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, or where flood routing through culverts is used, 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.[*]

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.)

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- Watershed development potential
- Type of precipitation (rain, snow, hail, or combinations thereof)

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 publication, AASHTO Highway Drainage Guidelines.

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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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 may be used for all routine designs provided there is at least 10 years of continuous or synthesized flow records for 10-year discharge estimates and 25 years for 100-year discharge estimates
- Suitable computer programs such as the USACE's HEC-HMS & HEC-1 and the NRCS' EFH-2,* TR-55 and TR-20 may be used to facilitate tedious hydrologic

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calculations. The TR-55 method has been found best suited for drainage areas between 200 and 2000 acres. 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-year discharges specified in the FEMA flood insurance study should be used to analyze the impacts of 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 subject to receipt of necessary regulatory approvals.

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 should 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.

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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. For other specific design frequency criteria, the user is directed to the various design chapters for channels, culverts, storm drains, bridges, etc.*

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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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, should also be designed for the 2% event.

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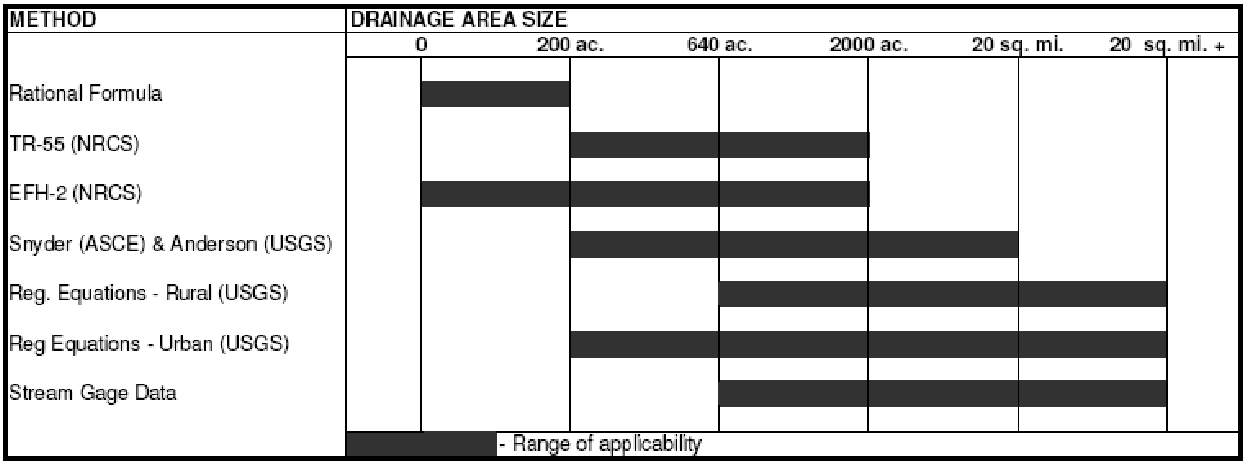
6.3.2 Peak Discharge Method Selection

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



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:

$$T_t = \frac{L}{3600V} \tag{6.1}$$

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.

$$t_c = T_{t1} + T_{t2} + \ldots T_{tm} \tag{6.2}$$

Where:

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

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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 ft/s).

¹ From AASHTO Model Drainage Manual -2005 page 7-35*

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 percent 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.

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

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probability, or the chance that the flood will be equaled or exceeded in a given year. If a flood has a 20 percent 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 $100 \div \text{RI}$.

The designer should note that the 5-year flood is not one that will necessarily be equaled or exceeded in any given five years. There is a 20 percent chance that the flood will be equaled or exceeded in any year; therefore, the 5-year flood could conceivably occur in several consecutive years, or it may not occur for many years. The same reasoning applies to floods with other return periods.

6.4.2.2 Design Frequency

Cross Drainage: 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
- Overtop the highway
- 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 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. If a storm drain system is designed to pass the 10[*]-year flow, it would not be uncommon for a larger event storm (such as the 100-year event) to cause inundation of the roadway. In this scenario, the

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larger event storm should be used as the “check” or review frequency in the hydraulic analysis.

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-year event. In some cases, a flood event larger than the 100-year 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-year rainfall will produce the 10-year flood. Depending on antecedent soil moisture conditions, and other hydrologic parameters this may be true or there may not be a direct relationship between rainfall and flood frequency.

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 every county and major city in the state. 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. A listing of gaged sites can be found at the following website: >http://www.afws.net/<.
- 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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6.4.3 Hydrologic Analysis Procedure Flowchart

The hydrologic analysis procedure flowchart, Figure 6-2 shows the steps needed for the hydrologic analysis, and the designs that will use the hydrologic estimates.

6.4.4 Peak Discharge Methods

6.4.4.1 Rational Method

6.4.4.1.1 Introduction

The Rational Method is recommended for estimating the design storm peak runoff for areas as large as 200 acres. In low-lying tidewater areas [*]where the terrain is flat, the Rational Method can be considered for areas up to 300 acres. While the Rational Method is relatively straightforward to apply, its concepts are quite sophisticated. 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.

6.4.4.1.2 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 highway crossings and dams that exist in the drainage area should be investigated to see how they affect the design flows
- Charts, graphs, and tables included in this chapter are not intended to replace reasonable and prudent engineering judgment in the design process

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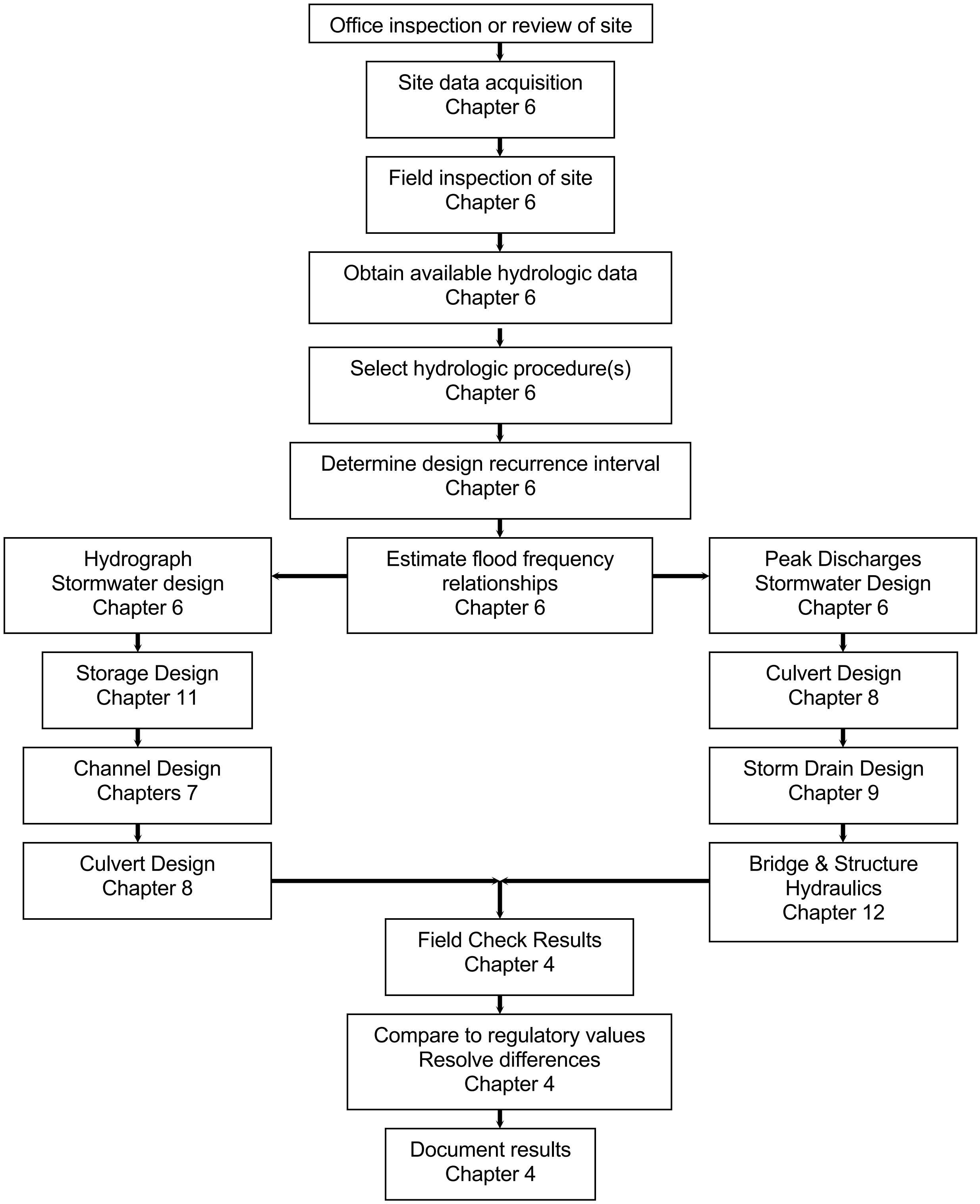

Figure 6-2. Hydrologic Analysis Procedure Flowchart

6.4.4.1.3 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 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)*

6.4.4.1.4 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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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.

