121.22	19.16	185.59	23.38	281.91	26.26

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6K-2 Regression Constants a and b for Virginia

		2 YEAR		10 YEAR		100 YEAR	
COUNTY	**#**	**A**	**B**	**A**	**B**	**A**	**B**
Frederick	34	93.79	13.15	141.02	15.77	211.40	17.42
Giles	35	106.14	16.72	165.04	20.80	252.79	23.46
Gloucester	36	119.62	16.09	182.54	20.40	276.43	23.35
Goochland	37	114.42	15.95	177.24	19.93	269.07	22.27
Grayson	38	119.29	18.94	176.02	22.06	262.24	24.25
Green	39	105.71	15.10	159.92	18.20	241.18	20.34
Greensville	40	129.97	17.80	194.08	22.01	291.37	24.83
Halifax	41	111.92	15.14	173.81	19.52	267.09	22.70
Hanover	42	122.80	17.29	185.01	20.91	278.40	23.40
Henrico	43	123.51	17.35	185.51	21.13	277.61	23.44
Henry	44	116.19	17.33	177.84	21.34	270.32	24.01
Highland	45	90.13	12.61	134.38	15.02	199.74	16.50
Isle of Wight	46	125.69	17.02	190.34	21.71	287.14	24.73
James City	47	121.86	16.58	185.06	20.81	279.14	23.67
King George	48	120.31	17.28	181.05	20.50	273.29	22.83
King & Queen	49	113.84	15.29	179.09	19.95	275.98	23.15
King William	50	114.92	15.58	180.36	20.13	277.03	23.26
Lancaster	51	109.80	14.49	170.27	18.72	259.78	21.41
Lee	52	93.78	14.40	143.28	17.58	215.10	19.22
Loudoun	53	104.05	14.91	157.67	17.71	237.83	19.65
Louisa	54	112.63	15.89	174.35	19.72	265.20	22.11
Lunenberg	55	122.01	16.82	184.70	20.80	278.38	23.48
Madison	56	106.87	15.33	161.43	18.49	242.78	20.62
Mathews	57	118.61	15.83	180.56	20.17	274.12	23.29
Mecklenberg	58	121.77	16.55	184.54	20.74	278.33	23.48
Middlesex	59	110.72	14.57	172.76	19.15	264.49	22.13
Montgomery	60	118.78	19.21	176.95	22.39	262.93	24.17
Nelson	62	103.46	14.52	160.23	18.36	245.04	20.89
New Kent	63	121.03	16.58	183.93	20.72	277.89	23.51
Norfolk	64	124.88	17.02	190.64	22.14	288.73	25.60

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6K-2 Regression Constants a and b for Virginia

COUNTY	#	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Northampton	65	111.07	14.78	173.72	19.63	267.48	23.04
Northumberland	66	111.20	14.99	171.55	19.00	260.59	21.63
Nottoway	67	122.38	17.06	183.97	20.87	275.78	23.19
Orange	68	116.77	16.63	178.14	20.19	270.55	22.72
Page	69	84.19	10.29	135.43	14.29	209.57	16.86
Patrick	70	123.68	19.26	189.08	23.60	284.78	26.12
Powhatan	72	114.14	15.64	175.93	19.65	266.86	22.15
Pittsylvania	71	112.30	16.02	173.58	20.27	263.51	22.98
Prince Edward	73	111.01	15.06	172.73	19.29	264.28	22.20
Prince George	74	126.22	17.46	188.62	21.39	283.12	24.09
Virginia Beach	75	129.20	17.84	196.25	22.74	294.74	26.33
Prince William	76	116.04	17.08	176.18	20.19	266.75	22.36
Pulaski	77	117.44	18.71	182.33	23.39	279.39	26.49
Rappahannock	78	104.86	15.05	159.40	18.34	239.30	20.19
Richmond	79	117.41	16.23	177.35	19.85	267.20	22.24
Roanoke	80	117.53	18.79	174.97	21.80	261.95	23.81
Rockbridge	81	84.23	10.46	143.41	15.89	229.43	19.56
Rockingham	82	83.83	10.55	128.80	13.37	195.24	15.29
Russell	83	92.64	14.17	143.00	17.32	216.40	19.36
Scott	84	92.64	14.17	143.00	17.32	216.40	19.36
Smyth	86	106.19	16.57	169.30	21.37	262.49	24.57
Southampton	87	129.91	17.77	195.84	22.34	294.40	25.43
Spotsylvania	88	117.31	16.86	179.21	20.48	269.84	22.55
Stafford	89	118.72	17.34	179.62	20.64	270.74	22.79
Surry	90	124.79	16.97	188.62	21.39	283.36	24.16
Sussex	91	130.37	18.03	193.23	21.91	287.99	24.56
Tazewell	92	91.25	13.56	141.61	17.04	217.59	19.48
Warren	93	89.03	11.53	137.69	14.73	210.46	16.87
Washington	95	106.65	16.86	162.19	20.02	244.60	21.98
Westmoreland	96	114.40	15.76	174.96	19.47	266.16	22.12

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.

Appendix 6K-2 Regression Constants a and b for Virginia

COUNTY	#	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Wise	97	89.83	13.49	132.05	15.44	194.10	16.35
Wythe	98	116.78	18.83	174.91	22.13	261.68	24.25
York	99	122.93	16.72	186.78	21.22	282.80	24.39

CITIES	#'s	2 YEAR A	2 YEAR B	10 YEAR A	10 YEAR B	100 YEAR A	100 YEAR B
Richmond	127/43	122.47	17.10	185.51	21.13	278.85	23.60
Hampton	114/27	123.93	16.94	186.78	21.22	283.18	24.56
Lynchburg	118/15	107.39	15.15	166.87	19.37	255.02	22.08
Suffolk	133/61	129.97	17.80	196.63	22.61	298.69	26.35
Newport News	121/94	126.11	17.37	189.27	21.62	285.24	24.71

Source: Virginia Stormwater Management Handbook, 1st Ed., Vol. II, 1999.