6.4.4.1.5 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 be greater than 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 be less than or equal to 1.0

6.4.4.1.6 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*.

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Time of concentration (t_c) for most drainage areas less than about 200 acres will normally be comprised of overland flow (OLF), channel flow or concentrated flow (CF), and conveyance flow in man made 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 feet.

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.

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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 feet. 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 inches. 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.

6.4.4.1.7 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 percent. The average range of C-values should be used for intermediate areas where the majority of grades and slopes range from 2 to 5

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percent. The highest range of C-values should be used for steep areas (grades greater than 5 percent), 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.

6.4.4.1.8 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 feet in urban areas and 400 feet in rural areas.

6.4.4.2 Anderson Method

6.4.4.2.1 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.

6.4.4.2.2 Application
This method was developed from analysis of drainage basins in Northern Virginia with drainage area sizes up to 570 square miles.

6.4.4.2.3 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.

6.4.4.2.4 Equations
The equation for the Anderson Method is as follows:

[*] Rev 9/09

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

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 percent of L

$K=1+0.015I$

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)

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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.

6.4.4.3 Snyder Method

6.4.4.3.1 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.

6.4.4.3.2 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.

6.4.4.3.3 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:

$$Q_p = 500AI_R \qquad (6.7)$$

Where:

Q_p = Peak discharge, cubic feet per second (cfs)
A = Basin area, square miles (sq. mi.)

* Rev 9/09

$$I_R = \left[\frac{\text{Runoff}}{T_c}\right], \text{ inches per hour (in/hr)}$$

T_c = Time of concentration, hours (hrs)

Refer to the Design Procedure and Sample Problem, Section 6.5.2.3, for the computation of I_R and T_c.

6.4.4.4 Rural Regression Method

6.4.4.4.1 Introduction

Regional regression equations are a commonly accepted method for estimating peak flows at ungaged sites or sites with insufficient data. Also, they have been shown to be accurate, reliable, and easy to use as well as providing consistent findings when applied by different hydraulic engineers (Newton and Herrin, 1982). 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 meteorologic characteristics.

For details on the application of Rural Regression Equations in Virginia, the user is directed to the following publication: "Methods for Estimating the Magnitude and Frequency of Peak Discharges of Rural, Unregulated Streams in Virginia," U.S.G.S. Water Resource Investigations Report 94-4148 (1995).

6.4.4.4.2 Application

The regression equations should be used routinely in design for drainage areas greater than one square mile. Where there is stream gage data, the findings from a Log Pearson III 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. Where there is less stream gage record, reasonable and prudent judgment along with consideration of the standard regression error should be used in reaching a design decision.

In instances where both multiple-parameter and area-only equations are available, it is recommended that the discharges be computed using both sets of equations. The designer should then carefully examine both sets of results and either choose the set considered most appropriate or use a weighting procedure to combine the two.

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 area-only equations and, if available, the multiple-parameter equations as outlined in the preceding paragraph.

For sites where the watershed spans two or more regions the procedure described in U.S.G.S. "Water-Resources investigation Report 94-4148" is acceptable. However, there may be times when the designer feels that the results based only on one region may be more applicable than prorating the discharges from two or more individual regional calculations. The department feels the designer should have the latitude to make such a decision. (The U.S.G.S. report 94-4148 is available on their web site at http://pubs.usgs.gov/wri .)

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 department recommends the following procedure in lieu of that presented in the U.S.G.S. Water- Resources Investigation Report 94-4148.

a) Determine the 2 thru 500 yr peak discharges at the gage using either the "s" value from Appendix 2 in the back of this publication or by conducting an independent log-Pearson Type III frequency analysis.

b) Prorate the peak discharges determined for the gage to the site using either the departments "PQTRANS" computer program or one of the two (or a combination of the two) equations listed below:

U.S.G.S transfer formula[1]:

$$Q_U = Q_k \left(\frac{A_u}{A_k}\right)^e$$

where: Q_U = The calculated magnitude of the peak at the site, in cfs

Q_k = The magnitude of the peak discharge at the gage, in cfs

Au= The drainage area at the site, in mi^2

A_k = The drainage area at the gage, in mi^2

e = A value that varies between 0.7 and 0.8 with 0.7 usually being preferred

[1] This formula was provided the department by the U.S. Geological Survey some years ago and has been used both extensively and effectively.

The N.R.C.S. formerly S.C.S. equation[2]

$$Q_U = Q_K \left[\frac{A_U^{\left(\frac{.894}{A_U^{.048}} - 1\right)}}{A_K^{\left(\frac{.894}{A_K^{.048}} - 1\right)}} \right]^{*}$$

where: Q_U = The calculated magnitude of the peak discharge at the site, in cfs

Q_k = The magnitude of the peak discharge at the gage, in cfs

Au= The drainage area at the site, in mi^2

A_k = The drainage area at the gage, in mi^2

This formula is provided in the N.R.C.S. formerly S.C.S. “NEH-4” publication

[*] Rev 9/09

6.4.4.4.3 Hydrologic Regions

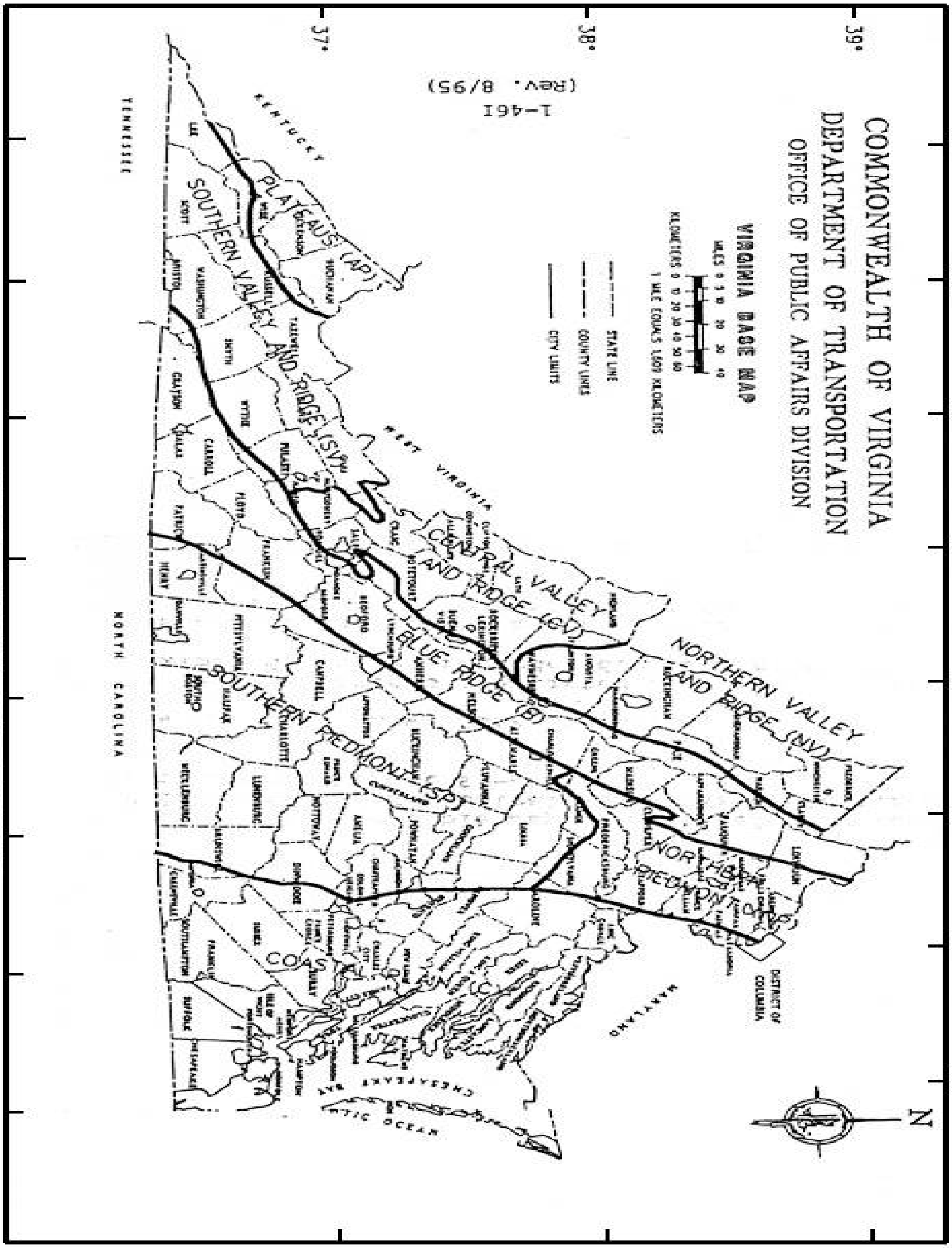
Regression analyses use stream gage data to define hydrologic regions. These are geographic regions having very similar flood frequency relationships and, as such, commonly display similar watershed, channel and meteorological characteristics; often termed hydrologically homogeneous geographic areas. Because of the distance between stream gages and sometimes due to the foregoing mixed flood population events, the regional boundaries cannot be considered as precise. Figure 6-3* shows the hydrologic regional boundaries for Virginia.

Problems related to hydrologic boundaries may occur in selecting the appropriate regression equation. First, the watershed of interest may lie partly within two or more hydrologic regions or a problem may occur when a watershed lies totally within a hydrologic region but close to a hydrologic region boundary. Another problem may occur when the watershed is partly or totally within an area subject to mixed population floods. In these instances, care must be exercised in using regression equations. A field visit is recommended to first collect all available historical flood data, as well as to compare the project's watershed characteristics with those of the abutting hydrologic regions.

6.4.4.4.4 Equations

Tables 6-4 and 6-5 contain the multi-parameter and drainage-area-only regression equations for estimating peak discharges in Virginia.

* Rev 9/09



PEAK - DISCHARGE REGIONS
(modified from Feeneman, 1938)

Figure 6-3. Peak Discharge Regions

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

Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record	Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record
Coastal Plain (C) – 29 Sites			**Northern Valley and Ridge (NV) – 29 Sites**		
$Q_{(2)} = 2.4(A)^{1.005}(SL)^{0.852}$	57.1	1.4	$Q_{(2)} = 73.0(A)^{0.955}(L)^{-0.307}(F)^{0.041}$	37.8	3.6
$Q_{(5)} = 4.0(A)^{0.999}(SL)^{0.884}$	59.7	2.5	$Q_{(5)} = 119(A)^{0.953}(L)^{-0.290}(F)^{0.063}$	33.5	7.4
$Q_{(10)} = 4.9(A)^{1.005}(SL)^{0.932}$	59.4	3.8	$Q_{(10)} = 153(A)^{0.944}(L)^{-0.273}(F)^{0.081}$	31.4	12.2
$Q_{(25)} = 6.0(A)^{1.016}(SL)^{0.998}$	61.0	5.6	$Q_{(25)} = 196(A)^{0.931}(L)^{-0.251}(F)^{0.107}$	30.9	18.5
$Q_{(50)} = 6.8(A)^{1.024}(SL)^{1.044}$	64.1	6.7	$Q_{(50)} = 228(A)^{0.926}(L)^{-0.241}(F)^{0.124}$	31.9	22.2
$Q_{(100)} = 7.6(A)^{1.033}(SL)^{1.088}$	68.5	7.5	$Q_{(100)} = 263(A)^{0.925}(L)^{-0.237}(F)^{0.138}$	33.8	24.4
$Q_{(200)} = 8.3(A)^{1.042}(SL)^{1.130}$	73.9	8.0	$Q_{(200)} = 300(A)^{0.928}(L)^{-0.239}(F)^{0.149}$	36.3	25.3
$Q_{(500)} = 9.2(A)^{1.055}(SL)^{1.185}$	82.7	8.5	$Q_{(500)} = 356(A)^{0.936}(L)^{-0.247}(F)^{0.161}$	40.8	25.1
Northern Piedmont (NP) – 19 Sites			**Central Valley and Ridge (CV) – 34 Sites**		
$Q_{(2)} = 179(A)^{0.655}$	51.1	1.6	$Q_{(2)} = 89.2(A)^{0.788}$	31.0	4.8
$Q_{(5)} = 317(A)^{0.644}$	49.3	3.3	$Q_{(5)} = 222(A)^{0.712}$	29.3	8.7
$Q_{(10)} = 438(A)^{0.641}$	50.2	4.9	$Q_{(10)} = 372(A)^{0.668}$	28.6	12.9
$Q_{(25)} = 626(A)^{0.640}$	53.8	6.7	$Q_{(25)} = 647(A)^{0.620}$	29.5	17.5
$Q_{(50)} = 793(A)^{0.640}$	58.0	7.7	$Q_{(50)} = 918(A)^{0.591}$	31.4	19.4
$Q_{(100)} = 984(A)^{0.641}$	63.5	8.2	$Q_{(100)} = 1{,}254(A)^{0.565}$	34.1	20.2
$Q_{(200)} = 1{,}200(A)^{0.643}$	70.1	8.5	$Q_{(200)} = 1{,}665(A)^{0.542}$	37.4	20.2
$Q_{(500)} = 1{,}535(A)^{0.646}$	80.4	8.6	$Q_{(500)} = 2{,}354(A)^{0.514}$	42.6	19.5
Southern Piedmont (SP) – 67 Sites			**Southern Valley and Ridge (SV) – 35 Sites**		
$Q_{(2)} = 21.6(A)^{0.881}(E)^{0.310}(L)^{-0.423}$	40.2	2.8	$Q_{(2)} = 45.7(A)^{0.880}$	45.0	1.7
$Q_{(5)} = 31.9(A)^{0.854}(E)^{0.351}(L)^{-0.417}$	35.7	6.2	$Q_{(5)} = 89.5(A)^{0.825}$	43.4	2.6
$Q_{(10)} = 38.8(A)^{0.848}(E)^{0.379}(L)^{-0.430}$	35.5	9.3	$Q_{(10)} = 127(A)^{0.800}$	44.2	3.3
$Q_{(25)} = 54.8(A)^{0.852}(E)^{0.392}(L)^{-0.463}$	38.0	12.3	$Q_{(25)} = 181(A)^{0.774}$	46.6	4.2
$Q_{(50)} = 74.3(A)^{0.860}(E)^{0.390}(L)^{-0.495}$	41.4	13.6	$Q_{(50)} = 228(A)^{0.759}$	49.1	4.7
$Q_{(100)} = 101(A)^{0.869}(E)^{0.382}(L)^{-0.529}$	45.7	14.2	$Q_{(100)} = 281(A)^{0.745}$	52.0	5.2
$Q_{(200)} = 136(A)^{0.879}(E)^{0.373}(E)^{-0.561}$	50.6	14.4	$Q_{(200)} = 339(A)^{0.733}$	55.3	5.5
$Q_{(500)} = 197(A)^{0.893}(E)^{0.361}(L)^{-0.602}$	58.0	14.2	$Q_{(500)} = 425(A)^{0.718}$	60.2	5.7
Blue Ridge (B) – 54 Sites			**Appalachian Plateaus (AP) – 17 Sites**		
$Q_{(2)} = 95.4(A)^{0.760}$	33.4	4.0	$Q_{(2)} = 262(A)^{0.749}(SL)^{-0.175}$	33.6	3.5
$Q_{(5)} = 201(A)^{0.726}$	34.1	6.5	$Q_{(5)} = 134(A)^{0.844}(SL)^{0.032}$	21.3	12.2
$Q_{(10)} = 298(A)^{0.710}$	35.5	8.8	$Q_{(10)} = 103(A)^{0.880}(SL)^{0.136}$	18.1	23.5
$Q_{(25)} = 450(A)^{0.695}$	38.8	11.0	$Q_{(25)} = 90.4(A)^{0.902}(SL)^{0.227}$	19.3	31.5
$Q_{(50)} = 584(A)^{0.687}$	42.2	12.0	$Q_{(50)} = 87.0(A)^{0.910}(SL)^{0.280}$	21.9	33.0
$Q_{(100)} = 735(A)^{0.680}$	46.2	12.5	$Q_{(100)} = 85.7(A)^{0.916}(SL)^{0.324}$	24.7	33.4
$Q_{(200)} = 907(A)^{0.674}$	50.7	12.6	$Q_{(200)} = 85.0(A)^{0.920}(SL)^{0.365}$	27.9	33.5
$Q_{(500)} = 1{,}165(A)^{0.667}$	56.7	12.8	$Q_{(500)} = 85.5(A)^{0.923}(SL)^{0.411}$	31.9	33.5

A=Drainage area, square miles (sq.mi.); E= Average basin elevation, feet above sea level (ft); F=Actual forestation percentage (%) + 1; L=Main channel length, miles (mi); SL=Main channel slope, feet per mile (ft/mi). Peak discharge regions are shown in Figure 6-3[*].

[*] Rev 9/09

Table 6-5. Drainage-Area-Only Regional Regression Equations for Estimating Peak Discharges of Streams in Virginia

Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of Record	Regression Equation	Standard Error of Prediction (percent)	Equivalent Years of record
Coastal Plain (C) – 29 Sites			**Northern Valley and Ridge (NV) – 29 Sites**		
$Q_{(2)} = 57(A)^{0.589}$	55.8	1.4	$Q_{(2)} = 72(A)^{0.785}$	39.2	3.4
$Q_{(5)} = 106(A)^{0.569}$	58.3	2.5	$Q_{(5)} = 128(A)^{0.794}$	35.0	6.9
$Q_{(10)} = 153(A)^{0.555}$	62.1	3.5	$Q_{(10)} = 178(A)^{0.796}$	32.7	11.4
$Q_{(25)} = 230(A)^{0.539}$	68.6	4.5	$Q_{(25)} = 254(A)^{0.797}$	32.1	17.6
$Q_{(50)} = 302(A)^{0.528}$	74.1	5.2	$Q_{(50)} = 317(A)^{0.798}$	33.2	21.1
$Q_{(100)} = 388(A)^{0.518}$	80.2	5.7	$Q_{(100)} = 386(A)^{0.800}$	35.3	23.1
$Q_{(200)} = 489(A)^{0.509}$	86.7	6.2	$Q_{(200)} = 461(A)^{0.802}$	38.2	23.7
$Q_{(500)} = 652(A)^{0.497}$	96.1	6.7	$Q_{(500)} = 569(A)^{0.805}$	43.2	23.2
Northern Piedmont (NP) – 19 Sites			**Central Valley and Ridge (CV) – 34 Sites**		
$Q_{(2)} = 179(A)^{0.655}$	51.1	1.6	$Q_{(2)} = 89\ (A)^{0.788}$	31.0	4.8
$Q_{(5)} = 317(A)^{0.644}$	49.3	3.3	$Q_{(5)} = 222(A)^{0.712}$	29.3	8.7
$Q_{(10)} = 438(A)^{0.641}$	50.2	4.9	$Q_{(10)} = 372(A)^{0.668}$	28.6	12.9
$Q_{(25)} = 626(A)^{0.640}$	53.8	6.7	$Q_{(25)} = 647(A)^{0.620}$	29.5	17.5
$Q_{(50)} = 793(A)^{0.640}$	58.0	7.7	$Q_{(50)} = 918(A)^{0.591}$	31.4	19.4
$Q_{(100)} = 983(A)^{0.641}$	63.5	8.2	$Q_{(100)} = 1{,}254(A)^{0.565}$	34.1	20.2
$Q_{(200)} = 1{,}200(A)^{0.643}$	70.1	8.5	$Q_{(200)} = 1{,}665(A)^{0.542}$	37.4	20.2
$Q_{(500)} = 1{,}535(A)^{0.646}$	80.4	8.6	$Q_{(500)} = 2{,}354(A)^{0.514}$	42.6	19.5
Southern Piedmont (SP) – 67 Sites			**Southern Valley and Ridge (SV) – 35 Sites**		
$Q_{(2)} = 122(A)^{0.635}$	40.2	2.8	$Q_{(2)} = 46(A)^{0.880}$	45.0	1.7
$Q_{(5)} = 233(A)^{0.610}$	38.7	5.4	$Q_{(5)} = 90(A)^{0.825}$	43.4	2.6
$Q_{(10)} = 335(A)^{0.596}$	38.5	8.0	$Q_{(10)} = 127(A)^{0.800}$	44.2	3.3
$Q_{(25)} = 504(A)^{0.581}$	40.8	10.9	$Q_{(25)} = 181(A)^{0.774}$	46.6	4.2
$Q_{(50)} = 661(A)^{0.570}$	43.8	12.3	$Q_{(50)} = 228(A)^{0.759}$	49.1	4.7
$Q_{(100)} = 849(A)^{0.559}$	47.7	13.2	$Q_{(100)} = 281(A)^{0.745}$	52.0	5.2
$Q_{(200)} = 1{,}070(A)^{0.549}$	52.2	13.7	$Q_{(200)} = 339(A)^{0.733}$	55.3	5.5
$Q_{(500)} = 1{,}418(A)^{0.538}$	59.0	13.9	$Q_{(500)} = 425(A)^{0.718}$	60.2	5.7
Blue Ridge (B) – 54 Sites			**Appalachian Plateaus (AP) – 17 Sites**		
$Q_{(2)} = 95(A)^{0.760}$	33.4	4.0	$Q_{(2)} = 93(A)^{0.840}$	32.7	3.7
$Q_{(5)} = 201(A)^{0.726}$	34.1	6.5	$Q_{(5)} = 162(A)^{0.828}$	19.9	14.0
$Q_{(10)} = 298(A)^{0.710}$	35.5	8.8	$Q_{(10)} = 230(A)^{0.809}$	17.8	24.3
$Q_{(25)} = 450(A)^{0.695}$	38.8	11.0	$Q_{(25)} = 341(A)^{0.784}$	20.7	27.5
$Q_{(50)} = 584(A)^{0.687}$	42.2	12.0	$Q_{(50)} = 441(A)^{0.767}$	24.0	26.5
$Q_{(100)} = 735(A)^{0.680}$	46.2	12.5	$Q_{(100)} = 557(A)^{0.751}$	27.8	25.2
$Q_{(200)} = 907(A)^{0.674}$	50.7	12.6	$Q_{(200)} = 691(A)^{0.736}$	31.4	24.2
$Q_{(500)} = 1{,}165(A)^{0.667}$	56.7	12.8	$Q_{(500)} = 902(A)^{0.717}$	36.3	23.1

A=Drainage area, square miles (sq. mi.) Peak-discharge regions are shown in Figure 6-3*.

* Rev 9/09

6.4.4.4.5 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.

6.4.4.5 Urban Regression Method
6.4.4.5.1 Introduction
Regression equations developed by the USGS (Sauer et al., 1983) as part of a nationwide project can be used to estimate peak runoff for urban watershed conditions.

Sauer et al. (1983) provides* a set of three and seven-parameter equations. The three-parameter equations display almost as high a degree of reliability as the seven-parameter equations and are easier to use. However, if it is felt that employing the additional variables will yield more accurate results, the seven-parameter equations may be used or both sets of equations used and the results compared. If the results using either or both the three and seven-parameter equations appear questionable they should be checked with the Anderson and/or Snyder methods. The equations account for regional runoff variations through the use of the equivalent rural peak runoff rate (RQ). The equations adjust RQ to an urban condition using the basin development factor (BDF) and the percentage of impervious area (IA).

6.4.4.5.2 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, predominately urban in nature.

6.4.4.5.3 Characteristics
The basin development factor (BDF) is an index of the prevalence of urban drainage characteristics. The BDF can range from 0-12. A value of zero (0) for BDF indicates that urban drainage characteristics are not prevalent, but does not necessarily mean the basin is non-urban. A value of 12 indicates full development throughout the basin. Before a BDF factor is assigned, the drainage area is subdivided into three sections as shown in Figure 6-4. Each section is then assigned a code of either zero (0) or one (1) based upon the following four characteristics: channel improvement, channel lining, storm drain, and curb and gutter. The total of the 12 codes provides the BDF factor.

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* Rev 9/09

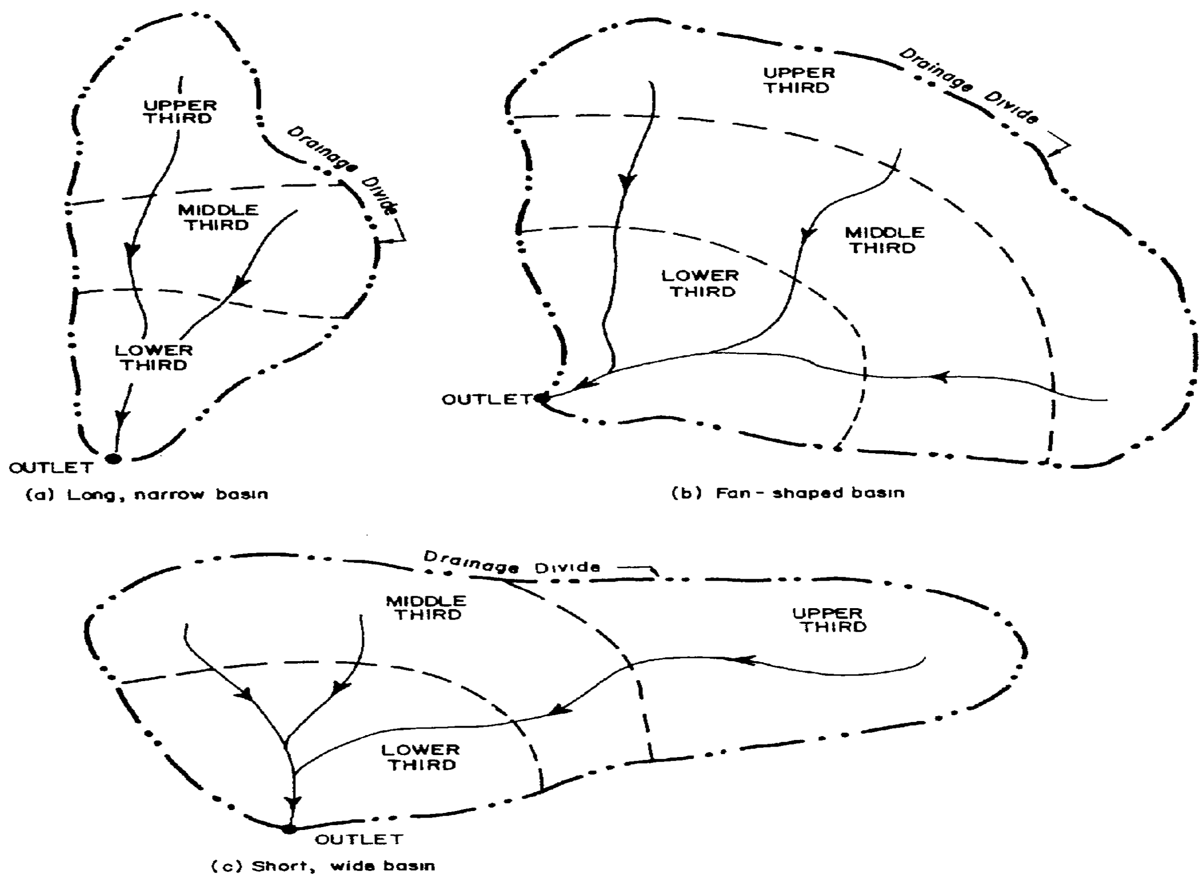

Figure 6-4*. Basin Subdivisions

Channel Improvement

If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a BDF code of one (1) is assigned. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a BDF code of zero (0) is assigned.

Channel Linings

If more than 50 percent of the length of the main drainage and principal tributaries has been lined with an impervious material, such as concrete, then a BDF code of one (1) is assigned to this aspect. If less than 50 percent of these channels are lined, then a BDF code of zero (0) is assigned. The presence of channel linings is a good indication that channel improvements have been performed and signifies a more highly developed drainage system.

* Rev 9/09

Storm Drain

Storm drains are enclosed drainage structures (usually pipes) frequently used on the secondary tributaries which receive drainage directly from streets or parking lots. Many of these drains empty into open channels; in some basins, however, they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a sub-basin consists of storm drains, a BDF code of one (1) is assigned to this aspect; if less than 50 percent, then a BDF code of zero (0). Note that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, the aspects of channel improvements and channel linings would also be assigned a BDF code of one (1).

Curb and Gutter

If more than 50 percent of a sub-basin is urbanized, and if more than 50 percent of the streets and highways in the sub-basin are constructed with curbs and gutters, then a BDF code of one (1) would be assigned to this aspect. Otherwise, it would receive a BDF code of zero (0). Drainage from curb and gutter streets frequently empties into storm drains.

6.4.4.5.4 Equations

The nationwide 3-parameter* equations for urban conditions take the following general form:

$$UQ_T = CA^{b1}(13\text{-}BDF)^{b2}RQ_T^{\ b3} \qquad (6.8.1)$$

Where:

UQ_T	=	Peak discharge for the urban watershed for recurrence interval T, cubic feet per second (cfs)
C	=	Regression constant (dimensionless)
A	=	Contributing drainage area, square miles (sq. mi.)
BDF	=	Basin development factor (dimensionless)
RQ_T	=	Peak discharge for an equivalent rural drainage basin in the same hydrologic area as the urban basin and for recurrence interval T, cubic feet per second (cfs)
b1, b2, b3	=	Regression exponents

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Lists the 3-parameter equations for various storm frequencies.

Table 6-6.1 3-Parameter Urban Equations

$UQ_2 = 13.2A^{0.21}(13\text{-}BDF)^{-0.43}RQ_2^{\ 0.73}$
$UQ_5 = 10.6A^{0.17}(13\text{-}BDF)^{-0.39}RQ_5^{\ 0.77}$
$UQ_{10} = 9.51A^{0.16}(13\text{-}BDF)^{-0.36}RQ_{10}^{\ 0.79}$
$UQ_{25} = 8.68A^{0.15}(13\text{-}BDF)^{-0.34}RQ_{25}^{\ 0.80}$
$UQ_{50} = 8.04A^{0.15}(13\text{-}BDF)^{-0.32}RQ_{50}^{\ 0.81}$
$UQ_{100} = 7.70A^{0.15}(13\text{-}BDF)^{-0.32}RQ_{100}^{\ 0.82}$
$UQ_{500} = 7.47A^{0.16}(13\text{-}BDF)^{-0.32}RQ_{500}^{\ 0.83}$

The nationwide 7-parameter equations for urban conditions take the following general form:*

$$UQ_T = CA^{b1}SL^{b2}(RI2+3)^{b3}(ST+8)^{b4}(13\text{-}BDF)^{b5}IA^{b6}RQ_T^{b7} \qquad (6.8.2)$$

Where:

UQ_T = Peak discharge for the urban watershed for recurrence interval T, cubic feet per second (cfs)

C = Regression constant (dimensionless)
A = Contributing drainage area, square miles (sq. mi.)
SL = Main channel slope, in ft./mi., measured between points that are 10% and 85% of the main channel length upstream from the study sight (for sites where SL is greater than 70 ft./mi., 70 ft./mi. should be used)
RI2 = Rainfall for the 2-hr., 2-yr. occurrence (based on NOAA's ATLAS-14), in inches
ST = Basin storage, the percentage of the drainage basin occupied by lakes, reservoirs, swamps, and wetlands (channel storage of a temporary nature, resulting from detention ponds or roadway embankment, is not included in the computation of ST)
BDF = Basin development factor (dimensionless)
IA = Percentage of the drainage basin occupied by impervious surfaces (e.g., houses, buildings, streets, parking lots)
RQ_T = As described above
b1, b2, b3, b4, b5, b6, b7 = Regression exponents

Lists the 7-parameter equations for various storm frequencies.

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Table 6-6.2. 7-Parameter Urban Equations

$UQ_2 = 2.35A^{0.41}\ SL^{0.17}\ (R12+3)^{2.04}\ (ST+8)^{-0.65}\ (13\text{-}BDF)^{-0.32}\ IA^{0.15}\ RQ_2^{0.47}$
$UQ_5 = 2.7A^{0.35}\ SL^{0.16}\ (RI2+3)^{1.86}\ (ST+8)^{-0.59}\ (13\text{-}BDF)^{-0.31}\ IA^{0.11}\ RQ_5^{0.54}$
$UQ_{10} = 2.99A^{0.32}\ SL^{0.15}\ (RI2+3)^{1.75}\ (ST+8)^{-0.57}\ (13\text{-}BDF)^{-0.30}\ IA^{0.09}\ RQ_{10}^{0.58}$
$UQ_{25} = 2.78A^{0.31}\ SL^{0.15}\ (RI2+3)^{1.75}\ (ST+8)^{-0.55}\ (13\text{-}BDF)^{-0.29}\ IA^{0.07}\ RQ_{25}^{0.60}$
$UQ_{50} = 2.67A^{0.29}\ SL^{0.15}\ (RI2+3)^{1.74}\ (ST+8)^{-0.53}\ (13\text{-}BDF)^{-0.28}\ IA^{0.06}\ RQ_{50}^{0.62}$
$UQ_{100} = 2.5A^{0.29}\ SL^{0.15}\ (RI2+3)^{1.76}\ (ST+8)^{-0.52}\ (13\text{-}BDF)^{-0.28}\ IA^{0.06}\ RQ_{100}^{0.63}$
$UQ_{500} = 2.27A^{0.29}\ SL^{0.16}\ (RI2+3)^{1.86}\ (ST+8)^{-0.54}\ (13\text{-}BDF)^{-0.27}\ IA^{0.05}\ RQ_{500}^{0.63}$

6.4.4.6 Analysis of Stream Gage Data

6.4.4.6.1 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.
- 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.

6.4.4.6.2 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 should be used for analysis and the Gumbel graphical method used as a check to

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ensure errors are not made – especially in the estimation of larger floods. Where serious discrepancies (20%+) are encountered in the findings between the two methods, special studies may be required. These special studies should consist of comparison with regression equations, application of other flood-frequency methods, and the collection and analysis of historical data. Outliers should be examined using the procedure found in Water Resources Council Bulletin 17B.

The U.S. Geological Survey has developed a computer program entitled "PEAKFQWIN" for performing Log Pearson Type III computations. It is available for downloading at the following location: http://water.usgs.gov/software/PeakFQ/.

Gage data may be obtained from various publications including "Annual Maximum Stages And Discharges of Selected Streams In Virginia", prepared by the U.S. Geological Survey. For additional information contact: District* Chief, U.S. Geological Survey, Richmond, Virginia, phone (804) 261-2639. Historical NWIS-W gage data for Virginia (currently over 584 gaging locations) can also be obtained from the USGS world wide web site: <http://water.usgs.gov/nwis>

6.4.4.6.3 Skews

Skewness is a measure of asymmetry or lop-sidedness of a statistical distribution. The skew coefficient is defined as the skewness divided by the cube of the standard deviation. Skew coefficients play an integral role in the Log-Pearson analysis.

There are two alternative methods for determining the value of the skew coefficient to be used in calculating the Log-Pearson curve fit. The value of skew that is calculated directly from the gage data is called the station skew. This value may not be a true representation of the actual skew of the data if the period of record is short or if there are extreme events in the period of record. WRC Bulletin 17B contains a map of generalized skew coefficients of the logarithms of annual maximum streamflows throughout the United States and average skew coefficients by one degree quadrangles over most of the country.

Often, the station skew and the generalized skew can be combined to provide a better estimate for a given sample of flood data. Bulletin 17B outlines a procedure for combining the station skew and the generalized skew to provide a weighted skew.

6.4.4.6.4 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 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

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results are averaged to obtain a value for the desired flood frequency relationships in the ungaged watershed. The following example uses an exponent of 0.8.

Table 6-7. Transposition of Data Sample Problem

Watershed	Q_{25}, cfs	Area, sq. mi
Gaged Watershed A	62,000	737.0
Gaged Watershed B	38,000	734.0
Gaged Watershed C	45,000	971.0
Ungaged Watershed D	Find Q_{25}	449.8

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

$$A:\ 62{,}000\left(\frac{449.8}{737.0}\right)^{0.8} = 41{,}766 \text{ cfs } (41{,}800 \text{ cfs})$$

$$B:\ 38{,}000\left(\frac{449.8}{734.0}\right)^{0.8} = 25{,}682 \text{ cfs } (25{,}700 \text{ cfs})$$

$$C:\ 45{,}000\left(\frac{449.8}{971.0}\right)^{0.8} = 24{,}314 \text{ cfs } (24{,}300 \text{ cfs})$$

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

$$D:\ Q_{25} = \frac{(41{,}766+25{,}682+24{,}314)}{3} = 30{,}587 \ (30{,}600 \text{ cfs})$$

6.4.5 Hydrograph Methods

6.4.5.1 Modified Rational Method

6.4.5.1.1 Introduction

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

6.4.5.1.2 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 acres.

6.4.5.1.3 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.

6.4.5.1.4 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.

6.4.5.1.5 Estimating the Critical Duration Storm

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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.

6.4.5.1.6 Equations

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

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

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

6.4.5.2 SCS Unit Hydrograph

6.4.5.2.1 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,

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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*.

6.4.5.2.2 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.

6.4.5.2.3 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.

6.4.5.2.4 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:

$$L = 0.6t_c \tag{6.10}$$

6.4.5.2.5 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.

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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 your local NRCS office or:

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.

6.4.5.2.6 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 affects 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

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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-yr. 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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6.5 Design Procedures and Sample Problems

6.5.1 Documentation Requirements

These items establish a minimum requirement. Also, see Chapter 3, Documentation. The following items used in the design or analysis should be included in the documentation file:

- Contributing watershed area size and identification of source (map name, etc.)
- Design frequency and decision for selection
- Hydrologic discharge and hydrograph estimating method and findings
- Flood frequency curves to include design, 100-year flood, discharge hydrograph and any historical floods
- Expected level of development in upstream watershed over the anticipated life of the facility (include sources of and basis for these development projections)

6.5.2 Peak Discharge Procedures and Sample Problems

6.5.2.1 Rational Method Procedure

The results of using the rational formula to estimate peak discharges is very sensitive to the parameters that are used. The designer must use good engineering judgment in estimating values that are used in the method. The following procedure should be used for the Rational Method:

Step 1: Gather background information such as topographic mapping, land use data, precipitation information, etc. and determine point of analysis

Step 2: Delineate drainage area and determine the various land use characteristics

Step 3: Determine areas of overland flow, channel flow, and pipe conveyance and check results to verify reasonable flow time and velocity

Step 4: Compute total time of concentration (t_c)

Step 5: Determine rainfall intensity, I, using the procedure described in Appendix 6C-1 and the B, D, & E factors in Appendix 6C-2[*]*.*

Step 6: Determine runoff coefficient(s), C, from Appendix 6E-1

Step 7: Compute peak discharge ($Q_{p.}$)

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6.5.2.1.1 Rational Method Sample Problem

Preliminary estimates of the maximum rate of runoff are needed at the inlet to a culvert for a 10-year and 100-year return period.

Step 1: Gather background information such as topographic mapping, land use data, precipitation information, etc. and determine point of analysis

Site Data: Richmond Area

Step 2: Delineate drainage area and determine the various land use characteristics

<u>Drainage Area:</u>
From a topographic map and field survey, the area of a drainage basin upstream from the point in question is found to be 90 acres.

<u>Land Use:</u>
From existing land use maps, land use for the drainage basin was estimated to be:

Residential (1/2 acre lots)	80% of total drainage area
Undeveloped (2% slope)	20% of total drainage area

Step 3: Determine areas of overland flow, channel flow, and pipe conveyance and check results to verify reasonable flow time and velocity

Overland Surface = Average Grass
Length of Overland Flow = 150 feet
Average Overland Slope = 2.0%
Length of Main Basin Channel = 2300 feet
Slope of Channel = 1.8%

Step 4: Compute total time of concentration (t_c)

<u>Overland Flow:</u>
A runoff coefficient (C) for the overland flow area is determined from Appendix 6E-1.

C = 0.30

Analyze overland flow (OLF) and channel flow (CF) time by all applicable methods.

Overland flow time by:

Appendix 6D-1 (Seelye) = 13 minutes
Appendix 6D-3 = 15 minutes

The two methods essentially agree, therefore, use OLF = 14 minutes (*average*)

Channel Flow:

Channel flow by:

Appendix 6D-5(Kirpich) = 14 minutes

Use CF = 14 minutes

Time of Concentration:
Total t_c = 14 minutes (OLF) + 14 minutes (CF) = 28 minutes

Step 5: *Determine rainfall intensity (i) using the equation shown in Appendix 6C-1 and*[*] *the B, D, & E factors obtained from Appendix 6C-2.*

From Appendix 6C-2 (for the city of Richmond)) read, for a 10-yr. storm duration, B = 47.91, D = 9.25, & E = 0.72. Then read, for a 100-yr. storm duration, B = 33.15, D = 5.25, & E = 0.56. Using a duration equal to 28 minutes, determine rainfall intensity, (i), for both storm durations by using the appropriate B, D, & E factors, and solving Equation 6.14.

$$i = B / (t_c + D)^E \qquad (6.14)$$

i_{10} (10-year return period) = $47.91 / (28 + 9.25)^{0.72}$ = 3.54 in/hr

i_{100} (100-year return period) = $33.15 / (28 + 5.25)^{0.56}$ = 4.66 in/hr

Step 6: *Determine runoff coefficients (C) from Appendix 6E-1*

The weighted runoff coefficient (C) for the total drainage area is computed.

Land Use	Column 1 % of Total Land Area	Column 2 Runoff Coefficient	Column 3 Weighted Runoff Coefficient*
Residential (1/2 Acre lots)	0.80	0.35	0.28
Undeveloped	0.20	0.30	0.06
Total Weighted Runoff Coefficient			0.34

*Column 3 = Column1 x Column 2

Step 7: *Compute peak discharge (Q_p)*

$$Q = C_f CiA$$

Determine Coefficient of Saturation, C_f , from Table 6-2:

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Q_{10}; $C_f = 1.0$
Q_{100}; $C_f = 1.25$

Check $C_f \times C \leq 1.0$:

Q_{10}; $C \times C_f = 0.34 \times 1.0 = 0.34 < 1.0$, therefore, OK
Q_{100}; $C \times C_f = 0.34 \times 1.25 = 0.43 < 1.0$, therefore, OK

Determine Discharges:

$Q_{10} = 0.34 \times 3.54$* $\times 90 = 108$ cfs
$Q_{100} = 1.25 \times 0.34 \times 4.66 \times 90 = 178$ cfs

These are the estimates of peak runoff for a 10-year and 100-year design storm for the given drainage area.

6.5.2.2 Anderson Method Procedure

6.5.2.2.1 Anderson Method Sample Problem

Estimate the 25-year peak discharge on Rabbit Branch near Burke, Virginia, for an expected future development consisting of 40% impervious surface and a drainage system of storm sewers for tributaries but a natural main channel.

Step 1: From topographic maps, determine the following data

A = 3.81 sq. miles
L = 3.40 miles from crossing site to watershed boundary
Elevation = 282 feet at 10% L (0.34 miles) above crossing site
395 feet at 85% L (2.90 miles) above crossing site

Step 2: Compute the average channel slope

$$\text{Slope} = \frac{(395-282)}{(2.90-0.34)} = \frac{113}{2.56} = 44.2 \text{ ft/mi}$$

Step 3: Compute time (T)

$$T = 0.9\left(\frac{L}{\sqrt{S}}\right)^{0.5} \textit{ for Developed Watershed, Partly Channeled}$$

$$= 0.9\left(\frac{L}{\sqrt{S}}\right)^{0.5} = 0.9\left(\frac{3.4}{\sqrt{44.2}}\right)^{0.5} = 0.64 \text{ hrs}$$

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Step 4: Compute K

$$K = 1.00 + 0.015\mathbf{I}, \text{ where } \mathbf{I} = 40\% \text{ for impervious surface}$$
$$= 1.00 + 0.015(40)$$
$$= 1.60$$

Step 5: Compute Flood Ratio (Rf) using equation from section 6.4.4.2.4 with a 25-yr R_N of 3.3 and a 100-yr R_{100} of 1.8

$$R_{25} = [R_N + 0.01\mathbf{I}(2.5R_{100} - R_N)] / K$$
$$= [3.3 + 0.01(40)(2.5(1.8) - 3.3)]/1.6$$
$$= 2.36^{*}$$

Step 6: Compute peak discharge (Q)

$$Q_{25} = R(230)KA^{0.82}T^{-0.48}$$
$$= 2.36(230)(1.6)(3.81)^{0.82}(0.64)^{-0.48}$$
$$= 3222 \text{ cfs (Say 3200 cfs)}$$

6.5.2.3 Snyder Method Procedure

The procedure to develop peak discharges using the Snyder Method is provided below.

Step 1: Obtain the following data from topographic maps.

DA	=	Watershed area, square miles
L	=	Length of principal channel, miles
S	=	Weighted slope of channel (mean height in feet of channel profile divided by $L \div 2$), percent
N	=	Friction factor, Manning's "n"
Mean height	=	(elevation at upper end of channel – elevation of lower end of channel) $\div 2$

% of watershed sewered
% of natural channel eliminated
% of impervious area

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Step 2: Compute T_c

$$T_c = C_t L'^{0.6} \qquad (6.16)$$

Where:

C_t = Adjustment factor defined by the development condition of the drainage area

C_t is defined in Table 6-8. This table is for natural basins, overland flow, and completely sewered areas. If the drainage area is partially sewered, determine C_t from Table 6-8 and then compute Adj. C_t from Equation 6.17.

Partially Sewered Area Adjustment:

$$\text{Adj } C_t = 1.7 - \frac{(\%\text{ Sewered} + \%\text{ Nat. Chan. Elim.})(1.7 - 0.42)}{200} \qquad (6.17)$$

Table 6-8. C_t Values for the Snyder Method

Type of Areas	C_t , Hours/mile
Natural Basins	1.7
Overland Flow	0.85
Sewered Areas	0.42

L' = Equivalent length of channel with slope of 1% and friction factor equal to 0.1 and is defined by Equation 6.18.

$$L' = \frac{10Ln}{\sqrt{S}} \qquad (6.18)$$

Step 3: Compute peak discharge, (Q_p) using the following steps:

Step 3a: Determine storm frequency (years)

Step 3b: Using Appendix 6C-2, determine precipitation (inches) for each frequency by adjusting the results of Equation 6.14 to give total rainfall.*

Step 3c: Using Appendix 6G-1, determine percent natural runoff for each frequency

Step 3d: Using Appendix 6G-2, determine percent runoff adjusted for impervious area

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Step 3e: Compute I_R

$$I_R = \left[\frac{\text{Runoff}}{T_c}\right] \tag{6.19}$$

Step 3f: Compute Q_p

$$Q_p = 500AI_R$$

6.5.2.3.1 Snyder Method Sample Problem

Find the 10- and 100-yr peak discharges for a tributary draining into the James River near Richmond, VA. The tributary is primarily undeveloped; however, several developments within the drainage basin have storm drains.

Step 1: Obtain the following data from topographic maps.

Site: A tributary of the James River near Richmond, VA.

DA = 4.11 sq. mi.
L = 2.75 mi.
S = 53 ft./mi.=l.0%
n = 0.045.

DA is 22% impervious
10% Natural Channel Eliminated
30% Storm Sewered

Step 2: Compute T_c

$$T_c = C_t L'^{0.6}$$

$$L' = \frac{10Ln}{\sqrt{S}} \qquad \text{Adj } C_t = 1.7 - \frac{(\%\,\text{Sewered} + \%\,\text{Nat. Chan. Elim.})(1.7-0.42)}{200}$$

Substitute Equations 6.17 and 6.18 into Equation 6.16.

$$T_c = \left[1.7 - \frac{(\%\,\text{Sewered} + \%\,\text{Nat. Chan. Elim.})(1.7-0.42)}{200}\right]\left[\frac{10Ln}{\sqrt{S}}\right]^{0.6}$$

$$= \left[1.7 - \frac{(30+10)(1.7-0.42)}{200}\right]\left[\frac{10(2.75)(0.045)}{\sqrt{1.0}}\right]^{0.6}$$

$$= 1.64 \text{ hrs}$$

Step 3: Compute peak discharge (Q_p) using the following steps:

The results of Steps 3a through 3g are summarized in Step 3g.

Step 3a: Determine storm frequency (years)

Step 3b: Using Appendix 6C-2[*] , determine precipitation (inches) for each frequency by adjusting results of Equation 6.14 to give total rainfall.

From Appendix 6C-2 (for the city of Richmond read, for a 10-yr. storm duration, B = 47.91, D = 9.25, & E = 0.72. Then read, for a 100-yr. storm duration, B = 33.15, D = 5.25, & E = 0.56. Using a duration equal to 28 minutes, determine rainfall intensity, (i), for both storm durations by using the appropriate B, D, & E factors, and solving Equation 6.14.

i_{10} (10-year return period) = $47.91 / (98.4^{*} + 9.25)^{0.72} = 1.65$ in/hr

i_{100} (100-year return period) = $33.15 / (98.4^{*} + 5.25)^{0.56} = 2.46$ in/hr

[*] T_c of 1.64 hours converted to minutes

$R_{10} = 1.64\ (1.65) = 2.7$ inches

$R_{100} = 1.64\ (2.46) = 4.0$ inches

Step 3c: Using Appendix 6G-1, determine percent natural runoff for each frequency

Step 3d: Using Appendix 6G-2, determine percent runoff adjusted for impervious area

Step 3e: Compute I_R

$$I_R = \left[\frac{\text{Runoff}}{T_c}\right]$$

Step 3g: Compute Q_p

$$Q_p = 500AI_R$$

Frequency (yrs.)	10	100
Rainfall (inches)	2.7	4.0
Percent Runoff Natural (%) (Appendix . 6G-1)	31	42
Percent Runoff Adjusted (%) (Appendix 6G-2)	46	54
Runoff (Runoff Adj. x Rainfall)	1.24	2.16
$I_R = [\text{Runoff} \div T_c]$ (in/hr)	0.76	1.32
Q_p = 500 AI_R, (cfs)	**1562 (1560 cfs)**	**2713 (2710 cfs)**

[*] Rev 9/09

6.5.2.4 Rural Regression Method Procedure

The rural regression method procedure is presented in the urban regression method procedure and sample problem discussed in Section 6.5.2.5.

6.5.2.5 Urban Regression Method Procedure

This procedure is not intended to require precise measurements. A certain amount of subjectivity is involved, and field checking should be performed to obtain the best estimate. The BDF is the sum of four assigned codes; therefore, with three sub-basins per basin, and four drainage characteristics to which codes are assigned in each sub-basin, the maximum value for a fully developed drainage system would be 12. Conversely, a totally undeveloped drainage system would receive a BDF of zero (0). This rating does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, and have some improvement of secondary tributaries, and still have an assigned BDF of zero (0). The following steps are used to apply the nationwide urban* equations:

Step 1: Determine the following input parameters: drainage area (A), main channel slope (SL), rainfall intensity (RI2), basin storage (ST), and impervious area (IA).

Step 2: Using the USGS regression equations, estimate peak runoff for natural flow conditions for an equivalent rural drainage basin (RQ_T) for a specific design frequency. Depending on the amount of data available, use Tables 6-4 and 6-5.

Step 3: Determine the basin development factor (BDF)

Step 4: Calculate peak runoff rates for desired return periods using the urban regression equations given in Tables 6-6.1 and/or 6-6.2. Note: where sufficient information is available, it's probably best to use both sets of equations and use the results that appear to be the most reasonable.

6.5.2.5.1 Urban Regression Method Sample Problem

Determine the peak discharge for the 10- and 100-yr design storms using both the 3 and 7-parameter urban regression equations for a drainage area of one (1) square mile in northern Spotsylvania County, VA. The main channel slope is 52 ft/mi and the channel length is 1.3 miles. The current land use is a mixture of rural cultivation and woods.

A plan is offered that will provide single family and commercial development within portions of this watershed. It is expected that this development will increase the watershed's percentage of imperviousness to 3%. There will be no Basin Storage.

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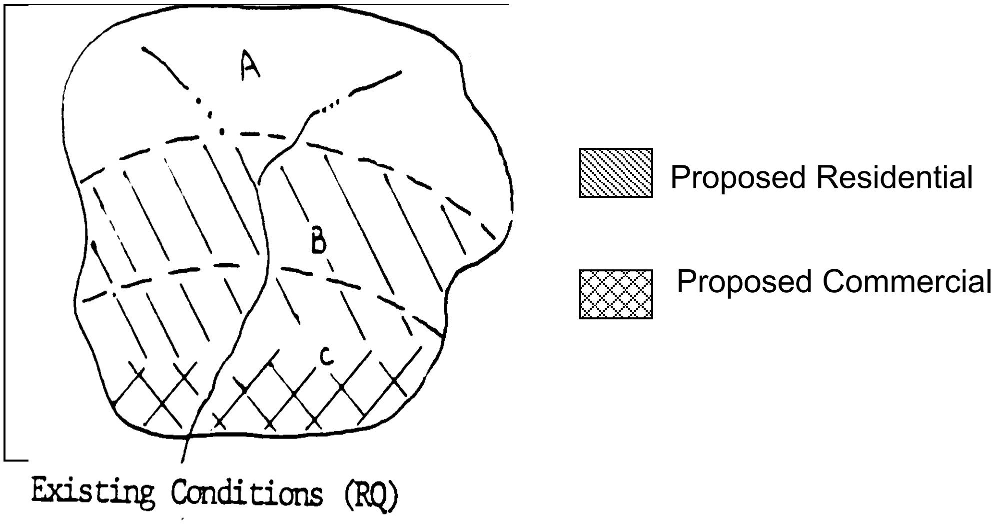

Figure 6-5. Basin Sketch

Step 1: Determine the following input parameters: drainage area (A), main channel slope (SL), rainfall intensity (RI2), basin storage (ST), and impervious area (IA).

Step 2: Using the USGS regression equations, estimate peak runoff for natural flow conditions for an equivalent rural drainage basin (RQ_T) for a specific design frequency.

From Table 6-5, obtain the USGS Rural Regression Equations for the Northern Piedmont and compute the 10-yr and 100-yr peak discharges for the existing conditions.

$Q_{10} = 438\ (1^{.641}) = 438$ cfs
$Q_{100} = 983$* $(1^{.641}) = 983$ cfs

Step 3: Determine the basin development factor (BDF) by subdividing the drainage area into thirds, illustrated in Figure 6-5, and assign a BDF code (0 or 1) based on the presence of four characteristics of the drainage system within each third.

The drainage area is subdivided into thirds as shown in Figure 6-5.

Proposed Conditions BDF:

	A	B	C
Storm drains	0	0	1
Channel improvements	0	0	1
Impervious channel linings	0	1	1
Curb & gutter	0	1	1
	0	2	4

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Total BDF =6

Step 4: Calculate peak runoff rates for desired return periods using the 3-parameter[*] urban regression equations given in Table 6-6.1

$UQ_{10} = 9.51(1)^{0.16}\ (13\text{-}6)^{-0.36}\ (438)^{0.79} = 576$ cfs
$UQ_{100} = 7.7(1)^{0.15}\ (13\text{-}6)^{-0.32}\ (983)^{0.82} = 1{,}176$ cfs

Step 5: Determine the 2-hr., 2-yr rainfall using the "B, D, & E" factors from Appendix 6C-2.

B = 65.52, D = 13.25, & E = 0.88, Storm Duration = 2 hrs. (120 minutes)

$RI2 = 2\ (65.52 / (120 + 13.25)^{0.88}) = 1.77$ inches

Step 6: Calculate peak runoff rates for desired return periods using the 7-parameter urban regression equations given in Table 6-6.2

$UQ_{10} = 2.99\ (1)^{0.32}\ (52)^{0.15}\ (1.77+3)^{1.75}\ (0+8)^{-0.57}\ (13\text{-}6)^{-0.30}\ (3)^{0.09}\ (438)^{0.58} = 533$ cfs

$UQ_{100} = 2.5\ (1)^{0.29}\ (52)^{0.15}\ (1.77+3)^{1.76}\ (0+8)^{-0.52}\ (13\text{-}6)^{-0.28}\ (3)^{0.06}\ (983)^{0.63} = 1141$ cfs

6.5.2.6 Analysis of Stream Gage Data Procedure

6.5.2.6.1 Statistical Method for Analyzing Stream Gage Data

The log-Pearson Type III distribution is the recommended statistical method. This method is defined by three standard statistical parameters: the mean, standard deviation and coefficient of skew. These parameters are determined from the data sample, which normally consists of the peak annual flows for a period of record. Formulas for the computation of these parameters are given below:

$$Q = \frac{(\sum X)}{N} \text{ (mean of logs)} \qquad (6.20)$$

Where:

N = Number of observations and X is the logarithm of the annual peak

The standard deviation of logs is:

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$$S_L = \left(\frac{\left[\Sigma X^2 - \frac{(\Sigma X)^2}{N} \right]}{[N-1]} \right)^{\frac{1}{2}} \tag{6.21}$$

The coefficient of skew of logs is:

$$G = \frac{\left[N^2\left(\Sigma X^3\right) - 3N(\Sigma X)\left(\Sigma X^2\right) + 2(\Sigma X)^3 \right]}{\left[N(N-1)(N-2)S_L^{\ 3} \right]} \tag{6.22}$$

Using these three parameters, the magnitude of the flood of the desired frequency can be determined from the equation.

$$\log Q = Q_L + KS_L \tag{6.23}$$

Where:

log Q = Logarithm of the flood magnitude
Q_L = Mean of the logarithms of the peak annual floods
K = Frequency factor for a particular return period and coefficient of skew (Values of K for different coefficients of skew and return periods are given in WRC Bulletin 17B)
S_L = Standard deviation of the logarithms of the peak annual flood

If a flood frequency curve is necessary, then by computing several values of Q for different return periods, the log-Pearson fit to the data can be plotted on standard log probability paper. If the skew of the sample data happens to be equal to zero, the plot of the log-Pearson fit to the data will be a straight line. If the skew is negative the plot will be a curve with a downward concavity. If the skew is positive, the plot will be a curve with upward concavity.

6.5.3 Hydrograph Procedures and Sample Problems

6.5.3.1 Modified Rational Method Hydrograph Procedure

Use of the Rational Method for calculation of peak Q is based upon the assumption that the highest Q from a small drainage area occurs when the duration (D_e) of the storm is equal to the time of concentration (t_c) or $D_e = t_c$.

Step 1: Obtain all of the basic background information that is required for the Rational Method. Refer to section 6.5.2.1, Rational Method procedures

Step 2: Compute the time of concentration (t_c) as described in the Rational Method

Step 3: Develop a range of hydrographs for various durations (D_e):

a. t_c
b. $1.5t_c$
c. $2t_c$
d. $3t_c$

Note: 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.

Step 4: Determine storm duration (D_e) for each scenario

Step 5: Compute Q_p, T_b, and D_e for each scenario

6.5.3.1.1 Modified Rational Method Hydrograph Sample Problem

For a 5 acre drainage site with a post development C-value of 0.7 and a time of concentration of 20 min, develop hydrographs (HYG) for a 2-year frequency storm in the Richmond area.

Step 1: Obtain all of the basic background information that is required for the Rational Method

DA = 5 acres
C = 0.7 (after development)

Step 2: Compute the time of concentration (t_c) as described in the Rational Method

Time of concentration (t_c) = 20 minutes

Step 3: Develop a range of hydrographs for various durations (D_e)

a. t_c
b. $1.5t_c$
c. $2t_c$
d. $3t_c$

The results of steps 3 through 3b are summarized after Step 3b and shown graphically in Figure 6-6[*].

Step 3a: Determine storm duration (D_e) and time base (T_b) for each scenario

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All hydrographs for this drainage area will have T_p = 20 minutes and also T_r = 20 minutes because t_c = 20 minutes for this drainage area.

Step 3b: Compute Q_p for each scenario

HYG #1 Duration (D_e) = 20 minutes
I = 57.69 / (20 + 11.50) $^{0.85}$ *(using B-D-E factors from Appendix 6C-2)*
Intensity (I) = 3.1 in/hr
Peak Q = 0.7 x 3.1 x 5 = 10.9 or 11 cfs
T_b = 20(or D_e) + 20(or T_r) = 40 minutes

HYG #2 D_e = 1.5 x t_c = 1.5 x 20 = 30 minutes
I = 57.69 / (30 + 11.50) $^{0.85}$ *(using B-D-E factors from Appendix 6C-2)*
Intensity (I) = 2.4 in/hr
Peak Q = 0.7 x 2.4 x 5 = 8.4 or 8 cfs
T_b = 30(or D_e) + 20(or T_r) = 50 minutes

HYG #3 D_e = 2 x t_c = 2 x 20 = 40 minutes
I = 57.69 / (40 + 11.50) $^{0.85}$ *(using B-D-E factors from Appendix 6C-2*
Intensity (I) = 2.0 in/hr
Peak Q = 0.7 x 2.0 x 5 = 7 cfs
T_b = 40(or D_e) + 20(or T_r) = 60 minutes*HYG #4 D_e = 3 x t_c = 3 x 20 = 60 minutes*

HYG #4 D_e = 3 x t_c = 3 x 20 = 60 minutes[*]
I = 57.69 / (60 + 11.50) $^{0.85}$ *(using B-D-E factors from Appendix 6C-2)*
Intensity (I) = 1.5 in/hr
Peak Q = 0.7 x 1.5 x 5 = 5.2 or 5 cfs
T_b = 60(or D_e) + 20(or T_r) = 80 minutes

[*] Rev 9/09

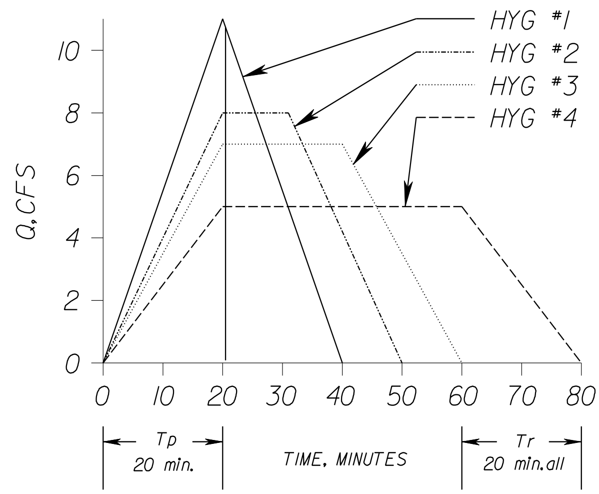

Figure 6-6. Example of Hydrographs for 2-year Frequency Storm Using the Modified Rational Method

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